

NATIONAL HIGHWAYS & INFRASTRUCTURE DEVELOPMENT CORPORATION LTD.

(MINISTRY OF ROAD TRANSPORT & HIGHWAYS, GOVT. OF INDIA)
3RD FLOOR, PTI BUILDING, 4-PARLIAMENT STREET, NEW DELHI – 110001



Consultancy Services for Feasibility Study, Preparation of Detailed Project Report and providing pre-construction services for 2-lane with Paved Shoulder of NH configuration for Baramulla-Gulmarg Road in the state of Jammu & Kashmir.



DRAFT DETAILED PROJECT REPORT VOLUME-II B: DESIGN REPORT STRUCTURES

June 2020

Rodic Consultants Pvt. Ltd. In JV with
Monarch Surveyors and Engineering Consultants Pvt. Ltd.



DESIGN OF ABUTMENT(A1) WITH
OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
16+955

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 = 40 kmph

	Right Dimensions	Skew Dimensions
Span -c/c of Brg.	= 13.750m	13.750m
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.	= 15.000m	15.00m
Type of Superstructure	= PSC precast Girder	
Width of Crash barrier (Both Side)	= 0.500m	
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	= 0.000m	
Railing/kerb on footpath edge	= 0.000m	
Total Width of Superstructure	= 8.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 3
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 2.800m (Right) 2.800 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 300 kN/m² Assumed
SBC of soil-Normal Case = 350 kN/m²
SBC of soil-Seismic Case = 438 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	= 1.885m	
Height of Tapering portion	= 0.106m	
Length of Dirt Wall at top (Uniform portion)	= 8.500m	8.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 8.500m	8.500m

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	

Abutmentcap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	8.500m	8.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	8.500m	8.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	8.500m	8.500m
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	=	4.200m
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	=	0.000m
Thickness of Return wall at Bottom	=	0.000m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	7.700m
abutment pedestal width	=	1.000m
abutment pedestal Height	=	1.000m
Width of Toe Slab	=	2.500m
Width of Heel Slab	=	4.200m
Thickness of Toe slab at tip	=	0.300m
Thickness of Toe slab near shaft	=	1.000m
Thickness of heel slab at tip	=	0.300m
Thickness of heel slab near shaft	=	1.000m
Width of backfill on heel slab	=	4.200m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	8.311m
Height of back fill at back fill edge of heel slab	=	7.611m

Across Traffic Direction:

Width of foundation -Uniform portion	=	8.500m (skew dimension)
Width of foundation -Tapering portion	=	8.500m (skew dimension)

Levels

Deck Level at Median Edge=	1762.587m	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	1762.495m	Height of Superstructure	=	1.420m
Deck level at center line =	1762.587m	Min. Height of Footpath Side Pedestal (1)	=	0.400m
Soffit Level at center of bridge =	1761.102m	Height of Pedestal (2)	=	0.470m
Abutment cap top level =	1760.550m	Height of Pedestal (3)	=	0.540m
Abutment cap bottom lvl (uniform portion ends)	1760.250m	Height of Pedestal (4)	=	0.000m
Abutment cap bottom lvl (corbel portion ends)	1759.950m	Height of Pedestal (5)	=	0.000m
Abutment shaft top level =	1759.950m	Height of Pedestal (6)	=	0.000m
Ground level/LBL =	1758.009m	Distance of nearest girder to c.l. of deck	=	0.000m
Abutment shaft bottom level =	1754.930m	Height (Avg.) of Dirt Wall	=	1.991m
Foundation level =	1753.930m	Abutment shaft Above G.L	=	1.940m
HFL =	1760.009m	Abutment Shaft below G.L	=	3.079m
		Height of abutment shaft	=	5.019m
		MSL	=	1755.930m
		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	=	5	
Z = Zone factor	=	0.36	
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	=	272.61 2.80
DL Reaction on Bearing No.-2	=	269.86 0.00
DL Reaction on Bearing No.-3	=	272.61 -2.80
DL Reaction on Bearing No.-4	=	0.00 0.00

DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		815.07 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	130.441 2.80
SIDL Reaction on Bearing No.-2	=	-113.782 0.00
SIDL Reaction on Bearing No.-3	=	130.441 -2.80
SIDL Reaction on Bearing No.-4	=	0.000 0.00
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		147.10 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	38.646 2.80
SIDL Reaction on Bearing No.-2	=	46.046 0.00
SIDL Reaction on Bearing No.-3	=	38.646 -2.80
SIDL Reaction on Bearing No.-4	=	0.000 0.00
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		123.34 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:****(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	671.00	KN	Corr. Transv	=	775.001 kNm	ecc.	1.16
min	=	329.00	KN	Corr. Transv	=	379.999 kNm	ecc.	1.16

SV Loading

max	=	1801.96	KN	Corr. Transv	=	540.589 kNm	ecc.	0.30
min	=	1618.04	KN	Corr. Transv	=	485.411 kNm	ecc.	0.30

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

Reduction Factor = 1 (for 2 Lane)

max	=	671.00	KN	Corr. Transv	=	775.001 kNm	ecc.	1.16
min	=	329.00	KN	Corr. Transv	=	379.999 kNm	ecc.	1.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.114
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for CI A Wheeled loading

Impact Factor upto abut. cap	=	1.114
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	2.800
Pedestal 2	=	0.470 x	0.800 x	0.800 =	0.3008 m ³	0.000
Pedestal 3	=	0.540 x	0.800 x	0.800 =	0.3456 m ³	-2.800
Pedestal 4	=	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
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					0.9024 m ³	-0.278

VOLUME CALCULATION

C.G. Of Footing	=	3.850 m
C.G. Of shaft from toe tip	=	3.000 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	8.50	0.300	1.885	4.807	-0.800	0.050	-3.800	0.000
-Trapering portion	1	8.50	0.300	0.106	0.271	-0.800	0.050	-3.800	0.000
Bracket (Rectangle)	1	8.50	0.300	0.300	0.765	-1.100	-0.250	-4.100	0.000
(Corbel)	0.5	1	8.50	0.300	0.383	-1.050	-0.200	-4.050	0.000
Cap (uniform portion)	1	8.50	1.850	0.300	4.718	-0.025	0.825	-3.025	0.000
Cap (Corbel Portion)	1	8.50	1.850	0.300	3.579	-0.025	0.825	-3.025	0.000
		8.50	1.000						
Shaft above HFL	1	8.50	1.000	0.000	0.000	0.000	0.850	-3.000	0.000
Shaft below HFL	1	8.50	1.000	5.019	42.666	0.000	0.850	-3.000	0.000
Solid Return Wall	2	4.20	0.500	8.007	33.629	-2.600	-1.750	-5.600	0.000
Cantilever Return wall(Rectangular portion)	2	0.00	0.000	0.000	0.000	-0.500	0.350	-3.500	0.000
Cantilever Return wall(Traingular portion)	2	0.00	0.000	0.000	0.000	-0.500	0.350	-3.500	0.000
Footing									
Heel Slab	1	8.50	4.200	0.650	23.205		-1.373	-5.223	0.000
Toe Slab	1	8.50	2.500	0.650	13.813		2.376	-1.474	0.000
Portion between Heel and Toe	1	8.50	1.000	1.000	8.500		0.850	-3.000	0.000
Back filling above HFL over Heel Slab	1	8.50	4.200	2.578	92.035		-1.750	-5.600	0.000
Back filling below HFL over Heel Slab	1	8.50	4.200	5.429	193.815		-1.795	-5.645	0.000
Backfill above Heel slab	1	8.50	4.200	7.961	284.199		-1.781	-5.631	0.000
Front Filling over Toe Slab	1	8.50	2.500	3.429	72.866		2.557	-1.293	0.000
Front Filling over Toe Slab in HFL Case	1	8.50	2.500	1.350	28.688		1.642	-2.208	0.000
Side filling between heel and toe	1	0.00	1.000	3.429	0.000		0.000	0.000	0.000
Approach Slab	1	8.500	1.750	0.300	4.463	-1.100	-0.250	-4.100	0.000
Back fill above HFL on flared portion of stem	1	8.50	0.000	2.578	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	8.50	0.000	5.429	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	0.050	-3.800

SECTIONAL PROPERTIES

Width of Footing (B)	=	7.700 m
Length of Footing (L)	=	8.500 m

A	=	7.700	x	8.500	=	65.450	m ²
ZL	=	8.500	x	9.882	=	83.994	m ³
ZT	=	IT1	+	IT2			
		distance of extreme point from centre					
IT1	=	7.700	x	51.177	=	394.06	m ⁴
IT2 (moment of inertia of triangle)	=	7.700	x	0.000	+	0.500 x 7.700 x	0.000 x 18.063
from centre of footing	=	0.000	m ⁴				
Moment of inertia of two triangle	=	0.000	m ⁴				
Total moment of inertia	=	394.06	m ⁴				
Distance of extreme point from centre of footing	=	4.250	+	0.000	=	4.250	m
Total Section modulus (ZT)	=	92.721	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

5.0

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	=	0.36	
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

= 7.019 m

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

= 8.378 m

Abutment Cap Top Level - Foundation Level

= 6.619 m

Dimensions of Abutment Shaft

Length = 8.50 m

Width = 1.00 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 0.531 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 51.177 m^4

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

Longitudinal Direction

Force = 162.771 KN
 D = 1085.51 KN
 T = 0.1633 sec

Transverse Direction

Force = 12228.962 KN
 D = 1285.510 KN
 T = 0.0205 sec

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

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

$\frac{S_a}{g}$ = 1.308

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

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

$\frac{S_a}{g}$ = 1.308

For Soft Soil Strata, with $N < 10$

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

$\frac{S_a}{g}$ = 1.308

Medium Soil Strata

S_a/g = 2.5

S_a/g = 1.308

Seismic Coeff. In Longitudinal Direction = 0.18

Seismic Coeff. In Transverse Direction = 0.282

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.180	In Longitudinal direction
Ah	=	0.282	In Transverse direction
Av	=	0.188	In Vertical direction

For Design of Foundation

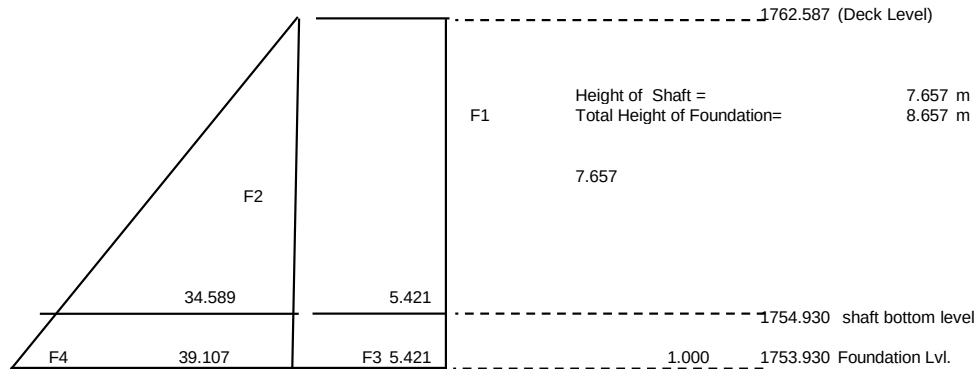
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.243	In Longitudinal direction
Ah	=	0.282	In Transverse direction
Av	=	0.188	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					1754.930 m (at Shaft Base)		
@ RL					1753.930 m (at Foundation Level)		
Due to Live Load Surcharge							
Intensity for rectangular portion	=	0.226	x	20	x	1.2	= 5.421 kN/m^2
F1	=	5.421	x	7.657	x	8.500	= 352.811 kN
M1	=	352.81	x	3.83	=	1350.735	kN.m at Shaft Bottom
F3	=	5.421	x	8.657	x	8.500	= 398.887 kN
M3	=	398.887	x	4.328	=	1726.584	kN.m at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	7.657	= 34.589 kN/m ²
F2	=	0.5	x	34.59	x	7.657	x 8.50
	=	1125.613	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	=	1125.61	x	3.22	=	3619.902	kN.m at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	8.657	= 39.107 kN/m ²
F4	=	0.5	x	39.11	x	8.657	x 8.50
	=	1438.820	kN				
M4	=	1438.82	x	3.64	=	5231.463	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	7.657	=	36.754	kN/m ²
F	=	0.5	x	36.754	x	7.657	x 8.500
	=	1196.045	kN				
M	=	1196.04	x	2.552	=	3052.705	kN.m at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	4.800	x	8.657	=	41.55	kN/m ²
F	=	0.5	x	41.554	x	8.66	x 8.500
	=	1528.850	kN				
M	=	1528.85	x	2.886	=	4411.753	kN.m at Foundation
Intensity of Passive pressure							
	=	5.249	x	20	x	0.000	= 0.000 kN/m ²
Force due to passive @ Foundation, F							
	=	0.5	x	0.000	x	0.000	x 8.50
	=	0.000	kN				

Moment due to passive @ Foundation, M
= 0.000 x

0.000 =

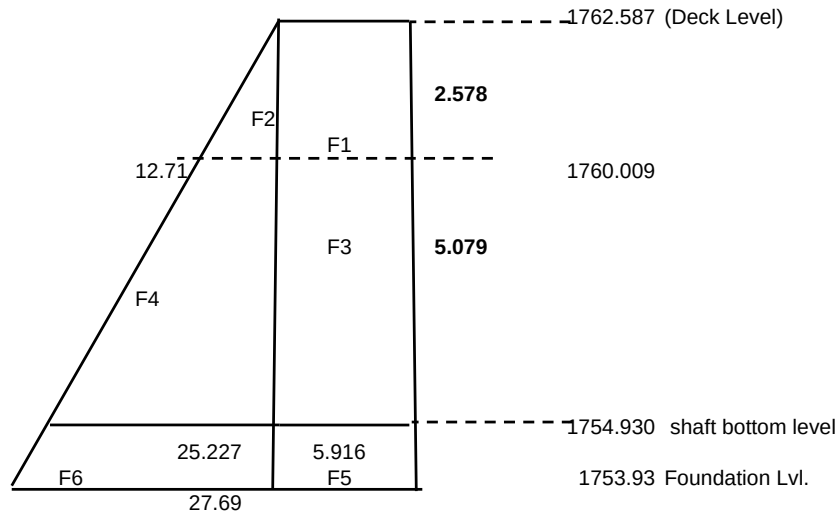
0.000 kN.m at Foundation

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	3619.902	5231.463	1125.613	1438.820
Due to Minimum Fluid Pressure	3052.705	4411.753	1196.045	1528.850
Governing of Two	3619.902	5231.463	1196.045	1528.850
Due to Live Load Surcharge	1350.735	1726.584	352.811	398.887
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****1754.9 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	7.657	x	8.500	=	385.009 kN
M1	=	385.01	x	3.83	=	1474.007 kN.m		at Shaft Bottom
F3	=	5.916	x	8.657	x	8.500	=	435.291 kN
M3	=	435.29	x	4.33	=	1884.157 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	2.578	=	12.709 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	5.079	=	12.519 kN/m ²
F2	=	0.5	x	12.71	x	2.578	x	8.50
	=	139.241 kN						
F4	=	$\left(\frac{12.71 + 25.23}{2} \right) \times$				5.08	x	8.50
	=	818.872 kN						
Total Force	=	958.113 kN						
M2	=	139.24	x	6.16	=	857.969 kN.m		
M4	=	818.87	x	2.26	=	1850.779 kN.m		
Total Moment	=	2708.75 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 2.578 = 12.709 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 6.079 = 14.984 \text{ kN/m}^2$$

$$F2 = 0.5 \times 12.71 \times 2.578 \times 8.50 = 139.241 \text{ kN}$$

$$F6 = \left(\frac{12.71 + 27.69}{2} \right) \times 6.08 \times 8.50 = 1043.779 \text{ kN}$$

$$\text{Total Force} = 1183.020 \text{ kN}$$

$$M2 = 139.24 \times 7.16 = 997.210 \text{ kN.m}$$

$$M6 = 1043.78 \times 2.66 = 2780.359 \text{ kN.m}$$

$$\text{Total Moment} = 3777.57 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

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

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 8.50 = 0.000 \text{ kN}$$

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	2708.749	3777.569	958.113	1183.020
Due to Minimum Fluid Pressure	3052.705	4411.753	1196.045	1528.850
Governing of Two	3052.705	4411.753	1196.045	1528.850
Due to Live Load Surcharge	1474.007	1884.157	385.009	435.291
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 5.0****Dynamic increment due to seismic force**

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

$$\begin{aligned} \alpha h &= 0.180 \\ \alpha v &= 0.188 \\ \phi &= 35.00 & 0.611 \\ \delta &= 22.50 & 0.393 \\ \alpha &= 0.00 & 0.000 \\ \beta &= 0.00 & 0.000 \end{aligned}$$

$$\begin{aligned} \alpha h &= \text{HORIZONTAL SEISMIC COEFFICIENT} \\ \alpha v &= \text{VERTICAL SEISMIC COEFFICIENT} \\ \phi &= \text{ANGLE OF INTERNAL FRICTION OF SOIL} \\ \delta &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL} \\ \alpha &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,} \\ \beta &= \text{SLOPE OF EARTH FILL} \end{aligned}$$

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.150 \\ 0.218 \end{matrix}$$

$$C_a = \begin{matrix} 1 \\ 0.377 \end{matrix} \begin{matrix} 2 \\ 0.302 \end{matrix}$$

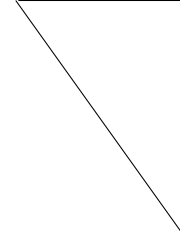
$$\begin{aligned} C_a &= 0.377 \\ K_a &= 0.226 \\ \text{Dynamic Increment} &= 0.377 \quad -0.226 \quad 0.151 \end{aligned}$$

3 Earth Pressure : DRY CASE (Seismic case)

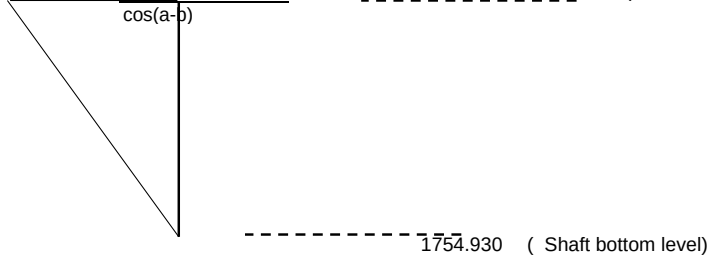
$$\begin{aligned} \text{Equivalent Live Load Surcharge height} &= 1.2 \text{ m} \\ \text{Assuming } \gamma_{\text{dry}} &= 20 \text{ kN/m}^3 \\ \gamma_{\text{water}} &= 10.00 \text{ kN/m}^3 \end{aligned}$$

Active Earth Pressure

$$3 (C_a - K_a) = 0.453$$

**Surcharge Earth Pressure**

$$2 (C_a - K_a) \frac{\cos a}{\cos(a-b)} = 0.30 \quad \text{--- 1762.587 (Deck Level)}$$



--- 1754.930 (Shaft bottom level)

--- 1753.930 (Foundation Bottom level)

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

$$= 0.302 \times 20.00 \times 1.2 = 7.251 \text{ kN/m}^2$$

at Shaft Bottom Level

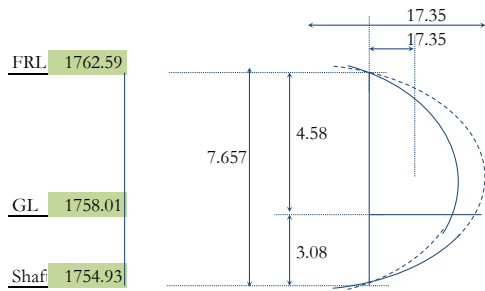
$$\begin{aligned} E1 &= 0.50 \times 7.251 \times 7.657 = 8.500 \\ &= 235.975 \text{ kN} \end{aligned}$$

$$M1 = 235.975 \times 5.130 = 1210.597 \text{ kN.m}$$

at Foundation Bottom Level

$$\begin{aligned} E2 &= 0.50 \times 7.251 \times 7.657 = 8.500 \\ &= 235.975 \text{ kN} \end{aligned}$$

$$M2 = 235.975 \times 6.130 = 1446.572 \text{ kN.m}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	17.35 t/m ²
h	=	7.657 m
y	=	0.7495 m
L	=	8.500 m

Parabola below Water Level

p_mid_height	=	17.35 t/m ²
h	=	7.657 m
y	=	-0.7495 m
L	=	8.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	485.6	4.9
Parabola below Water Level	267.3	2.0
Total Dynamic Earth Pressure	752.9	3.8

$$E3 = 752.859 \text{ kN}$$

$$E4 = 752.859 \text{ kN}$$

$$M3 = 752.86 \times 3.83 = 2882.319 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 752.86 \times 4.83 = 3635.178 \text{ kN.m (Foundation Bottom level)}$$

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)	3619.902	5231.463	1125.613	1438.820
Due to active Earth Pressure (dynamic Increment)	2882.319	3635.178	752.859	752.859
Total Earth Pressure	6502.222	8866.641	1878.471	2191.679
Due to Minimum Fluid Pressure	3052.705	4411.753	1196.045	1528.850
Governing of Two	3619.902	8866.641	1878.471	2191.679
Due to Live Load Surcharge (Static)	1350.735	1726.584	352.811	398.887
Due to Live Load Surcharge(Dynamic)	1210.597	1446.572	235.975	235.975
Due to Passive pressure		0.000		0.000

Earth Pressure : Normal HFL Case

αh	=	0.180			
αv	=	0.188			
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	=	$\tan^{-1} \frac{Ws}{Ws-1}$		$\frac{\alpha h}{(1 \pm \alpha v)}$	=
					0.294
					0.417
C_a'	=	1	2		
		0.550	0.515		
Ca'	=		0.550		
Ka'	=		0.246		
Dynamic Increment	=	0.550	-0.246	0.304	
Dynamic Increment	=			0.304	
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	=			7.70	m
Dynamic Active earth pressure Variation					
$3 (ca'-ka')$					
Dynamic Surcharge earth pressure Variation					
$2 (Ca'-Ka') \cos a$					1762.587 (Deck Level)
$\cos(a-b)$					
				2.578	
					1760.009 HFL
				5.079	
					1754.930 shaft bottom level
					1753.93 Foundation Bottom Lvl.

Earth Pressure Diagram

h'	=	5.079
h	=	7.657

Dynamic active earth pressure

$3 (ca'-ka')$	=	0.453	at deck level
$3 (ca'-ka')$	=	0.912	at deck level
$3 (ca'-ka')h'/h$	=	0.605	at HFL level

Dynamic Surcharge earth pressure

$2 (Ca'-Ka') \frac{\cos a}{\cos(a-b)}$	=	0.608	at deck level	Pressure	=	14.59	kN/m ²
	=	0.403	at HFL level	Pressure	=	9.68	kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	7.251	kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	9.68	kN/m ²

at Shaft Bottom Level

F1	=	8.464	x	2.578	x	8.500	=	185.464	kN
M1	=	185.46	x	6.37	=	1181.035	kN.m		
F2	=	0.50	x	9.676	x	5.079	x	8.500	
	=	208.863	kN						

$$M2 = 208.86 \times 3.40 = 710.746 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 394.327 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 1891.782 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

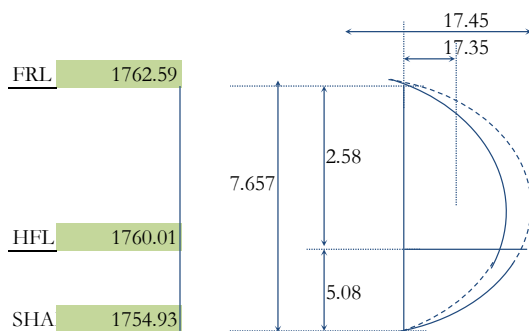
$$F3 = 8.464 \times 2.578 \times 8.500 = 185.464 \text{ kN}$$

$$M3 = 185.46 \times 7.37 = 1366.499 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 9.676 \times 5.079 \times 8.500 \\ &= 208.863 \text{ kN} \end{aligned}$$

$$M4 = 208.86 \times 4.40 = 919.609 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 394.327 \text{ kN} \\ \text{Total Moment at Founding level} &= 2286.109 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 17.35 \text{ t/m}^2 \\ h &= 7.657 \text{ m} \\ y &= -1.2505 \text{ m} \\ L &= 8.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 17.45 \text{ t/m}^2 \\ h &= 7.657 \text{ m} \\ y &= 1.2505 \text{ m} \\ L &= 8.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	198.6	6.0
Parabola below Water Level	557.5	3.1
Total Dynamic Earth Pressure	756.1	3.8

Total Force (F1 + F2)	=	756.093 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	756.093 kN	at Foundation Bottom Level

$$M1 = 756.09 \times 3.83 = 2892.184 \text{ kN.m}$$

$$\text{Total Mome} = 2892.184 \text{ kN.m} \quad \text{at Shaft Bottom}$$

$$M1 = 756.09 \times 4.83 = 3648.277 \text{ kN.m}$$

$$\text{Total Mome} = 3648.277 \text{ kN.m} \quad \text{at Foundation Bottom Level}$$

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)	2708.749	3777.569	958.113	1183.020
Due to active Earth Pressure (Dynamic Increment)	2892.184	3648.277	756.093	756.093
Total Earth Pressure	5600.933	7425.846	1714.206	1939.113
Due to Minimum Fluid Pressure	3052.705	4411.753	1196.045	1528.850
Governing of Two	5600.933	7425.846	1714.206	1939.113
Due to Live Load Surcharge(Static)	1474.007	1884.157	385.009	435.291
Due to Live Load Surcharge (Dynamic Increment)	1891.782	2286.109	394.327	394.327
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	=	815.07	1.35	1.35	1100.35	1100.35	
SIDL except wc	=	147.10	1.35	1.35	198.59	198.59	
WC	=	123.34	1.75	1.75	215.84	215.84	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	671.00	1.5	0.20	1006.49	134.20	(1-70RW)
CWLLmax-Reaction case	=	1801.96	1	0.20	1801.96	360.39	SV Loading
CWLLmin	=	329.00	1.5	0.20	493.51	65.80	(1-70RW)
CWLLmin	=	1618.04	1	0.20	1618.04	323.61	SV Loading
CWLLmax-Transv. Moment Case		671.00	1.5	0.20	1006.49	134.20	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 554 = 227.7 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1514.77	0	75.739	75.739	-75.739	75.739	
DL+SIDL+LL-Max Reaction case	2008.28	341.55	100.414	271.189	241.136	271.189	(1-70RW)
	3626.32	0	181.316	181.316	-181.316	181.316	SV Loading
DL+SIDL+LL-Min Reaction case	2521.27	341.55	126.063	296.838	215.487	296.838	(1-70RW)
	3316.74	0	165.837	165.837	-165.837	165.837	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2521.27	341.55	126.063	296.838	215.487	296.838	

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	1085.51	1514.77	586.18	75.739	368.826	510.437	586.176	
DL+SIDL+LL-Max Reaction case		1580.57	631.72	79.029	394.887	552.687	631.716	Dry Case
DL+SIDL+LL-Min Reaction case		1648.97	631.72	82.449	398.306	549.267	631.716	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1648.97	631.72	82.449	398.306	549.267	631.716	

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	1085.51	1514.77	175.853	75.739	163.665	100.114	175.853	
DL+SIDL+LL-Max Reaction case		1580.57	221.393	79.029	189.725	142.364	221.393	Dry Case
DL+SIDL+LL-Min Reaction case		1648.97	221.393	82.449	193.145	138.944	221.393	HFL Case

DL+SIDL+LL-Max Transv. Moment case		1648.97	221.393	82.449	193.145	138.944	221.393

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	=	815.07	1.35	1.35	1100.35	1100.35
SIDL except wc	=	147.10	1.35	1.35	198.59	198.59
WC	=	123.34	1.75	1.75	215.84	215.84
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	671.00	1.5	0.75	1006.49	503.25
CWLLmax-Transv. Moment Case	=	671.00	1.5	0.75	1006.49	503.25
CWLLmin	=	329.00	1.5	0.75	493.51	246.75

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1514.77	0.000	75.739	75.739	-75.739	75.739	
DL+SIDL+LL-Max Reaction case	2008.28	341.550	100.414	271.189	241.136	271.189	(1-70RW)
	3626.32	0.000	181.316	181.316	-181.316	181.316	SV Loading
DL+SIDL+LL-Min Reaction case	2521.27	341.550	126.063	296.838	215.487	296.838	(1-70RW)
	3316.74	0.000	165.837	165.837	-165.837	165.837	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2521.27	341.550	126.063	296.838	215.487	296.838	

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	1085.51	1514.77	791.34	75.739	471.407	715.598	791.337	
DL+SIDL+LL-Max Reaction case		1761.53	836.88	88.076	506.515	748.801	836.877	Dry Case
DL+SIDL+LL-Min Reaction case		2018.02	836.88	100.901	519.340	735.976	836.877	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2018.02	836.88	100.901	519.340	735.976	836.877	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1085.51	1514.77	237.401	75.739	194.439	161.662	237.401	
DL+SIDL+LL-Max Reaction case		1761.53	282.941	88.076	229.547	194.865	282.941	Dry Case
DL+SIDL+LL-Min Reaction case		2018.02	282.941	100.901	242.372	182.040	282.941	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2018.02	282.941	100.901	242.372	182.040	282.941	

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	=	815.07	1	1.00	815.07	815.07
SIDL except wc	=	147.10	1	1.00	147.10	147.10
WC	=	123.34	1	1.00	123.34	123.34
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	671.00	1	0.20	671.00	134.20
CWLLmax- Transv. Moment Case		671.00	1	0.20	671.00	134.20
CWLLmin	=	329.00	1	0.20	329.00	65.80

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1085.51	0.000	54.276	54.276	-54.276	54.276	
DL+SIDL+LL-Max Reaction case	1414.51	227.700	70.726	184.576	156.974	184.576	(1-70RW)
	2703.55	0.000	135.177	135.177	-135.177	135.177	SV Loading
DL+SIDL+LL-Min Reaction case	1756.51	227.700	87.825	201.675	139.875	201.675	(1-70RW)
	2887.47	0.000	144.374	144.374	-144.374	144.374	SV Loading
DL+SIDL+LL-Max Transv. Moment case	1756.51	227.700	87.825	201.675	139.875	201.675	

Dry Case

HFL Case

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1085.51	527.558	54.276	318.055	473.282	527.558	
DL+SIDL+LL-Max Reaction case	1151.31	573.098	57.566	344.115	515.532	573.098	Dry Case
DL+SIDL+LL-Min Reaction case	1219.71	573.098	60.985	347.534	512.113	573.098	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1219.71	573.098	60.985	347.534	512.113	573.098	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1085.51	158.267	54.276	133.409	103.992	158.267	
DL+SIDL+LL-Max Reaction case	1151.31	203.807	57.566	159.469	146.242	203.807	Dry Case
DL+SIDL+LL-Min Reaction case	1219.71	203.807	60.985	162.889	142.822	203.807	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1219.71	203.807	60.985	162.889	142.822	203.807	

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	=	815.07	1.1	1.10	896.58	896.58
SIDL except wc	=	147.10	1.1	1.10	161.81	161.81
WC	=	123.34	1.35	1.35	166.51	166.51
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	671.00	1.5	0.00	1006.49	0.00
CWLLmax-Transv. Moment Case		671.00	1.5	0.00	1006.49	0.00
CWLLmin	=	329.00	1.5	0.00	493.51	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1224.90	0.000	61.245	61.245	-61.245	61.245	
DL+SIDL+LL-Max Reaction case	1718.40	341.550	85.920	256.695	255.630	256.695	(1-70RW)
	2842.93	0.000	142.147	142.147	-142.147	142.147	SV Loading
DL+SIDL+LL-Min Reaction case	2231.39	341.550	111.570	282.345	229.980	282.345	(1-70RW)
	3026.86	0.000	151.343	151.343	-151.343	151.343	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2231.39	341.550	111.570	282.345	229.980	282.345	

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	1085.51	1224.90	791.34	61.245	456.913	730.092	791.337	
DL+SIDL+LL-Max Reaction case		1224.90	791.34	61.245	456.913	730.092	791.337	Dry Case
DL+SIDL+LL-Min Reaction case		1224.90	791.34	61.245	456.913	730.092	791.337	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1224.90	791.34	61.245	456.913	730.092	791.337	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1085.51	1224.90	237.401	61.245	179.945	176.156	237.401	
DL+SIDL+LL-Max Reaction case		1224.90	237.401	61.245	179.945	176.156	237.401	Dry Case
DL+SIDL+LL-Min Reaction case		1224.90	237.401	61.245	179.945	176.156	237.401	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1224.90	237.401	61.245	179.945	176.156	237.401	

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	=	815.07	1	1	1	815.07	815.07	815.07
SIDL except wc	=	147.10	1	1	1	147.10	147.10	147.10
WC	=	123.34	1.20	1.20	1.20	148.01	148.01	148.01
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	671.00	1	0.75	0	671.00	503.25	0.00
CWLLmax-Transv. Moment Case	=	671.00	1	0.75	0	671.00	503.25	0.00
CWLLmin	=	329.00	1	0.75	0	329.00	246.75	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)		
DL+SIDL	1110.18	0.000	55.509	55.509	-55.509	55.509		
DL+SIDL+LL-Max Reaction case	1439.18	227.700	71.959	185.809	155.741	185.809	(1-70RW)	Dry Case
	2728.21	0.000	136.411	136.411	-136.411	136.411	SV Loading	
DL+SIDL+LL-Min Reaction case	1781.17	227.700	89.059	202.909	138.641	202.909	(1-70RW)	HFL Case
	2912.14	0.000	145.607	145.607	-145.607	145.607	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1781.17	227.700	89.059	202.909	138.641	202.909		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1110.18	0.000	55.509	55.509	-55.509	55.509	
DL+SIDL+LL-Max Reaction case	1356.93	170.775	67.847	153.234	102.928	153.234	Dry Case
DL+SIDL+LL-Min Reaction case	1613.43	170.775	80.671	166.059	90.104	166.059	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1613.43	170.775	80.671	166.059	90.104	166.059	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1110.18	0.000	55.509	55.509	-55.509	55.509	
DL+SIDL+LL-Max Reaction case	1110.18	0.000	55.509	55.509	-55.509	55.509	Dry Case
DL+SIDL+LL-Min Reaction case	1110.18	0.000	55.509	55.509	-55.509	55.509	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1110.18	0.000	55.509	55.509	-55.509	55.509	

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	=	40.00	kmph	40.00	kmph
Live Load	W	=	671.00	kN	671.00	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

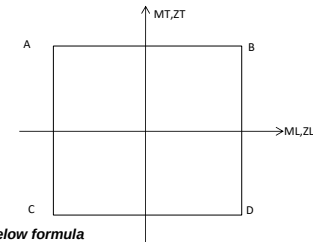
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum 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 = 1753.930 m

Properties of Footing Base:

A	=	65.450	m ²
ZL	=	83.994	m ³
ZT	=	92.721	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 = PxL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxT (kNm)
Superstructure								
Dead Load	1			815.073	0.850	692.812	0.000	0.000
SIDL except Wearing Course	1			147.100	0.850	125.035	0.000	0.000
Wearing Course	1			123.338	0.850	104.837	0.000	0.000
Bearing Pedestal	1	25	0.902	22.560	0.850	19.176	-0.278	-6.272
				1108.070		941.860		-6.272
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	4.807	120.169	0.050	6.008	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.271	6.773	0.050	0.339	0.000	0.000
Bracket - Uniform portion	1	25	0.765	19.125	-0.250	-4.781	0.000	0.000
Bracket - Tapered portion	1	25	0.383	9.563	-0.200	-1.913	0.000	0.000
Cap - (uniform portion)	1	25	4.718	117.938	0.825	97.298	0.000	0.000
Cap - (corbel portion)	1	25	3.579	89.466	0.825	73.809	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.050	1.400	0.000	0.000
Approach Slab	1	25	4.463	111.563	-0.250	-27.891	0.000	0.000
				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	33.629	840.735	-1.750	-1471.286	0.000	0.000
Abutment Shaft	1	25	42.666	1066.644	0.850	906.647	0.000	0.000
Back filling over heel slab	1	20	284.199	5683.976	-1.781	-10121.887	0.000	0.000
Front Filling over toe slab	1	20	72.866	1457.325	2.557	3727.066	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	23.205	580.125	-1.373	-796.556	0.000	0.000
Toe slab	1	25	13.813	345.313	2.376	820.339	0.000	0.000
portion between heel & toe	1	25	8.500	212.500	0.850	180.625	0.000	0.000
Vertical Components of active earth pressure	1			633.271	-3.850	-2438.092	0.000	0.000
				10819.887		-9193.145		0.000
Total				12430.553		-8107.015		-6.272

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	54.276	1761.102	389.264
due to Earth pressure	1	1528.850		5231.463

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		61.245	1761.102	439.248
due to Earth pressure	1.5	2293.276		7847.195

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxL2 (kNm)
0.900	733.566	-3.000	-2200.697
0.900	132.390	-3.000	-397.170
1.000	123.338	-3.000	-370.013
0.900	20.304	-3.000	-60.912
	1009.597		-3028.791
0.900	108.152	-3.800	-410.977
0.900	6.096	-3.800	-23.165
0.900	17.213	-4.100	-70.571
0.900	8.606	-4.050	-34.855
0.900	106.144	-3.025	-321.085
0.900	80.519	-3.025	-243.570
0.900	0.000	-3.500	0.000
0.900	0.000	-3.500	0.000
0.900	25.200	-3.800	-95.760
0.900	100.406	-4.100	-411.666
	452.336		-1611.649
0.900	756.662	-5.600	-4237.304
0.900	959.979	-3.000	-2879.938
0.900	5115.578	-5.631	-28804.674
0.900	1311.592	-1.293	-1695.272
0.900	0.000	0.000	0.000
0.900	522.113	-5.223	-2727.034
0.900	310.781	-1.474	-458.203
0.900	191.250	-3.000	-573.750
0.900	569.944	-7.700	-4388.565
	9737.899		-45764.740
	11199.831		-50405.181

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
54.28	389.26	0.00	0.00
1528.85	5231.46	0.00	0.00
1583.126	5620.727	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
61.24	439.25	0.00	0.00
2293.28	7847.19	0.00	0.00

2354.520

8286.442

2354.520

8286.442

0.000

0.000

Summary of Forces For SBC		
P	12430.553	kN
ML	-2486.288	kNm
MT	-6.272	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 = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Forces from Case :DL+SIDL		12430.553		-8107.015		-6.272
CWLL-Max. Reaction case	1	670.996	0.850	570.347	1.155	775.001
Vertical Components of LL Surcharge	1	165.225	-3.850	-636.115	0.000	0.000
Total		13266.774		-8172.782		768.729

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	184.576	1761.102	1323.777
due to Earth pressure	1	1528.850		5231.463
due to Live load surcharge	1	398.887		1726.584

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		256.695	1761.102	1841.017
due to Earth pressure	1.5	2293.276		7847.195
due to Live load surcharge	1.2	478.665		2071.901
		3028.636		11760.112

Summary of Forces For SBC		
P	13266.774	kN
ML	109.042	kNm
MT	768.729	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 = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Superstructure				1108.070		941.860		-6.272
Substructure & Foundation -Portion 1				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	33.629	840.735	-1.750	-1471.286	0.000	0.000
Shaft above HFL	1	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1	15	42.666	639.986	0.850	543.988	0.000	0.000
Back filling above HFL over heel slab	1	20	92.035	1840.692	-1.750	-3221.211	0.000	0.000
Back filling below HFL over heel slab	1	10	193.815	1938.153	-1.795	-3479.233	0.000	0.000
Front Filling over toe slab	1	10	72.866	728.662	1.642	1196.446	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	23.205	348.075	-1.373	-477.934	0.000	0.000
Toe slab	1	15	13.813	207.188	2.376	492.203	0.000	0.000
Portion between Heel & Toe	1	15	8.500	127.500	0.850	108.375	0.000	0.000
Vertical Components of active earth pressure	1			633.271	-3.850	-2438.092	0.000	0.000
				7304.262		-8746.743		0.000
Total				8914.927		-7660.613		-6.272

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = Px eL2 (kNm)
	11199.83142		-50405.18
0.000		-3.000	0.00
0.900		-7.700	-1145.01
	11199.831		-51550.19

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
184.58	1323.78	0.00	0.00
1528.85	5231.46	0.00	0.00
398.887	1726.584	0.000	0.000
2112.31	8281.82	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
256.70	1841.02	0.00	0.00
2293.28	7847.19	0.00	0.00
478.66	2071.90	0.00	0.00
3028.636	11760.112	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = Px eL2 (kNm)
	1009.597		-3028.791
	452.336		-1611.649
0.900	756.662	-5.600	-4237.304
0.900	0.000	-3.000	0.000
0.900	575.988	-3.000	-1727.963
0.900	1656.623	-5.600	-9277.088
0.900	1744.338	-5.645	-9847.010
0.900	655.796	-2.208	-1448.014
0.900	0.000	0.000	0.000
0.900	313.268	-5.223	-1636.220
0.900	186.469	-1.474	-274.922
0.900	114.750	-3.000	-344.250
0.900	569.944	-7.700	-4388.565
	6573.836		-33181.336
	8035.768		-37821.777

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	54.276	1761.102	389.264
due to Earth pressure	1	1528.850		4411.753

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		61.245	1761.102	439.248
due to Earth pressure	1.5	2293.276		6617.629
		2354.520		7056.877

Summary of Forces For SBC

P	8914.927	KN
ML	-2859.596	kNm
MT	-6.272	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 = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Forces from Case : DL+SIDL		8914.927		-7660.613		-6.272
CWLL-Min. Reaction case	1	329.004	0.850	279.653	1.155	379.999
Vertical Components of LL Surcharge	1	180.303	-3.850	-694.168	0.000	0.000
Total		9424.235		-8075.128		373.727

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	201.675	1761.102	1446.415
due to Earth pressure	1	1528.850		4411.753
due to Live load surcharge	1	435.291		1884.157

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		282.345	1761.102	2024.975
due to Earth pressure	1.5	2293.276		6617.629
due to live load surcharge	1.2	522.349		2260.989
		3097.969		10903.593

Summary of Forces For SBC

P	9424.235	KN
ML	-332.803	kNm
MT	373.727	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
54.28	389.26	0.00	0.00
1528.85	4411.75	0.00	0.00
1583.13	4801.02	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
61.24	439.25	0.00	0.00
2293.28	6617.63	0.00	0.00
2354.52	7056.88	0.00	0.00

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = Px eL2 (kNm)
	8035.768		-37821.777
0.000	0.000	-3.000	0
0.900	162.273	-7.700	-1249.503
	8198.042		-39071.280

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
201.68	1446.42	0.00	0.00
1528.85	4411.75	0.00	0.00
435.291	1884.157	0.000	0.000
2165.82	7742.33	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
282.34	2024.97	0.00	0.00
2293.28	6617.63	0.00	0.00
522.349	2260.989	0.000	0.000
3097.97	10903.59	0.00	0.00

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

Seismic Effect Factor =	1	ah=	0.243	In Longitudinal direction	Weight of shaft below Ground level	=	654.287 KN
		ah=	0.282	In Transverse direction	Weight of back fill below Ground level	=	2198.406 KN
		av=	0.188	In Vertical direction			

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															
Dead Load	1			815.073		69.062	46.042	0.850	692.812	39.135	1762.049		0.000	0.000	560.694
SIDL except Wearing Course	1			147.100		12.464	8.309	0.850	125.035	7.063	1763.036		0.000	0.000	113.496
Wearing Course	1			123.338		10.451	6.967	0.850	104.837	5.922	1762.587		0.000	0.000	90.471
Bearing Pedestal	1	25	0.902	22.560		1.912	1.274	0.850	19.176	1.083			-0.278	-6.272	
				1108.070		93.888	62.592		941.860	53.203					764.661
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	4.807	120.169	29.201	10.182	6.788	0.050	6.008	0.339	1761.645	225.271	0.000	0.000	78.550
Dirt Wall-Tapered portion	1	25	0.271	6.773	1.646	0.574	0.383	0.050	0.339	0.019	1760.649	11.059	0.000	0.000	3.856
Bracket - Uniform portion	1	25	0.765	19.125				-0.250	-4.781						
Bracket - Tapered portion	1	25	0.383	9.563				-0.200	-1.913						
Cap - (uniform portion)	1	25	4.718	117.938	28.659	9.993	6.662	0.825	97.298	5.496	1760.400	185.408	0.000	0.000	64.650
Cap - (corbel portion)	1	25	3.579	89.466	21.740	7.581	5.054	0.825	73.809	4.169	1760.100	134.126	0.000	0.000	46.768
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1762.587	0.000	0.000	0.000	0.000
Cantilever Return Wall-Traingle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1762.587	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.050	1.400				0.000	0.000	
Approach Slab	1	25	4.463	111.563				-0.250	-27.891				0.000	0.000	
				502.595	81.246	28.330	18.886		144.270	10.024		555.864		0.000	193.824
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	33.629	840.735	204.299	71.237	47.491	-1.750	-1471.286	-83.109	1758.607	955.428	0.000	0.000	333.147
Abutment Shaft	1	25	42.666	1066.644	100.203	34.940	23.293	0.850	906.647	19.799	1758.979	505.948	0.000	0.000	176.419
Back filling over heel slab	1	20	284.199	5683.976	0.000	0.000	0.000	-1.781	-10121.887	0.000	1758.607	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	72.866	1457.325				2.557	3727.066				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	23.205	580.125				-1.373	-796.556				0.000	0.000	
Toe slab	1	25	13.813	345.313				2.376	820.339				0.000	0.000	
portion between heel & toe	1	25	8.500	212.500				0.850	180.625				0.000	0.000	
Vertical component of active earth pressure	1			595.979				-3.850	-2294.518				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			311.844				-3.850	-1200.600				0.000	0.000	
				11094.440	304.501	106.176	70.784		-10250.172	-63.310		1461.376		0.000	509.566
Total =				12705.105	385.747	228.394	152.263		-9164.042	-0.083		2017.240		-6.272	1468.050
							-152.263			0.083					

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
Superstructure								
Dead Load	0.9			733.566	41.437	-3.000	-2200.697	-124.312
SIDL except Wearing Course	0.9			132.390	7.478	-3.000	-397.170	-22.435
Wearing Course	1.00			123.338	6.967	-3.000	-370.013	-20.901
Bearing Pedestal	0.9	25	0.9024	20.304		-3.000	-60.912	
				1009.597	55.883		-3028.791	-167.648
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	4.807	108.152	6.109	-3.800	-410.977	-23.215
Dirt Wall-Tapered portion	0.9	25	0.271	6.096	0.344	-3.800	-23.165	-1.309
Bracket - Uniform portion	0.9	25	0.765	17.213				
Bracket - Tapered portion	0.9	25	0.383	8.606				
Cap - (uniform portion)	0.9	25	4.718	106.144	5.996	-3.025	-321.085	-18.137
Cap - (corbel portion)	0.9	25	3.579	80.519	4.548	-3.025	-243.570	-13.759
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-3.500	0.000	0.000
Cantilever Return Wall-Traingle portion	0.9	25	0.000	0.000	0.000	-3.500	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-3.800	-95.760	
Approach Slab	0.9	25	4.463	100.406		-4.100	-411.666	
				452.336	16.998		-1506.223	-56.420

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	29.201	1761.645	225.271
1.0	1.646	1760.649	11.059
1.0	28.659	1760.400	185.408
1.0	21.740	1760.100	134.126
1.0	0.000	1762.587	0.000
1.0	0.000	1762.587	0.000
	81.246		555.864

Substructure & Foundation -Portion 2												
Abutment Shaft	0.9	25	42.666	959.979	17.268	-3.000	-2879.938	-51.803	1.0	82.537	1758.979	416.749
Solid Return wall	0.9	25	33.629	756.662	42.742	-5.600	-4237.304	-239.355	1.0	204.299	1758.607	955.428
Back filling over heel slab	0.9	20	284.199	5115.578	0.000	-5.631	-28804.7	0.000	1.0	0.000	1758.607	0.000
Front Filling over toe	0.9	20	72.866	1311.592		-1.293	-1695.272					
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000					
Heel slab	0.9	25	23.205	522.113		-5.223	-2727.034					
Toe slab	0.9	25	13.813	310.781		-1.474	-458.203					
portion between heel & toe	0.9	25	8.500	191.250		-3.000	-573.750					
Vertical component of active earth pressure	0.9			569.944		-7.700	-4388.565					
Vertical component of dynamic increment of earth pressure	0.9			280.660		-7.700	-2161.081					
				10018.558	60.010		-47925.82	-291.159		286.835		1372.177
Total =				11480.491	132.890		-52460.83	-515.227		368.081		1928.041
					-132.890	515.227						

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	527.558	93.888	1761.102	3783.646	764.661
due to Substructure	1	385.747	134.506		2017.240	703.390
due to Earth pressure	1	2191.679			8866.641	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.337	1761.102	5675.47
due to Substructure	1.5	552.122		2892.06
due to Active Earth pressure	1	1438.820		5231.46
due to dynamic Earth pressure	1.5	1129.288		5452.77
		3911.567		19251.761

Summary of Forces For SBC

	Downward	Upward	
P	12857.368	12552.843	KN
ML	5503.403	5503.568	kNm
MT	1461.778	1461.778	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
527.56	3783.65	0.00	0.00	0.00	0.00	93.89	764.66
385.75	2017.24	0.00	0.00	0.00	0.00	134.51	703.39
2191.68	8866.64						
3104.98	14667.53	0.00	0.00	0.00	0.00	228.39	1468.05

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.34	5675.47	0.00	0.00
552.12	2892.06	0.00	0.00
1438.820	5231.463	0.000	0.000
1129.288	5452.767	0.000	0.000
3911.57	19251.76	0.00	0.00

Summary of Restoring Forces

Vertical Load	11347.601	kN
Moment	-52976.062	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 = 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 = 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				1108.070		93.888	62.592		941.860	53.203				-6.272	764.661
Forces from Substructure				11597.035	385.747	134.506	89.671		-10105.901	-53.286		2017.240		0.000	703.390
CWLL-Max. Reaction case	0.20			134.20		11.371	7.581	0.850	114.069	6.444	1763.787		1.155	155.000	112.083
Vertical component of LL Surcharge	0.20			33.04				-3.850	-127.223				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			19.55				-3.850	-75.263				0.000	0.000	
Total =				12891.898	385.747	239.765	159.843		-9252.458	6.361		2017.240		148.728	1580.133

-159.843

-6.361

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				1009.597	55.883	0.000	-3028.79	-167.648
Forces from Substructure				10470.894	77.008	0.000	-49432.04	-347.578
CWLL-Max. Reaction case	0.00			0.00	0.000	-3.000	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-7.700	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.700	0.00	
Total =				11480.491	132.890		-52460.83	-515.227
					-132.890			515.227

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	573.098	105.259	1761.102	4110.259	876.744
due to Substructure	1	385.747	134.506		2017.240	703.390
due to Earth pressure	1	2191.679			8866.641	
due to Live load surcharge	0.20	126.972			634.631	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.337	1761.102	5675.469
due to Substructure	1.5	552.122		2892.062
due to Active Earth pressure	1	1438.820		5231.463
due to dynamic Earth pressure	1.5	1129.288		5452.76716
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		3911.567		19251.761

Summary of Forces For SBC

	Downward	Upward	
P	13051.742	12732.055	KN
ML	6382.674	6369.953	kNm
MT	1728.861	1728.861	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
573.10	4110.26	0.00	0.00	0.00	0.00	105.26	876.74
385.75	2017.24	0.00	0.00	0.00	0.00	134.51	703.39
2191.68	8866.64						
126.97	634.63						
3277.50	15628.77	0.00	0.00	0.00	0.00	239.77	1580.13

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.34	5675.47	0.00	0.00
552.12	2892.06	0.00	0.00
1438.820	5231.463		
1129.288	5452.767		
0.000	0.000		
0.000	0.000		
3911.57	19251.76	0.00	0.00

Summary of Restoring Forces

Vertical Load	11347.601	kN
Moment	-52976.062	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 = αh x P (kN)	FT = 0.3 x αh 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				1108.070		93.888	62.592		941.860	53.203				-6.272	764.661
Substructure & Foundation -Portion 1				502.595	81.246	28.330	18.886		144.270	10.024		555.864		0.000	193.824
Substructure & Foundation -Portion 2															
Solid Return wall	1		33.63	840.74	204.299	71.237	47.491	-1.75	-1471.286	-83.109	1758.61	955.428	0.00	0.000	333.147
Shaft above HFL	1	25	0.000	0.000	0.000	0.000	0.000	0.850	0.000	0.000	1759.979	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	42.666	639.986	100.203	34.940	23.293	0.850	543.988	19.799	1759.009	508.929	0.000	0.000	177.458
Back filling above HFL over heel slab	1	20	92.035	1840.692	0.000	0.000	0.000	-1.750	-3221.211	0.000	1761.298	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	193.815	1938.153	0.000	0.000	0.000	-1.795	-3479.233	0.000	1757.220	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	72.866	728.662				1.642	1196.446				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	23.205	348.075				-1.373	-477.934				0.000	0.000	
Toe slab	1	15	13.813	207.188				2.376	492.203				0.000	0.000	
portion between heel & toe	1	15	8.500	127.500				0.850	108.375				0.000	0.000	
Vertical component of active earth pressure	1			490.023				-3.850	-1886.589				0.000	0.000	

Vertical component of dynamic increment of earth pressure	1			313.184				-3.850	-1205.758				0.000	0.000	
				7474.198	304.501	106.176	70.784		-9400.998	-63.310		1464.357		0.000	510.605
Total =				9084.864	385.747	228.394	152.263		-8314.868	-0.083		2020.221		-6.272	1469.090
								-152.263		0.083					

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
Superstructure				1009.597	55.883		-3028.791	-167.648
Substructure & Foundation -Portion 1				452.336	16.998		-1506.223	-56.420
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	33.6294	756.662	42.742	-5.60	-4237.304	-239.355
Shaft above HFL	0.9	25	0.000	0.000	0.000	-3.000	0.000	0.000
Shaft below HFL	0.9	15	42.666	575.988	17.268	-3.000	-1727.963	-51.803
Back filling above HFL over heel slab	0.9	20	92.035	1656.623	0.000	-5.600	-9277.088	0.000
Back filling below HFL over heel slab	0.9	10	193.815	1744.338	0.000	-5.645	-9847.010	0.000
Front Filling over toe	0.9	10	72.866	655.796		-2.208	-1448.014	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	23.205	313.268		-5.223	-1636.220	
Toe slab	0.9	15	13.813	186.469		-1.474	-274.922	
portion between heel & toe	0.9	15	8.500	114.750		-3.000	-344.250	
Vertical component of active earth pressure	0.9			441.021		-7.700	-3395.859	
Vertical component of dynamic increment of earth pressure	0.9			281.865		-7.700	-2170.364	
				6726.778	60.010		-34358.99	-291.159
Total =				8188.711	132.890		-38894.01	-515.227
					-132.890		515.227	

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
	81.246		555.864
1.0	204.299	1758.607	955.428
1.0	0.000	1759.979	0.000
1.0	82.537	1759.009	419.204
1.0	0.000	1761.298	0.000
1.0	0.000	1757.220	0.000
	286.835		1374.632
	368.081		1930.496

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	527.558	93.888	1761.102	3783.646	764.661
due to Substructure	1	385.747	134.506		2020.221	704.429
due to Earth pressure	1	1939.113			7425.846	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.337	1761.102	5675.469
due to Substructure	1.5	552.122		2895.745
due to Active Earth pressure	1	1183.020		3777.569
due to dynamic Earth pressure	1.5	1134.139		5472.415
		3660.618		17821.198

Summary of Forces For SBC

	Downward	Upward
P	9237.127	8932.601
ML	4914.763	4914.929
MT	1462.818	1462.818

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
527.56	3783.65	0.00	0.00	0.00	0.00	93.89	764.66
385.75	2020.22	0.00	0.00	0.00	0.00	134.51	704.43
1939.11	7425.85						
2852.42	13229.71	0.00	0.00	0.00	0.00	228.39	1469.09

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.34	5675.47	0.00	0.00
552.12	2895.74	0.00	0.00
1183.020	3777.569		
1134.139	5472.415		
3660.62	17821.20	0.00	0.00

Summary of Restoring Forces

Vertical Load	8055.821	kN
Moment	-39409.235	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				1108.070		93.888	62.592		941.860	53.203				-6.272	764.661
Forces from Substructure				7976.793	385.747	134.506	89.671		-9256.727	-53.286				0.000	704.429
CWLL-Max. Reaction case	0.20			65.80		5.575	3.717	0.850	55.931	3.159	1763.787		1.155	76.000	54.957
Vertical component of LL Surcharoe	0.20			36.06				-3.850	-138.834				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			32.67				-3.850	-125.768				0.000	0.000	
Total =				9219.392	385.747	233.970	155.980		-8523.539	3.076		2020.221		69.728	1524.046
						-155.980				-3.076					

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				1009.597	55.883		-3028.791	-167.648
Forces from Substructure				7179.114	77.008		-35865.22	-347.578
CWLL-Max. Reaction case	0.00			0.00	0.00		-3.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-3.000	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.700	0.000	0.000
Total =				8188.711	132.890		-38897.01	-515.227
						-132.890		515.227

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	573.098	99.464	1761.102	4110.259	819.617
due to Substructure	1	385.747	134.506		2020.221	704.429
due to Earth pressure	1	1939.113			7425.846	
due to Live load surcharge	0.20	165.924			834.053	

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
573.10	4110.26	0.00	0.00	0.00	0.00	99.46	819.62
385.75	2020.22	0.00	0.00	0.00	0.00	134.51	704.43
1939.11	7425.85						
165.92	834.05						
3063.88	14390.38	0.00	0.00	0.00	0.00	233.97	1524.05

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.337	1761.102	5675.469
due to Substructure	1.5	552.122		2895.74473
due to Active Earth pressure	1	1183.020		3777.569
due to dynamic Earth pressure	1.5	1134.139		5472.41535
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		3660.618	17821.198	

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
791.34	5675.47	0.00	0.00
552.12	2895.74	0.00	0.00
1183.020	3777.569		
1134.139	5472.415		
0.000	0.000		
0.000	0.000		
3660.62	17821.20	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	9375.372	9063.412	kN
ML	5869.917	5863.764	kNm
MT	1593.774	1593.774	kNm

Summary of Restoring Forces

Vertical Load	8055.821	kN
Moment	-39412.235	kNm

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

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	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 = PxL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxL1T	MTs due to FT
Superstructure				1108.070		312.961	62.592		941.860	53.203				-6.272	2548.869
Substructure & Foundation -Portion 1				502.595	24.374	94.432	18.886		144.270	10.024		166.759		0.000	646.079
Substructure & Foundation -Portion 2				11094.440	91.350	353.921	70.784		-10250.172	-63.310		438.413		0.000	1698.553
Total =				12705.105	115.724	761.314	152.263		-9164.042	-0.083		605.172		-6.272	4893.501
						-152.263				0.083					

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	203.807	350.864	1761.102	1461.707	2922.479
due to Substructure	1	115.724	448.353		605.172	2344.632
due to Earth pressure	1	2191.679			8866.641	
due to Live load surcharge	0.2	126.972			634.631	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		237.40	1761.102	1702.641
due to Substructure	1.5	165.64		867.618
due to Active Earth pressure	1	1438.82		5231.463
due to dynamic Earth pressure	1.5	338.79		1635.830
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		2180.64		9437.552

Summary of Forces For SBC

	Downward	Upward
P	13051.742	12732.055
ML	2322.053	2309.332
MT	5415.839	5415.839

Case 11 : DL+SIDL-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
Superstructure	1108.070	62.592	941.860	53.203	-6.272
Substructure & Foundation -Portion 1	502.595	18.886	144.270	10.024	0.000
Substructure & Foundation -Portion 2	7474.198	70.784	-9400.998	-63.310	0.000
Total =	9084.864	152.263	-8314.868	-0.083	-6.272
		-152.263		0.083	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	158.27	312.96	1761.10	1135.09	2548.87
due to Substructure	1	115.72	448.35		606.07	2348.10
due to Earth pressure	1	1939.11			7425.85	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		237.401	1761.102	1702.64
due to Substructure	1.5	165.637		868.72
due to Active Earth pressure	1	1183.020		3777.57
due to dynamic Earth pressure	1.5	340.242		1641.72
		1926.300		7990.66

Summary of Forces For SBC

	Downward	Upward
P	9237.127	8932.601
ML	852.056	852.222
MT	4890.694	4890.694

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
203.81	1461.71	0.00	0.00	0.00	0.00	350.86	2922.48
115.72	605.17	0.00	0.00	0.00	0.00	448.35	2344.63
2191.68	8866.64						
126.97	634.63						
2638.18	11568.15	0.00	0.00	0.00	0.00	799.22	5267.11

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
237.40	1702.64	0.00	0.00
165.64	867.62	0.00	0.00
1438.820	5231.463		
338.786	1635.830		
0.000	0.000		
0.000	0.000		
2180.64	9437.55	0.00	0.00

Summary of Restoring Forces

Vertical Load	11347.601	kN
Moment	-52976.062	kNm

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
Superstructure	1009.597	55.883		-3028.791	-167.648
Substructure & Foundation	452.336	16.998		-1506.223	-56.420
Substructure & Foundation	6726.778	60.010		-34358.994	-291.159
Total =	8188.711	132.890		-38894.008	-515.227
		-132.890			515.227

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
158.27	1135.09	0.00	0.00	0.00	0.00	312.96	2548.87
115.72	606.07	0.00	0.00	0.00	0.00	448.35	2348.10
1939.11	7425.85						
2213.10	9167.01	0.00	0.00	0.00	0.00	761.31	4896.97

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
237.40	1702.64	0.00	0.00
165.64	868.72	0.00	0.00
1183.020	3777.569		
340.242	1641.725		
1926.30	7990.66	0.00	0.00

Summary of Restoring Forces

Vertical Load	8055.821	kN
Moment	-39409.235	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 =	9219.392	155.980	-8523.54	3.076	69.728
		-155.980		-3.076	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	203.807	331.54605	1761.102	1461.707	2732.05748
due to Substructure	1	115.724	448.35276		606.066	2348.09711
due to Earth pressure	1	1939.113			7425.846	
due to Live load surcharge	0.2	165.924			834.053	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		237.40	1761.102	1702.64
due to Substructure	1.5	165.64		868.72
due to Active Earth pressure	1	1183.02		3777.57
due to dynamic Earth pressure	1.5	340.24		1641.72
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		1926.30		7990.66

Summary of Forces For SBC

	Downward	Upward	
P	9375.372	9063.412	kN
ML	1807.210	1801.057	kNm
MT	5149.882	5149.882	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 (	1763.787 -	=	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 (	1763.787 -	=	0.000 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	=	$P/A + ML/ZL - MT/ZT$

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 =	8188.711	132.890		-38897.01	-515.227
		-132.890			515.227

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
203.81	1461.71	0.00	0.00	0.00	0.00	331.55	2732.06
115.72	606.07	0.00	0.00	0.00	0.00	448.35	2348.10
1939.11	7425.85						
165.92	834.05						
2424.57	10327.67	0.00	0.00	0.00	0.00	779.90	5080.15

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
237.40	1702.64	0.00	0.00
165.64	868.72	0.00	0.00
1183.020	3777.569		
340.242	1641.725		
0.000	0.000		
0.000	0.000		
1926.30	7990.66	0.00	0.00

Summary of Restoring Forces

Vertical Load	8055.821	kN
Moment	-39412.235	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cos θ	MT Cos θ	FT Sin θ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

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	12430.553	-2486.288	-6.272	219.457	160.256	219.593	160.391	219.593	160.256	2354.520	7839.882	3.33	8286.442	50405.18	6.08
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	13266.774	109.042	768.729	209.693	212.290	193.112	195.708	212.290	193.112	3028.636	7839.882	2.59	11760.112	51550.19	4.38
								SAFE	SAFE			SAFE			SAFE
Normal HFL Case															
Case 3 : DL+SIDL-Normal HFL Case	8914.927	-2859.596	-6.272	170.187	102.097	170.323	102.232	170.323	102.097	2354.520	5625.038	2.39	7056.877	37821.78	5.36
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	9424.235	-332.803	373.727	151.984	144.060	143.923	135.998	151.984	135.998	3097.969	5738.629	1.85	10903.593	39071.28	3.58
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	12857.368	5503.568	1461.778	146.688	277.734	115.157	246.204	277.734	115.157	3911.567	7943.321	2.03	19251.761	52976.06	2.75
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	13051.742	6382.674	1728.861	142.072	294.051	104.780	256.759	294.051	104.780	3911.567	7943.321	2.03	19251.761	52976.06	2.75
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	9237.127	4914.929	1462.818	98.394	215.424	66.841	183.871	215.424	66.841	3660.618	5639.074	1.54	17821.198	39409.24	2.21
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	9375.372	5869.917	1593.774	90.549	230.319	56.171	195.941	230.319	56.171	3660.618	5639.074	1.54	17821.198	39412.24	2.21
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	12857.368	1442.948	4887.229	231.976	266.334	126.557	160.916	266.334	126.557	2567.129	7943.321	3.09	11461.995	52976.06	4.62
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	13051.742	2322.053	5415.839	230.180	285.471	113.360	168.651	285.471	113.360	2180.644	7943.321	3.64	9437.552	52976.06	5.61
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	9237.127	852.222	4890.694	183.733	204.025	78.240	98.532	204.025	78.240	1926.300	5639.074	2.93	7990.658	39409.24	4.93
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	9375.372	1807.210	5149.882	177.271	220.302	66.187	109.219	220.302	66.187	1926.300	5639.074	2.93	7990.658	39412.24	4.93
								SAFE	SAFE			SAFE			SAFE

DESIGN OF FOUNDATION

Foundation Lvl = 1753.930 m

Properties of Footing Base:

A	=	65.450	m ²
ZL	=	83.994	m ³
ZT	=	92.721	m ³

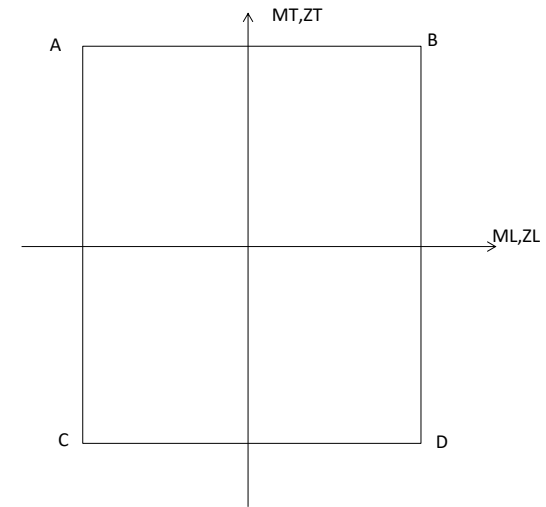
Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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	1.35			1100.348	0.850	935.296	0.000	0.000
SIDL except Wearing Course	1.35			198.585	0.850	168.797	0.000	0.000
Wearing Course	1.75			215.841	0.850	183.465	0.000	0.000
Bearing Pedestal	1.35	25	0.902	30.456	0.85	25.888	-0.278	-8.467
				1545.230		1313.445		-8.467
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	0.050	8.111	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	0.050	0.457	0.000	0.000
Bracket - Uniform portion	1.35	25	0.765	25.819	-0.250	-6.455	0.000	0.000
Bracket - Tapered portion	1.35	25	0.383	12.909	-0.200	-2.582	0.000	0.000
Cap - (uniform portion)	1.35	25	4.718	159.216	0.825	131.353	0.000	0.000
Cap - (corbel portion)	1.35	25	3.579	120.779	0.825	99.642	0.000	0.000
Cantilever Return Wall-Rectangle p	1.35	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle port	1.35	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier or	1.35			37.800	0.050	1.890	0.000	0.000
Crash Barrier	1.35	25	4.463	150.609	-0.250	-37.652	0.000	0.000
Approach Slab	1.35	25	4.463	150.609	-0.250	-37.652	0.000	0.000
				678.504		194.765		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	33.629	1134.992	-1.750	-1986.236	0.000	0.000
Abutment Shaft	1.35	25	42.666	1439.969	0.850	1223.974	0.000	0.000
Back filling over heel slab	1.35	20	284.199	7673.367	-1.781	-13664.548	0.000	0.000
Front Filling over toe slab	1.35	20	72.866	1967.389	2.557	5031.539	0.000	0.000
Side filling between heel and toe	1.35	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.35	25	23.205	783.169	-1.373	-1075.351	0.000	0.000
Toe slab	1.35	25	13.813	466.172	2.376	1107.457	0.000	0.000
portion between heel & toe	1.35	25	8.500	286.875	0.850	243.844	0.000	0.000
Vertical Components of active earth pressure	1.5			949.906	-3.850	-3657.138	0.000	0.000
				14701.838		-12776.459		0.000
Total				16925.572		-11268.249		-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		75.739	1761.102	543.198
due to Earth pressure	1.5	2293.276		7847.195

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
75.74	543.20	0.00	0.00
2293.28	7847.19	0.00	0.00
2369.014	8390.393	0.000	0.000

**Summary of Forces About C.G. OF Footing**

P	16925.572	kN
ML	-2877.856	kNm
MT	-8.467	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical load**

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		16925.572		-11268.249		-8.467
CWLL-Max. Reaction case	1.5	1006.495	0.850	855.520	1.155	1162.501
Vertical Components of LL Surcharge	1.2	198.269	-3.850	-763.337	0.000	0.000
Total		18130.336		-11176.066		1154.034

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		271.189	1761.102	1944.967
due to Earth pressure	1.5	2293.276		7847.195
due to Live load surcharge	1.2	478.665		2071.901
		3043.129		11864.063

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
271.19	1944.97	0.00	0.00
2293.28	7847.19		
478.66	2071.90		
3043.129	11864.063	0.000	0.000

Summary of Forces About C.G. OF Footing

P	18130.336	KN
ML	687.997	kNm
MT	1154.034	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1545.230		1313.445		-8.467
Substructure & Foundation -Portion 1				678.504		194.765		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	33.629	1134.992	-1.750	-1986.236	0.000	0.000
Shaft above HFL	1.35	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1.35	15	42.666	863.981	0.850	734.384	0.000	0.000
Back filling above HFL over heel slab	1.35	20	92.035	2484.934	-1.750	-4348.635	0.000	0.000
Back filling below HFL over heel slab	1.35	10	193.815	2616.507	-1.795	-4696.964	0.000	0.000
Front Filling over toe slab	1.35	10	72.866	983.694	1.642	1615.202	0.000	0.000
Side filling between heel and toe	1.35	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.35	15	23.205	469.901	-1.373	-645.211	0.000	0.000
Toe slab	1.35	15	13.813	279.703	2.376	664.474	0.000	0.000
Portion between Heel & Toe	1.35	15	8.500	172.125	0.850	146.306	0.000	0.000
Vertical Components of active earth pressure	1.5			949.906	-3.850	-3657.138	0.000	0.000
				9955.744		-12173.817		0.000
Total				12179.478		-10665.607		-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		75.739	1761.102	543.198
due to Earth pressure	1.5	2293.276		6617.629

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
75.74	543.20	0.00	0.00
2293.28	6617.63		
2369.014	7160.827	0.000	0.000

Summary of Forces About C.G. OF Footing

P	12179.478	kN
ML	-3504.780	kNm
MT	-8.467	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical load**

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		12179.478		-10665.607		-8.467
CWLL-Min. Reaction case	1.5	493.505	0.850	419.480	1.155	569.999
Vertical Components of LL Surcharge	1.2	216.364	-3.850	-833.002	0.000	0.000
Total		12889.347		-11079.129		561.532

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		296.838	1761.102	2128.925
due to Earth pressure	1.5	2293.276		6617.629
due to Live load surcharge	1.2	522.349		2260.989

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
296.84	2128.93	0.00	0.00
2293.28	6617.63		
522.35	2260.99		
3112.463	11007.543	0.000	0.000

Summary of Forces About C.G. OF Footing

P	12889.347	kN
ML	-71.586	kNm
MT	561.532	kNm

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

Seismic Effect Factor =	1.50	$\alpha_h =$	0.243 In Longitudinal direction	Weight of shaft below Ground level	=	654.287 KN
		$\alpha_h =$	0.282 In Transverse direction	Weight of back fill below Ground level	=	2198.406 KN
		$\alpha_v =$	0.188 In Vertical direction			

Forces due to Vertical Load

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	1.35			1100.348		103.593	69.062	0.850	935.296	58.703	1762.049		0.000	0.000	841.041
SIDL except Wearing Course	1.35			198.585		18.696	12.464	0.850	168.797	10.594	1763.036		0.000	0.000	170.244
Wearing Course	1.75			215.841		15.676	10.451	0.850	183.465	8.883	1762.587		0.000	0.000	135.706
Bearing Pedestal	1.35	25	0.9024	30.456		2.867	1.912	0.850	25.888	1.625			-0.278	-8.467	
				1545.230		140.833	93.888		1313.445	79.805				-8.467	1146.991

Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	43.802	15.273	10.182	0.050	8.111	0.509	1761.645	337.907	0.000	0.000	117.824
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	2.469	0.861	0.574	0.050	0.457	0.029	1760.649	16.588	0.000	0.000	5.784
Bracket - Uniform portion	1.35	25	0.765	25.819				-0.250	-6.455						
Bracket - Tapered portion	1.35	25	0.383	12.909				-0.200	-2.582						
Cap - (uniform portion)	1.35	25	4.718	159.216	42.988	14.990	9.993	0.825	131.353	8.244	1760.400	278.112	0.000	0.000	96.975
Cap - (corbel portion)	1.35	25	3.579	120.779	32.610	11.371	7.581	0.825	99.642	6.254	1760.100	201.189	0.000	0.000	70.152
Cantilever Return Wall-Rectangle p	1.35	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1762.587	0.000	0.000	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1762.587	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.050	1.890				0.000	0.000	0.000
Approach Slab	1.35	25	4.463	150.609				-0.250	-37.652				0.000	0.000	
				678.504	121.869	42.494	28.330		194.765	15.036		833.796		0.000	290.736
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	33.629	1134.992	306.448	106.855	71.237	-1.750	-1986.236	-124.664	1758.607	1433.142	0.000	0.000	499.721
Abutment Shaft	1.35	25	42.666	1439.969	212.134	73.969	49.313	0.850	1223.974	41.916	1758.979	1071.118	0.000	0.000	373.487
Back filling over heel slab	1.35	20	284.199	7673.367	0.000	0.000	0.000	-1.781	-13664.548	0.000	1758.607	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	72.866	1967.389				2.557	5031.539				0.000	0.000	
Side filling between heel and toe	1.35	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1.35	25	23.205	783.169				-1.373	-1075.351				0.000	0.000	
Toe slab	1.35	25	13.813	466.172				2.376	1107.457				0.000	0.000	
portion between heel & toe	1.35	25	8.500	286.875				0.850	243.844				0.000	0.000	
Vertical component of active earth pressure	1.00			595.979				-3.850	-2294.518						
Vertical component of dynamic increment of earth pressure	1.50			467.766				-3.850	-1800.901						
				14815.678	518.582	180.824	120.549		-13214.740	-82.749		2504.260		0.000	873.208
Total =				17039.411	640.451	364.151	242.767		-11706.530	12.093		3338.056		-8.467	2310.935
							-242.767			-12.093					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		791.337	140.833	1761.102	5675.469	1146.991
due to Substructure		640.451	223.318		3338.056	1163.944
due to Active Earth pressure	1.00	1438.820			5231.463	
due to dynamic increment of EP	1.50	1129.288			5452.767	

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
791.34	5675.47	0.00	0.00	0.00	0.00	140.83	1146.99
640.45	3338.06	0.00	0.00	0.00	0.00	223.32	1163.94
1438.82	5231.46						
1129.29	5452.77						
3999.90	19697.76	0.00	0.00	0.00	0.00	364.15	2310.93

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	17282.179	16796.644	KN
ML	8003.318	7979.132	kNm
MT	2302.468	2302.468	kNm

Heel slab	1.35	15	23.205	469.901				-1.373	-645.211				0.000	0.000	
Toe slab	1.35	15	13.813	279.703				2.376	664.474				0.000	0.000	
portion between heel & toe	1.35	15	8.500	172.125				0.850	146.306				0.000	0.000	
Vertical component of active earth pressure	1.00			490.023				-3.850	-1886.589						
Vertical component of dynamic increment of earth pressure	1.50			469.776				-3.850	-1808.636						
				9965.637	363.065	126.597	84.398		-12211.905	-113.477		1720.702		0.000	599.990
Total =				12189.370	484.934	309.924	206.616		-10703.694	-18.636		2554.498		-8.467	2037.717
								-206.616		18.636					

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		791.337	140.833	1761.102	5675.469	1146.991
due to Substructure		484.934	169.091		2554.498	890.726
due to Active Earth pressure	1.00	1183.020			3777.569	
due to dynamic increment of EP	1.50	1134.139			5472.415	

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
791.34	5675.47	0.00	0.00	0.00	0.00	140.83	1146.99
484.93	2554.50	0.00	0.00	0.00	0.00	169.09	890.73
1183.02	3777.57						
1134.14	5472.42						
3593.43	17479.95	0.00	0.00	0.00	0.00	309.92	2037.72

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	12395.99	11982.75	kN
ML	6757.62	6794.89	kNm
MT	2029.25	2029.25	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Forces due to Vertical Load**

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 = 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				1545.230		140.833	93.888		1313.445	79.805				-8.467	1146.991
Forces from Substructure				10644.141	484.934	169.091	112.728		-12017.140	-98.441		2554.498		0.000	890.726
CWLL-Min. Reaction case	0.75			246.75		31.362	20.908	0.850	209.740	17.772	1763.787		1.155	284.999	309.131
Vertical component of LL Surcharge	0.20			36.061				-3.850	-138.834						
Vertical component of dynamic increment LL Surcharge	1.50			245.003				-3.850	-943.263						
Total =				12717.187	484.934	341.285	227.524		-11576.051	-0.864		2554.498		276.532	2346.848
								-227.524		0.864					

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		836.877	172.194	1761.102	6002.082	1456.122
due to Substructure		484.934	169.091		2554.498	890.726
due to Active Earth pressure	1	1183.020			3777.569	
due to dynamic increment of EP	1.50	1134.139			5472.415	
due to Live load surcharge	0.20	87.058			376.831	

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
836.88	6002.08	0.00	0.00	0.00	0.00	172.19	1456.12
484.93	2554.50	0.00	0.00	0.00	0.00	169.09	890.73
1183.02	3777.57						
1134.14	5472.42						
87.06	376.83						

due to dynamic increment of Surchage	1.50	591.491			3429.163	

591.49	3429.16						
4317.52	21612.56	0.00	0.00	0.00	0.00	341.29	2346.85

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	12944.711	12489.664	kN
ML	10035.643	10037.372	kNm
MT	2623.380	2623.380	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Forces due to Vertical Load**

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 = 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				1545.230		469.442	93.888		1313.445	79.805				-8.467	3823.303
Substructure & Foundation -Portion 1				678.504	36.561	141.648	28.330		194.765	15.036		250.139		0.000	969.119
Substructure & Foundation -Portion 2				14815.678	155.575	602.746	120.549		-13214.740	-82.749		751.278		0.000	2910.694
Total =				17039.411	192.135	1213.836	242.767		-11706.530	12.093		1001.417		-8.467	7703.116
							-242.767			-12.093					

Forces due to Horizontal Load

	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	237.401	469.442	1761.102	1702.641	3823.303
due to Substructure	192.135	744.394		1001.417	3879.813
due to Active Earth pressure	1438.820			5231.463	
due to dynamic increment of EP	338.786			1635.830	

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
237.40	1702.64	0.00	0.00	0.00	0.00	469.44	3823.30
192.14	1001.42	0.00	0.00	0.00	0.00	744.39	3879.81
1438.82	5231.46						
338.79	1635.83						
2207.14	9571.35	0.00	0.00	0.00	0.00	1213.84	7703.12

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	17282.179	16796.644	kN
ML	-2123.087	-2147.272	kNm
MT	7694.649	7694.649	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				17722.320	285.408	-11970.465	48.337	572.783
					-285.408		-48.337	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		282.941	682.647	1761.102	2029.254	5924.85876
due to Substructure		192.135	744.394		1001.417	3879.81292
due to Earth pressure		1438.820			5231.463	
due to dynamic increment of EP		338.786			1635.830	
due to Live load surcharge		79.777			345.317	
due to dynamic increment of Surcharge		106.189			650.957	

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	18007.728	17436.911	KN
ML	-1027.890	-1124.564	kNm
MT	10377.455	10377.455	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure				1545.230	93.888	1313.445	79.805	-8.467
Substructure & Foundation -Portion 1				678.504	28.330	194.765	15.036	0.000
Substructure & Foundation -Portion 2				9965.637	84.398	-12211.905	-113.477	0.000
Total =				12189.370	206.616	-10703.694	-18.636	-8.467

-206.616

18.636

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		237.401	469.442	1761.102	1702.641	3823.30302
due to Substructure		145.480	563.638		766.349	2969.086
due to Active Earth pressure	1.00	1183.020			3777.569	
due to dynamic increment of EP	1.50	340.242			1641.725	

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	12395.986	11982.755	KN
ML	-2834.046	-2796.774	kNm
MT	6783.921	6783.921	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
282.94	2029.25	0.00	0.00	0.00	0.00	682.65	5924.86
192.14	1001.42	0.00	0.00	0.00	0.00	744.39	3879.81
1438.82	5231.46						
338.79	1635.83						
79.78	345.32						
106.19	650.96						
2438.65	10894.24	0.00	0.00	0.00	0.00	1427.04	9804.67

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
237.40	1702.64	0.00	0.00	0.00	0.00	469.44	3823.30
145.48	766.35	0.00	0.00	0.00	0.00	563.64	2969.09
1183.02	3777.57						
340.24	1641.72						
1906.14	7888.28	0.00	0.00	0.00	0.00	1033.08	6792.39

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Forces due to Vertical Load**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	$ML = P \times e_{L1}$	MLs due to F_v	$MT = P \times e_T$
Total =	12717.187	227.524	-11576.051	-0.864	276.532
		-227.524		0.864	

Forces due to Horizontal Load

	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	282.941	573.981	1761.102	2029.254	4853.74002
due to Substructure	145.480	563.638		766.349	2969.0855
due to Earth pressure	1183.020			3777.569	
due to dynamic increment of EP	340.242			1641.725	
due to Live load surcharge	87.058			376.831	
due to dynamic increment of Surcharge	177.447			1028.749	

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
282.94	2029.25	0.00	0.00	0.00	0.00	573.98	4853.74
145.48	766.35	0.00	0.00	0.00	0.00	563.64	2969.09
1183.02	3777.57						
340.24	1641.72						
87.06	376.83						
177.45	1028.75						
2216.19	9620.48	0.00	0.00	0.00	0.00	1137.62	7822.83

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	12944.711	12489.664	kN
ML	-1956.439	-1954.710	kNm
MT	8099.358	8099.358	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.) = $1.50 \times 0.00 = 0.000$ KN
 Transverse Moment due to C.F. = $0.000 \times (1763.787 - 1753.930) = 0.000$ kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = $0.75 \times 0.00 = 0.000$ KN
 Transverse Moment due to C.F. = $0.000 \times (1763.787 - 1753.930) = 0.000$ kNm

Base pressure on corner A = $\sigma_A = P/A - ML/ZL + MT/ZT$
 Base pressure on corner B = $\sigma_B = P/A + ML/ZL + MT/ZT$
 Base pressure on corner C = $\sigma_C = P/A - ML/ZL - MT/ZT$
 Base pressure on corner D = $\sigma_D = P/A + ML/ZL - MT/ZT$

Forces along Long. Axis		Forces along Trans. Axis	
FT Cos θ	MT Cos θ	FT Sin θ	MT Sin θ
0.00	0.00	0.00	0.00

Normal

0.000 KN
0.000 kNm

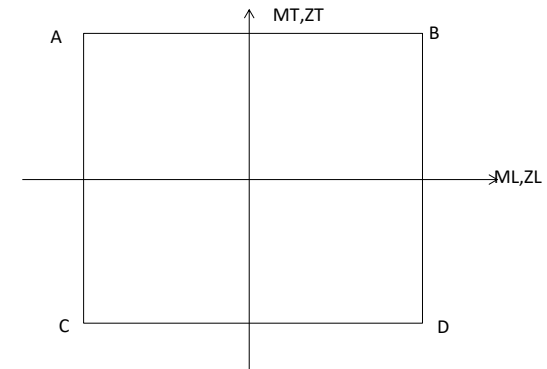
Seismic

0.00	0.00	0.00	0.00

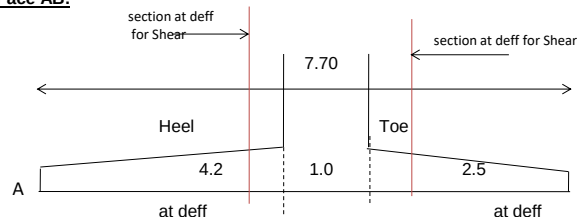
0.000 KN
0.000 kNm

Summary of Design Base Pressure

LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D
	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²
Normal Dry Case							
Case 1 : DL+SIDL-Normal Dry Case	16925.572	-2877.856	-8.467	292.774	224.249	292.957	224.432
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	18130.336	687.997	1154.034	281.266	297.648	256.373	272.755
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	12179.478	-3504.780	-8.467	227.723	144.270	227.906	144.453
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	12889.347	-71.586	561.532	203.843	202.138	191.730	190.026
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	17282.179	8003.318	2302.468	193.600	384.168	143.935	334.504
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	18007.728	10617.415	3514.185	186.631	439.444	110.830	363.643
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	12395.986	6794.893	2029.249	130.385	292.179	86.613	248.408
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	12944.711	10037.372	2623.380	106.573	345.574	49.986	288.988
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	17282.179	-2123.087	7694.649	372.315	321.762	206.341	155.788
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	18007.728	-1027.890	10377.455	399.296	374.821	175.453	150.978
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	12395.986	-2796.774	6783.921	295.859	229.264	149.529	82.934
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	12944.711	-1954.710	8099.358	308.404	261.860	133.700	87.156

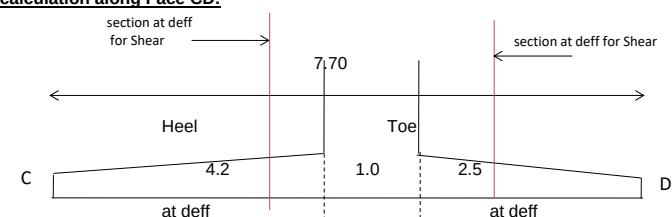


Pressure calculation along Face AB:



Case : 1	292.774	263.558	255.40	246.498	238.337	224.249
Case : 2	281.266	288.251	290.20	292.329	294.280	297.648
Case : 3	227.723	192.142	182.20	171.366	161.427	144.270
Case : 4	203.843	203.116	202.91	202.692	202.489	202.138
Case : 5	193.600	274.851	297.55	322.295	344.990	384.168
Case : 6	186.631	294.422	324.53	357.362	387.470	439.444
Case : 7	130.385	199.368	218.64	239.648	258.917	292.179
Case : 8	106.573	208.474	236.94	267.976	296.439	345.574

Pressure calculation along Face CD:



Case : 1	292.957	263.740	255.58	246.680	238.520	224.432
Case : 2	256.373	263.358	265.31	267.436	269.387	272.755
Case : 3	227.906	192.325	182.39	171.548	161.610	144.453
Case : 4	191.730	191.004	190.80	190.579	190.376	190.026
Case : 5	143.935	225.187	247.88	272.631	295.326	334.504
Case : 6	110.830	218.620	248.73	281.561	311.669	363.643
Case : 7	86.613	155.597	174.86	195.877	215.145	248.408
Case : 8	49.986	151.888	180.35	211.390	239.853	288.988

Case : 9	372.315	350.761	344.74	338.176	332.155	321.762
Case : 10	399.296	388.861	385.95	382.768	379.853	374.821
Case : 11	295.859	267.465	259.53	250.886	242.955	229.264
Case : 12	308.404	288.560	283.02	276.972	271.429	261.860

Average MAX Base Pressure for Design of Heel Slab-along Face AB	=	392.621 kN/m ²
Average MIN Base Pressure for Design of Heel Slab-along Face AB	=	171.755 kN/m ²
Average MAX Base Pressure for Design of Toe Slab-along Face AB	=	398.403 kN/m ²
Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	387.470 kN/m ²
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	388.861 kN/m ²

Case : 9	206.341	184.787	178.77	172.201	166.181	155.788
Case : 10	175.453	165.018	162.10	158.925	156.010	150.978
Case : 11	149.529	121.135	113.20	104.556	96.625	82.934
Case : 12	133.700	113.855	108.31	102.268	96.725	87.156

Average MAX Base Pressure for Design of Heel Slab-along Face CD	=	274.268 kN/m ²
Average MIN Base Pressure for Design of Heel Slab-along Face CD	=	115.168 kN/m ²
Average Base Pressure for Design of Toe Slab-along Face CD	=	322.602 kN/m ²
Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	311.669 kN/m ²
Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	263.740 kN/m ²

Calculation of Moment and Shear Force Along Traffic Direction:

Case 1 : Maximum Base Pressure Case (Dry Case)

Heel Slab - Maximum Moment Calculation

Max Average Base Pressure for Design of Heel Slab	=	392.621 kN/m ²							
Upward moment due to Base pressure	=	3462.919 kNm/m							
		factor							
Downward moment due to backfill	=	1.35 x	284.199	/	8.500	x	20	x	2.100
	=	1895.773 kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	23.205	/	8.500	x	25	x	1.723
	=	158.760 kNm/m							

Net Moment at face of shaft	=	3462.919	-1895.773	-158.760	=	1408.386 kNm/m	Tension at Bottom of Heel Slab

Case 2 : Minimum Base Pressure Case (HFL Case)**Heel Slab - Maximum Moment Calculation**

Min Average Base Pressure for Design of Heel Slab	=	115.168	kN/m ²							
Upward moment due to Base pressure	=	1015.784	kNm/m							
Downward moment due to backfill	=	1.35 x	284.199	/	8.500	x	10	x	2.100	
	=	947.887	kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	23.205	/	8.500	x	15	x	1.723	
	=	95.256	kNm/m							

Net Moment at face of shaft	=	1015.784	-947.887	-95.256	=	-27.358	kNm/m	Tension at Top of Heel Slab	

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	0.847	m							
effective depth at critical section	=	0.764	m							
Base pressure at deff from face of wall	=	388.861	kN/m ²							
Shear Force due to upward pressure at deff from face of wall	=	394.079	x	3.283	x	8.500	=	10996.961	KN	
		factor								
Downward Force due to backfill	=	1.35 x	284.199	x	20	=	7673.367	KN		
Downward Force due to self weight of Heel slab	=	1.35 x	23.205	x	25	=	783.169	KN		
Net Shear Force	=	10996.961	-7673.367	-783.169	=	2540.425	KN			
Net Shear Force / unit meter	=	2540.425	/	8.500	=	298.874	KN/m			

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	398.403	kN/m ²							
Upward moment due to Base pressure	=	1245.010	kNm/m							
Downward moment due to self weight of Toe slab	=	1.35 x	13.813	/	8.500	x	25	x	1.026	
	=	56.2500	kNm/m							

Net Moment at face of shaft	=	1245.010	-56.250	=	1188.760	kNm/m	Tension at Bottom of Toe Slab	

Toe Slab - Shear Calculation at deff from Face of Wall

For shear, critical section is assumed to be located at a distance equal to effective depth from face of wall

Depth of slab at critical section	=	0.743	m							
effective depth at critical section	=	0.656	m							
Base pressure at deff from face of wall	=	387.470	kN/m ²							
upward shear force due to base pressure	=	413.457	x	1.583	x	8.500	=	5563.274	KN	
C.g. Of base pressure	=	0.679	m							
moment due to upward pressure at critical section	=	3779.719	kNm							
tanβ	=	0.280								
reduction in shear force (V _{cod})	=	M tanβ	=	1423.929	KN					
		d								
Downward force due to self weight of toe slab	=	1.35 x	0.522	x	1.583	x	8.500	x	25	
	=	236.880	KN							
Net Shear Force at deff	=	5563.274	-	236.880	-	1423.929	=	3902.465	KN	
Net Shear Force / unit meter	=	3902.465	/	8.500	=	459.113	KN/m			

Design Input :

Design length	=	1000	mm
Clear Cover For Foundation	=	75	mm
Grade of Concrete for Footing	=	M 35	
fck	=	35.00	N/mm ²

	f_{ctm}	=	2.77 N/mm ²	
Grade of Reinforcement Steel		=	Fe 500D	(HYSD Bars)
	f_y or f_{yk}	=	500.00 N/mm ²	
	f_{yd}	=	434.78 N/mm ²	($f_y/1.15$)
	E_s	=	200000.00 N/mm ²	

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	Toe Slab
Ultimate bending moment, M_u (kNm/m)	=	27.36	1188.76
Effective depth required (d _{req}) (mm)	=	68.42	450.98
Effective depth provided (d _{pro}) (mm)	=	917.00	917.00
Check for provided depth	=	SAFE	SAFE
$R = M_u/(b d^2)$	=	0.03	1.41
Total depth provided (mm)	=	1000.00	1000.00
Limiting depth of neutral axis (mm)	=	565.66	565.66
Actual depth of neutral axis (mm)	=	69.38	138.89
Check for Neutral axis depth	=	OK	OK
Lever arm (z) , mm	=	888.14	859.22
Moment of Resistance w.r.to steel	=	776.39	1503.70
Check for Moment Capacity	=	SAFE	SAFE
Ast reqd (mm ² / m)	=	74.88	3416.87
cl. 16.6.1 (2) of IRC :112-2011			
$A_{s,min} = 0.26 f_{ctm} b_t d / f_{yk} \geq 0.0013 b_t d$	=	1321.45	1321.45
Governing Ast (mm ² / m)	=	1321.45	3416.87
Tension Reinforcement			
Dia (mm)	=	16.00	25.00
Spacing (mm)	=	200.00	200.00
+ Dia (mm)	=	16.00	20.00
Spacing (mm)	=	200.00	200.00
Ast provided (mm ² / m)	=	2010.62	4025.17
Check for Ast provided	=	OK	OK
As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement			
Secondary Reinforcement (mm ² /m)	=	402.12	805.03
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	100.00
Ast provided (mm ² /m)	=	565.49	1130.97
Check for Ast provided	=	OK	OK

Shear Reinforcement Calculation:

		Along Traffic Direction		
		Heel Slab	Toe Slab	
Ultimate Shear Force (V_{Ed})	=	298.874	459.113	kN/m
Ast provided	=	2010.619	4025.17	mm ² /m
Depth of slab at critical section	=	847.167	743.240	mm
Effective depth at critical section	=	764.167	655.740	mm
Percentage of steel provided (ρ_1)	=	0.0024	0.0054	
cl. 10.3.1 of IRC :112-2011				
$\rho_1 = A_s/(b_w d) \leq 0.02$	=	OK	OK	
Actual shear stress= $v_{ED} = (V_{Ed}/b*0.9d)$	=	0.435	0.778	N/mm ²
Max shear capacity, $0.135 f_{ck}(1-f_{ck}/310)$	=	4.192	4.192	N/mm ²
Depth Check for Shear Resistance	=	SAFE	SAFE	

cl. 10.3.2(2) Eq. 10.2 of IRC :112-2010					
$K = 1 + \sqrt{200/d} \leq 2.0$	=	1.512	1.552		
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010					
$v_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	=	0.341	0.355		N/mm ²
$0.12 K (80 \rho_1 f_{ck})^{0.33}$	=	0.339	0.457		N/mm ²
$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 f_{cd}$	=	0.000	0.000		N/mm ²
cl. 10.3.2(2) Eq. 10.1 of IRC :112-2011					
$V_{Rd,c} = [0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}]b_w d$ subjected to minimum ($v_{min} + 0.15 \sigma_{cp}$) $b_w d$	=	312.547	418.972		kN
Check for Shear Reinforcement		OK, No shear reinf. Req.	Provide Shear Reinf.		
Balance Shear Force= $V_{Rd,s} = V_{Ed} - V_{Rd,c}$	=	0.000	40.141		kN/m
b	=	8.500	8.500		m
Total Shear Force	=	0.000	341.201		kN
$\theta = 0.5 \times \sin^{-1} [v_{Ed} / (0.18 f_{ck} (1 - f_{ck}/250))]$		2.300	4.128		
$\cot \theta = (< 1 \cot \theta < 2.5)$	=	2.500	2.500		
$f_{ywd} = 0.8 \times f_y / 1.15$	=	347.826	347.826		N/mm ²
Provide Shear Reinforcement					
Legged	=	15	15		
Dia	=	12	12		mm
Area of Shear Reinf, A_{sw}	=	1696.460	1696.460		mm ²
$z = 0.9 \times d$	=	687.750	590.166		mm
Spacing of shear Reinforcement required					
$S = A_{sw} z^* f_{ywd} \cot \theta / V_{Rd}$	=	0.000	2551.584		mm
As per Clause 10.3.3.5 of IRC:112-2011					
$A_{sw} / (b S) = \rho_{w,min} = (0.072 f_{ck}^{0.5}) / f_{yk}$	=	0.001	0.001		
Spacing of shear Reinforcement required	=	234.276	234.276		mm
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011					
$S_{max} = 0.75 d$	=	573.125	491.805		mm
Governing Spacing of Shear Reinf.	=	0.000	234.276		mm
Provided Spacing of Shear Reinf.	=	200	200		mm

SLS CHECK OF FOUNDATION

Foundation Lvl = 1753.930 m

Properties of Footing Base:

A	=	65.450	m ²
ZL	=	83.994	m ³
ZT	=	92.721	m ³

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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			815.073	0.850	692.812	0.000	0.000
SIDL except Wearing Course	1.00			147.100	0.850	125.035	0.000	0.000
Wearing Course	1.20			148.005	0.850	125.804	0.000	0.000
Bearing Pedestal	1.00	25	0.902	22.560	0.850	19.176	-0.278	-6.272
				1132.738		962.827		-6.272
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	4.807	120.169	0.050	6.008	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.271	6.773	0.050	0.339	0.000	0.000
Bracket - Uniform portion	1.00	25	0.765	19.125	-0.250	-4.781	0.000	0.000
Bracket - Tapered portion	1.00	25	0.383	9.563	-0.200	-1.913	0.000	0.000
Cap - (uniform portion)	1.00	25	4.718	117.938	0.825	97.298	0.000	0.000
Cap - (corbel portion)	1.00	25	3.579	89.466	0.825	73.809	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1.00	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.050	1.400	0.000	0.000
Approach Slab	1.00	25	4.463	111.563	-0.250	-27.891	0.000	0.000
				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	33.629	840.735	-1.750	-1471.286	0.000	0.000
Abutment Shaft	1.00	25	42.666	1066.644	0.850	906.647	0.000	0.000
Back filling over heel slab	1.00	20	284.199	5683.976	-1.781	-10121.887	0.000	0.000
Front Filling over toe slab	1.00	20	72.866	1457.325	2.557	3727.066	0.000	0.000
Side filling between heel and toe	1.00	20	0.000	0.000	0.000	0.000	0.000	0.000

Heel slab	1.00	25	23.205	580.125	-1.373	-796.556	0.000	0.000
Toe slab	1.00	25	13.813	345.313	2.376	820.339	0.000	0.000
portion between heel & toe	1.00	25	8.500	212.500	0.850	180.625	0.000	0.000
Vertical Components of active earth pressure	1.00			633.271	-3.850	-2438.092	0.000	0.000
				10819.887		-9193.145		0.000
Total				12455.220		-8086.047		-6.272

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		55.509	1761.102	398.110
due to Earth pressure	1.00	1528.850		5231.463

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
55.51	398.11	0.00	0.00
1528.85	5231.46		
1584.359	5629.573	0.000	0.000

Summary of Forces

P	12455.220	KN
ML	-2456.475	kNm
MT	6.272	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical load**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Forces from Case :DL+SIDL		12455.220		-8086.047		-6.272
CWLL-Max. Reaction case	1.00	670.996	0.850	570.347	1.155	775.001
Vertical Components of LL Surcharge	0.80	132.180	-3.850	-508.892	0.000	0.000
Total		13258.396		-8024.592		768.729

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		185.809	1761.102	1332.623
due to Earth pressure	1.00	1528.850		5231.463
due to Live load surcharge	0.80	319.110		1381.267

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
185.81	1332.62	0.00	0.00
1528.85	5231.46		
319.11	1381.27		
2033.769	7945.353	0.000	0.000

Summary of Forces

P	13258.396	kN
ML	-79.239	kNm
MT	768.729	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1132.738		962.827		-6.272
Substructure & Foundation -Portion 1				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	33.629	840.735	-1.750	-1471.286	0.000	0.000
Shaft above HFL	1.00	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1.00	15	42.666	639.986	0.850	543.988	0.000	0.000
Back filling above HFL over heel slab	1.00	20	92.035	1840.692	-1.750	-3221.211	0.000	0.000
Back filling below HFL over heel slab	1.00	10	193.815	1938.153	-1.795	-3479.233	0.000	0.000
Front Filling over toe slab	1.00	10	72.866	728.662	1.642	1196.446	0.000	0.000
Side filling between heel and toe	1.00	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.00	15	23.205	348.075	-1.373	-477.934	0.000	0.000
Toe slab	1.00	15	13.813	207.188	2.376	492.203	0.000	0.000
Portion between Heel & Toe	1.00	15	8.500	127.500	0.850	108.375	0.000	0.000
Vertical Components of active earth pressure	1.00			633.271	-3.850	-2438.092	0.000	0.000
				7304.262		-8746.743		0.000
Total				8939.595		-7639.646		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (kN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		55.509	1761.102	398.110
due to Earth pressure	1.00	1528.850		4411.753

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
55.51	398.11	0.00	0.00
1528.85	4411.75		
1584.359	4809.863	0.000	0.000

Summary of Forces

P	8939.595	kN
ML	-2829.783	kNm
MT	6.272	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical load**

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		8939.595		-7639.646		-6.272
CWLL-Min. Reaction case	1.00	329.004	0.850	279.653	1.155	379.999
Vertical Components of LL Surcharge	0.80	144.243	-3.850	-555.335	0.000	0.000
Total		9412.841		-7915.327		373.727

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		202.909	1761.102	1455.261
due to Earth pressure	1.00	1528.850		4411.753
due to Live load surcharge	0.80	348.233		1507.326

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
202.91	1455.26	0.00	0.00
1528.85	4411.75		
348.23	1507.33		
2079.992	7374.340	0.000	0.000

Summary of Forces

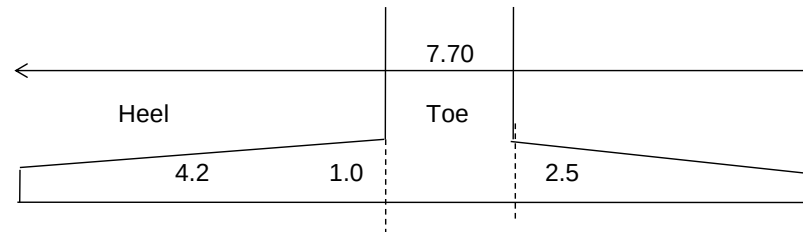
P	9412.841	kN
ML	-540.988	kNm
MT	373.727	kNm

Centrifugal Force : Normal Case

$$\begin{aligned}
 \text{Centrifugal Force (C.F.)} &= 1.00 \times 0.00 = 0.000 \text{ KN} \\
 \text{Transverse Moment due to C.F.} &= 0.000 \times (1763.787 - 1753.930) = 0.000 \text{ kNm}
 \end{aligned}$$

$$\begin{aligned}
 \text{Base pressure on corner A} &= \sigma_A = P/A - ML/ZL + MT/ZT \\
 \text{Base pressure on corner B} &= \sigma_B = P/A + ML/ZL + MT/ZT \\
 \text{Base pressure on corner C} &= \sigma_C = P/A - ML/ZL - MT/ZT \\
 \text{Base pressure on corner D} &= \sigma_D = P/A + ML/ZL - MT/ZT
 \end{aligned}$$

LOAD CASES	Design Base Pressure						
	P	ML	MT	σ_A	σ_B	σ_C	σ_D
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²
Case 1 : DL+SIDL-Normal Dry Case	12455.220	-2456.475	6.272	219.615	161.123	219.479	160.988
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	13258.396	-79.239	768.729	211.807	209.920	195.226	193.339
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	8939.595	-2829.783	6.272	170.345	102.964	170.209	102.829
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	9412.841	-540.988	373.727	154.289	141.407	146.227	133.346

Pressure calculation along Face AB:

Case 1 :	219.615	187.71	180.114	161.123
Case 2:	211.807	210.78	210.533	209.920
Case 4:	170.345	133.59	124.841	102.964
Case 5:	154.289	147.26	145.590	141.407

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 211.293 kN/m²

Average Base Pressure for Design of Toe Slab-along Face AB = 210.227 kN/m²

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 203.662 kN/m²

Average Base Pressure for Design of Toe Slab-along Face AB = 170.619 kN/m²

Pressure calculation along Face CD:

Case 1 :	219.479	187.57	179.979	160.988
Case 2:	195.226	194.20	193.951	193.339
Case 4:	170.209	133.46	124.706	102.829
Case 5:	146.227	139.20	137.528	133.346

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 203.527 kN/m²

Average Base Pressure for Design of Toe Slab-along Face CD = 193.645 kN/m²

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 203.527 kN/m²

Average Base Pressure for Design of Toe Slab-along Face CD = 170.483 kN/m²

Moment Calculation

	Rare Combination		Quasi-Permanent		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	211.29	210.23	203.66	170.62	kN/m ²
Upward moment due to Base pressure	1863.60	656.96	1796.30	533.18	kNm/m
Downward moment due to backfill	1404.28	0.00	1404.28	0.00	kNm/m
Downward moment due to self weight of slab	117.60	41.67	117.60	41.67	kNm/m
Net Moment	341.72	615.29	274.43	491.52	kNm/m
	Tension at Bottom of Heel Slab	Tension at Bottom of Toe Slab	Tension at Bottom of Toe Slab	Tension at Bottom of Toe Slab	

Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff	=	2	
Ecm	=	32308.25 N/mm ²	
Es	=	200000.00 N/mm ²	
Eceff	=	$\frac{E_{cm}}{(1 + \phi)}$	1.08E+04
Modular Ratio (m)	=	Es/ Eceff	18.57

		Rare Combination		Quasi Permanent Comb.		
		Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Working bending moment, M	=	341.72	615.29	274.43	491.52	kNm/m
Dx	=	1.00	1.00			m
Dy	=	1.00	1.00			m
Section Modulus (ZL) of uncracked sec	=	0.17	0.17			m ³
Bending Stress (M/ZL)	=	2.050	3.692			N/mm ²
Tensile stress of concrete , fctm	=	2.771	2.771			N/mm ²
Cracked or Uncracked Section	=	Uncracked	Cracked			
Section properties of Cracked section:						
Note: Stresses under Service load are usually within Linear Elastic Range hence such analysis involved use of Modulus ratio.						
Clear Cover, c	=	75.000	75.000			mm
Maximum dia used, ϕ	=	16.000	25.000			mm
Effective Depth deff (dy)	=	917.000	917.000			mm
Ast provided	=	2010.619	4025.166			mm ² /m
Percentage of steel , pt	=	0.0022	0.0044			
$k = \sqrt{2 \text{ pt} * m + (\text{pt} * m)^2} - \text{pt} * m$	=	0.248	0.330			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	226.999	302.982			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	690.001	614.018			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	607.001	526.518			mm
Cracked moment of Inertia (Icr)	=	$Dx * (k * dy)^3 / 3 + m \text{ Ast} * (dy - k * dy)^2$				
Icr	=	2.168E+10	3.745E+10			mm ⁴
Maximum compressive stress in concrete	=	3.579	4.977	2.874	3.976	< 16.8, SAFE

Maximum Tensile stress in steel	=	177.712	160.633	142.714	128.320	< 300, SAFE

Check For Crack Width in Quasi-Permanent Load Combination

Crack width , W_k = $S_r \max (\epsilon_{sm} - \epsilon_{cm})$

Above Formula For Calculation of $S_r \max$ is applicable if the spacing between the reinf. is less or equal to $5*(c+\phi/2)$

$5*(c+\phi/2)$	=	415.000	437.500	mm
Provided Spacing	=	100.000	100.000	mm
Check for Applicability of Formula	=	OK	OK	
Maximum crack spacing , $S_{r \max}$	=	$3.4 c +$	$0.425 k_1 k_2 \phi$	
		$\rho_{p \text{ eff}}$		
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , y_c	=	226.999	302.982	mm
$\rho_{p \text{ eff}} = A_s/A_{c \text{ eff}}$	=	, where $A_{c \text{ eff}}$ = effective area of concrete in tension surrounding the reinf.		
$h_{c \text{ eff}} = \text{Min of } 2.5 (D_y - d_y) , D_y - y_c/3 , D_y/2$	=	207.500	207.500	mm
$A_{c \text{ eff}} = D_x * h_{c \text{ eff}}$	=	207500.000	207500.000	mm
$\rho_{p \text{ eff}} = A_s/A_{c \text{ eff}}$	=	0.010	0.019	
Maximum crack spacing , $S_{r \max}$	=	535.710	474.090	mm
$(\epsilon_{sm} - \epsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct \text{ eff}} (1 + \alpha_e \rho_{p \text{ eff}})}{\rho_{p \text{ eff}}}$	/ E_s	
tensile stress in steel , σ_{sc}	=	142.714	128.320	N/mm ²
K_t	=	0.500	0.500	
Tensile strength of concrete = $f_{ct \text{ eff}} = f_{ctm}$	=	2.771	2.771	N/mm ²
$\alpha_e = E_s/E_{cm}$	=	6.190	6.190	
$(\epsilon_{sm} - \epsilon_{cm})$	=	0.00043	0.0004	
Crack width , $W_k = S_r \max (\epsilon_{sm} - \epsilon_{cm})$	=	0.229	0.183	mm
Check	=	SAFE	SAFE	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom lvl = 1754.930 m

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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	1.35			1100.348	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1.35			198.585	0.000	0.000	0.000	0.000
Wearing Course	1.75			215.841	0.000	0.000	0.000	0.000
Bearing Pedestal	1.35	25	0.902	30.456	0.000	0.000	-0.278	-8.467
				1545.230		0.000		-8.467
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	-0.800	-129.782	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	-0.800	-7.315	0.000	0.000
Bracket - Uniform portion	1.35	25	0.765	25.819	-1.100	-28.401	0.000	0.000
Bracket - Tapered portion	1.35	25	0.383	12.909	-1.050	-13.555	0.000	0.000
Cap - (uniform portion)	1.35	25	4.718	159.216	-0.025	-3.980	0.000	0.000
Cap - (corbel portion)	1.35	25	3.579	120.779	-0.025	-3.019	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.800	-30.240	0.000	0.000
Approach Slab	1.35	25	4.463	150.609	-1.100	-165.670	0.000	0.000
				678.504		-381.963		0.000
Substructure-Portion 2								
Abutment Shaft	1.35	25	42.666	1439.969	0.000	0.000	0.000	0.000
Total				3663.703		-381.963		-8.467

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		75.739	1761.102	467.459
due to Earth pressure	1.5	1794.067		5429.854

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
75.74	467.46	0.00	0.00
1794.07	5429.85	0.00	0.00
1869.806	5897.313	0.000	0.000

Summary of Forces

P	3663.703	kN
ML	5515.350	kNm
MT	8.467	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical Load**

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		3663.703		-381.963		-8.467
CWLL-Max. Reaction case	1.5	1006.495	0.000	0.000	1.155	1162.501
Total		4670.197		-381.963		1154.034

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		271.189	1761.102	1673.778
due to Earth pressure	1.5	1794.067		5429.854
due to Live load surcharge	1.2	423.373		1620.882

Summary of Forces

P	4670.197	KN
ML	8342.551	kNm
MT	1154.034	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1545.230		0.000		-8.467
Substructure-Portion 1				678.504		-381.963		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	0.000	0.000	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	23.500	42.666	1353.571	0.000	0.000	0.000	0.000
Total				3577.305		-381.963		-8.467

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		75.739	1761.102	467.459
due to Earth pressure	1.5	1794.067		4579.058

Summary of Forces

P	3577.305	KN
ML	4664.554	kNm
MT	8.467	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical Load**

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		3577.305		-381.963		-8.467
CWLL-Max. Reaction case	1.5	493.505	0.000	0.000	1.155	569.999
Total		4070.810		-381.963		561.532

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
271.19	1673.78	0.00	0.00
1794.07	5429.85		
423.37	1620.88		
2488.629	8724.514	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
75.74	467.46	0.00	0.00
1794.07	4579.06	0.00	0.00
1869.806	5046.517	0.000	0.000

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		296.838	1761.102	1832.087
due to Earth pressure	1.5	1794.067		4579.058
due to Live load surcharge	1.2	462.011		1768.809

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
296.84	1832.09	0.00	0.00
1794.07	4579.06		
462.01	1768.81		
2552.917	8179.953	0.000	0.000

Summary of Forces

P	4070.810	KN
ML	7797.990	kNm
MT	561.532	kNm

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

Seismic Effect Factor = **1.50** $\alpha_h =$ 0.180 In Longitudinal direction
 $\alpha_h =$ 0.282 In Transverse direction
 $\alpha_v =$ 0.188 In Vertical direction

Weight of shaft below Ground level = 654.3 KN

Forces due to Vertical Load

Lloads	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	1.35			1100.348		103.593	69.062	0.000	0.000	0.000	1762.049		0.000	0.000	737.447
SIDL except Wearing Course	1.35			198.585		18.696	12.464	0.000	0.000	0.000	1763.036		0.000	0.000	151.548
Wearing Course	1.75			215.841		15.676	10.451	0.000	0.000	0.000	1762.587		0.000	0.000	120.030
Bearing Pedestal	1.35	25	0.9024	30.456		2.867	1.912	0.000	0.000	0.000			-0.278	-8.467	
				1545.230		140.833	93.888		0.000	0.000				-8.467	1009.026
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	32.446	15.273	10.182	-0.800	-129.782	-8.146	1761.645	217.856	0.000	0.000	102.551
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	1.829	0.861	0.574	-0.800	-7.315	-0.459	1760.649	10.459	0.000	0.000	4.923
Bracket - Uniform portion	1.35	25	0.765	25.819				-1.100	-28.401						
Bracket - Tapered portion	1.35	25	0.383	12.909				-1.050	-13.555						
Cap - (uniform portion)	1.35	25	4.718	159.216	31.843	14.990	9.993	-0.025	-3.980	-0.250	1760.400	174.166	0.000	0.000	81.985
Cap - (corbel portion)	1.35	25	3.579	120.779	24.156	11.371	7.581	-0.025	-3.019	-0.190	1760.100	124.873	0.000	0.000	58.782
RCC Railing or Crash Barrier	1.35	25		37.800				-0.800	-30.240				0.000		
Approach Slab	1.35	25	4.463	150.609				-1.100	-165.670				0.000		
				678.504	90.273	42.494	28.330		-381.963	-9.044		527.354		0.000	248.241
Substructure-Portion 2															
Abutment Shaft	1.35	25	42.666	1439.969	157.136	73.969	49.313	0.000	0.000	0.000	1758.979	636.284	0.000	0.000	299.518
Total =				3663.703	247.410	257.296	171.531		-381.963	-9.044		1163.638		-8.467	1556.785
							-171.531								
										9.044					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		586.176	140.833	1761.102	3617.876	1009.026
due to Substructure		247.410	116.463		1163.638	547.760
due to Active Earth pressure	1.00	1196.045			3619.902	
due to dynamic increment of EP	1.50	1129.288			4323.479	

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
586.18	3617.88	0.00	0.00	0.00	0.00	140.83	1009.03
247.41	1163.64	0.00	0.00	0.00	0.00	116.46	547.76
1196.04	3619.90						
1129.29	4323.48						
3158.92	12724.89	0.00	0.00	0.00	0.00	257.30	1556.79

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3835.233	3492.172	KN
ML	12333.888	12351.976	kNm
MT	1548.318	1548.318	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Forces due to Vertical Load**

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				1545.230		140.833	93.888		0.000	0.000				-8.467	1009.026
Forces from Substructure				2118.473	247.410	116.463	77.642		-381.963	-9.044		1163.638		0.000	547.760
CWLL-Max. Reaction case	0.20			134.20		17.056	11.371	0.000	0.000	0.000	1763.787		1.155	155.000	151.068
Total =				3797.902	247.410	274.352	182.901		-381.963	-9.044		1163.638		146.533	1707.853
							-182.901			9.044					

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		631.716	157.889	1761.102	3898.948	1160.094
due to Substructure		247.410	116.463		1163.638	547.760
due to Active Earth pressure	1.00	1196.045			3619.902	
due to dynamic increment of EP	1.50	1129.288			4323.479	
due to Live load surcharge	0.20	70.562			270.147	
due to dynamic increment of Surcharge	1.50	353.963			1815.895	

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
631.72	3898.95	0.00	0.00	0.00	0.00	157.89	1160.09
247.41	1163.64	0.00	0.00	0.00	0.00	116.46	547.76
1196.04	3619.90						
1129.29	4323.48						
70.56	270.15						
353.96	1815.90						
3628.98	15092.01	0.00	0.00	0.00	0.00	274.35	1707.85

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3980.803	3615.000	kN
ML	14701.003	14719.091	kNm
MT	1854.386	1854.386	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Forces due to Vertical Load**

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				1545.230		140.833	93.888		0.000	0.000				-8.467	1009.026
Substructure-Portion 1				678.504	90.273	42.494	28.330		-381.963	-9.044		527.354		0.000	248.241
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1759.979	0.000	0.000	0.000	0.000
Shaft below HFL	1.350	23.500	42.666	1353.571	139.857	65.835	43.890	0.000	0.000	0.000	1759.009	570.475	0.000	0.000	268.540
				1353.571	139.857	65.835	43.890		0.000	0.000		570.475		0.000	268.540
Total =				3577.305			166.108		-381.963	-9.044				-8.467	
							-166.108			9.044					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		586.176	140.833	1761.102	3617.876	1009.026
due to Substructure		230.130	108.329		1097.829	516.781
due to Active Earth pressure	1.00	958.113			2708.749	
due to dynamic increment of EP	1.50	1134.139			4338.276	

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
586.18	3617.88	0.00	0.00	0.00	0.00	140.83	1009.03
230.13	1097.83	0.00	0.00	0.00	0.00	108.33	516.78
958.11	2708.75						
1134.14	4338.28						
2908.56	11762.73	0.00	0.00	0.00	0.00	249.16	1525.81

Summary of Forces

	Downward	Upward	
P	3743.412	3411.197	KN
ML	11371.722	11389.811	kNm
MT	1534.274	1534.274	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Forces due to Vertical Load**

Lloads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 = 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				1545.230		140.833	93.888		0.000	0.000				-8.467	1009.026
Forces from Substructure				2032.075	230.130	108.329	72.219		-381.963	-9.044		1097.829		0.000	516.781
CWLL-Min. Reaction case	0.20			65.80		8.363	5.575	0.000	0.000	0.000	1763.787		1.155	76.000	74.072
Total =				3643.105	230.130	257.525	171.683		-381.963	-9.044		1097.829		67.533	1599.879

-171.683

9.044

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		631.716	149.196	1761.102	3898.948	1083.097
due to Substructure		230.130	108.329		1097.829	516.781
due to Active Earth pressure	1.00	958.113			2708.749	
due to dynamic increment of EP	1.50	1134.139			4338.276	
due to Live load surcharge	0.20	77.002			294.801	
due to dynamic increment of Surcharge	1.50	591.491			2837.672	

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
631.72	3898.95	0.00	0.00	0.00	0.00	149.20	1083.10
230.13	1097.83	0.00	0.00	0.00	0.00	108.33	516.78
958.11	2708.75						
1134.14	4338.28						
77.00	294.80						
591.49	2837.67						
3622.59	15176.28	0.00	0.00	0.00	0.00	257.52	1599.88

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3814.788	3471.422	KN
ML	14785.269	14803.357	kNm
MT	1667.412	1667.412	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				3663.703	171.531 -171.531	-381.963	-9.044 9.044	-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		175.853	469.442113	1761.102	1085.363	3363.41861
due to Substructure		74.223	388.210624		349.091	1825.86541
due to Active Earth pressure	1.00	1196.045			3619.902	
due to dynamic increment of EP	1.50	338.786			1297.044	

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
175.85	1085.36	0.00	0.00	0.00	0.00	469.44	3363.42
74.22	349.09	0.00	0.00	0.00	0.00	388.21	1825.87
1196.04	3619.90						
338.79	1297.04						
1784.91	6351.40	0.00	0.00	0.00	0.00	857.65	5189.28

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3835.233	3492.172	kN
ML	5960.393	5978.481	kNm
MT	5197.751	5197.751	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				3797.902	182.901 -182.901	-381.963	-9.044 9.044	146.533

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		221.393	526.296618	1761.102	1366.436	3866.97897
due to Substructure		74.223	388.210624		349.091	1825.865
due to Earth pressure	1.00	1196.045			3619.902	
due to dynamic increment of EP	1.50	338.786			1297.044	
due to Live load surcharge	0.20	70.562			270.147	
Surcharge	1.50	106.189			544.769	

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
221.39	1366.44	0.00	0.00	0.00	0.00	526.30	3866.98
74.22	349.09	0.00	0.00	0.00	0.00	388.21	1825.87
1196.04	3619.90						
338.79	1297.04						
70.56	270.15						
106.19	544.77						
2007.20	7447.39	0.00	0.00	0.00	0.00	914.51	5692.84

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3980.803	3615.000	kN
ML	7056.381	7074.469	kNm
MT	5839.377	5839.377	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				3577.305	166.108 -166.108	-381.963	-9.044 9.044	-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		175.853	469.442113	1761.102	1085.363	3363.419
due to Substructure		69.039	361.097138		329.349	1722.605
due to Active Earth pressure	1.00	958.113			2708.749	
due to dynamic increment of EP	1.50	340.242			1301.483	

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
175.85	1085.36	0.00	0.00	0.00	0.00	469.44	3363.42
69.04	329.35	0.00	0.00	0.00	0.00	361.10	1722.60
958.11	2708.75						
340.24	1301.48						
1543.25	5424.94	0.00	0.00	0.00	0.00	830.54	5086.02

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3743.412	3411.197	kN
ML	5033.936	5052.024	kNm
MT	5094.491	5094.491	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				3643.105	171.683 -171.683	-381.963	-9.044 9.044	67.533

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		221.393	497.319073	1761.102	1366.436	3610.32485
due to Substructure		69.039	361.097138		329.349	1722.605
due to Earth pressure	1.00	958.113			2708.749	
due to dynamic increment of EP	1.50	340.242			1301.483	
due to Live load surcharge	0.20	77.002			294.801	
Surcharge	1.50	177.447			851.302	

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
221.39	1366.44	0.00	0.00	0.00	0.00	497.32	3610.32
69.04	329.35	0.00	0.00	0.00	0.00	361.10	1722.60
958.11	2708.75						
340.24	1301.48						
77.00	294.80						
177.45	851.30						
1843.24	6852.12	0.00	0.00	0.00	0.00	858.42	5332.93

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3814.788	3471.422	kN
ML	6461.112	6479.200	kNm
MT	5400.462	5400.462	kNm

Centrifugal Force : Normal Case

$$\begin{aligned} \text{Centrifugal Force (C.F.)} &= 1.50 \times 0.00 \\ \text{Transverse Moment due to C.F.} &= 0.000 \times (1763.787 - 1754.930) = 0.000 \text{ kNm} \end{aligned}$$

Centrifugal Force : Seismic Case

$$\begin{aligned} \text{Centrifugal Force (C.F.)} &= 0.20 \times 0.00 \\ \text{Transverse Moment due to C.F.} &= 0.000 \times (1763.787 - 1754.930) = 0.000 \text{ kNm} \end{aligned}$$

$$\begin{aligned} &= 0.000 \text{ KN} \\ &= 0.000 \text{ kNm} \end{aligned}$$

Normal

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

Seismic

0.00	0.00	0.00	0.00

Summary of ULS Forces for Design of Abutment Shaft

		Total forces at bottom of abutment shaft		
LOAD CASES		P	ML	MT
Normal Dry Case		kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case		3663.703	5515.350	8.467
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		4670.197	8342.551	1154.034
Normal HFL Case				
Case 3 : DL+SIDL-Normal HFL Case		3577.305	4664.554	8.467
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		4070.810	7797.990	561.532
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	3835.233	12333.888	1548.318
	UP	3492.172	12351.976	1548.318
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	3980.803	14701.003	1854.386
	UP	3615.000	14719.091	1854.386
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	3743.412	11371.722	1534.274
	UP	3411.197	11389.811	1534.274
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	3814.788	14785.269	1667.412
	UP	3471.422	14803.357	1667.412

Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	3835.233	5960.393	5197.751
	UP	3492.172	5978.481	5197.751
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	3980.803	7056.381	5839.377
	UP	3615.000	7074.469	5839.377
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	3743.412	5033.936	5094.491
	UP	3411.197	5052.024	5094.491
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	3814.788	6461.112	5400.462
	UP	3471.422	6479.200	5400.462
MAX =		4670.20	14803.36	5839.38

Design of Wall:**Material Property:**

Grade of Concrete	=	M 35
fck	=	35 N/mm ²
fcd	=	15.633 N/mm ²
Grade of steel	=	Fe 500
fy	=	500 N/mm ²
fyd	=	434.783 N/mm ²
Es	=	200000.00 N/mm ²

Cross section of Wall:

Thickness of Wall (B)	=	1.000 m
Depth of Wall (D)	=	8.500 m
Area of Concrete (Ac)	=	8.500 m ²
Clear Cover to earth faces	=	75 mm
Clear Cover to non earth faces	=	50 mm
Maximum Dia of Vertical Reinf.	=	32 mm
Dia of Horizontal Reinf.	=	16 mm

As per Clause 7.6.4.1 of IRC:112-2011

Ultimate axial force (Pu) = 4670.20 kN

$$0.1 f_{cd} A_c = 0.1 \times 15.63 \times 8500000 = 13288333.3 \text{ N} = 13288.33 \text{ kN}$$

Since Axial Force is less than axial capacity of section , Section will design as bending element . Neglecting axial force

PART 1: LONGITUDINAL MOMENT : VERTICAL REINFORCEMENT ON EARTH FACE

Ultimate Design bending moment (ML)	=	14803.36 kNm	=	1741.571 kNm/m

Check For Depth of Wall :

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 1741.57 \text{ kNm/m}$$

$$b = 1000.00 \text{ mm}$$

$$\text{Effective Depth Required (dreq)} = \text{SQRT} \left(\frac{1741.57 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$(dreq) = 549.155 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 656.15 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 1000.00 \text{ mm}$$

$$\text{Effective depth provided (deff)} = 893 \text{ mm}$$

OK

$$R = \frac{Mu}{(b d^2)} = 2.18$$

Minimum Longitudinal Reinforcement in wall on each face

$$A_{st \text{ min}} = 0.0012 \times b \times D = 1200.00 \text{ mm}^2/\text{m}$$

Area of Steel Required:

$$\frac{p_t}{100} = \frac{A_{st \text{ req}}}{b D} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$$

$$A_{st \text{ req}} = 5444.283 \text{ mm}^2/\text{m}$$

$$A_{st \text{ required}} = \max(A_{st \text{ min}}, A_{st \text{ req}}) = 5444.28 \text{ mm}^2/\text{m}$$

$$\text{Total area of steel required in full length} = 46276.40 \text{ mm}^2$$

Provide	32	mm dia	@	200.00	mm c/c	=	6475.61	mm ² /m	OK
Provide	25	mm dia	@	200.00	mm c/c	=			

Effective length of shaft = 8254 mm

Calculation of reinforcement in numbers

Provide	32 mm dia	-	42.00 nos	=	54885.98	mm ²	OK
Provide	25 mm dia	-	43.00 nos	=			

Percentage of steel = 0.646 %

Check for Moment of Resistance of Section due to Steel

$$\begin{aligned} \text{Limiting Depth of Neutral Axis, } X_m &= \frac{0.0035 \cdot d}{(0.0035 + f_{yd}/E_s)} \\ &= \frac{0.0035}{0.0035 + \frac{893.00}{200000}} \times 893.00 \\ &= 550.85 \text{ mm} \end{aligned}$$

$$\begin{aligned} \text{Depth of Neutral Axis, } X &= \frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b} \\ &= \frac{434.78}{0.36} \times \frac{6475.61}{35.00 \times 1000.00} \\ &= 223.45 \text{ mm} \quad \text{OK} \end{aligned}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned} z &= d - 0.416 \cdot X \\ &= 893.00 - 92.96 \\ &= 800.04 \text{ mm} \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned} MR &= f_{yd} \cdot A_{st} \cdot z \\ &= 434.78 \times 6475.61 \times 800.04 \\ &= 2.25E+09 \text{ Nmm/m} \\ &= 2252.51 \text{ kNm/m} > 1741.57 \text{ kNm/m} \end{aligned}$$

Moment of Resistance of Wall is More than Design Bending Moment, HENCE Wall IS SAFE IN BENDING

LONGITUDINAL REINFORCEMENT ON NON EARTH FACE

Minimum Longitudinal Reinforcement in wall on each face

$$\begin{aligned} A_{st \text{ min}} &= 0.0012 \times b \times D \\ &= 1200.00 \text{ mm}^2/\text{m} \\ &= 10200.00 \text{ mm}^2 \end{aligned}$$

Provide	20 mm dia	@	200.00 mm c/c	=	1570.80 mm ² /m	OK

Calculation of reinforcement in numbers

Provide	20 mm dia	@	42.00 nos	=	13194.69 mm ²	OK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

$$\begin{aligned} \text{maximum of following} &= 0.2500 \times 8046.40 = 2011.601 \text{ As per IRC: 112-2011, Clause 16.3.2} \\ &= 0.001 \times 1.00E+06 = 1000.000 \\ \text{Maximum Horizontal Reinf.} &= 2012 \text{ mm}^2 \text{ per meter} \\ \text{Min dia of bar} &= 0.250 \times 32 = 8 \text{ mm} \\ &\text{or } 8 \text{ mm} \\ \text{Maximum Spacing between} &\leq 300 \text{ mm c/c} \end{aligned}$$

2 Legged	16 dia	@	150 c/c	=	2680.826 mm ²	OK

Confinement Reinforcement

As per Clause 17.2.1.3 (Note 2) of IRC:112-2011

$$\text{Distance between links or ties (ST)} = \frac{1}{3} \times 843 = 281$$

$$\text{or } 200.00 \text{ mm}$$

$$\text{Governing Spacing} = 200.00 \text{ mm}$$

As per Clause 17.2.1.3 (Note 1) of IRC:112-2011

The Spacing of hoops and ties in the longitudinal direction (SL)

$$\text{SL} = 5 \times 32 = 160 \text{ mm}$$

$$\text{or } 1/5 \times 843 = 168.6 \text{ mm}$$

$$\text{Min} = 100 \text{ mm}$$

2 Legged	16 dia	@	100 c/c	=	4021.239 mm ²	OK
50 Legged	8 dia	@	100 c/c	=	25132.741 mm ²	
75 links	8 dia	@	100 c/c	=	37699.112 mm ²	
66853.092 mm ²						

Minimum Confinement Reinforcement:

$$n_k = \frac{\text{NED}}{A_k f_{ck}} = \frac{4670197.2}{297500000} = 0.0157$$

$$A_C = 8.500 \text{ mm}^2$$

$$A_{CC} = 0.875 \times 8.400 = 7.350 \text{ mm}^2$$

$$\rho_L = 0.00647 \text{ per meter}$$

$$\rho_L = 0.05502$$

$$f_{yd} = 434.783$$

$$f_{cd} = 15.633$$

$$\omega_{wd,req} = 0.37 \frac{A_C}{A_{CC}} n_k + 0.13 \frac{f_{yd}}{f_{cd}} (\rho_L - 0.01)$$

$$\omega_{wd,req} = 0.1695$$

$$\omega_{wd} = \max(\omega_{wd,req}, 0.12) = 0.1695$$

As per Clause 17.2.1.1 (4) of IRC:112-2011

$$\text{Confined Reinforcement} = \omega_{wd} = \rho_w f_{yd} / f_{cd} \quad \text{where, } \rho_w = \frac{A_{sw}}{S_L \cdot b}$$

Volumetric ratio,

$$A_{sw} = 66853.092 \text{ mm}^2$$

$$S_L = 100.000 \text{ mm}$$

$$b = 843.000 \text{ mm}$$

$$\rho_w = 0.793$$

$$\omega_{wd,c} = 22.055$$

$$\omega_{wd,c} \geq \omega_{wd} \quad \text{as per equation 17.7 of IRC:112-2011}$$

$\omega_{wd,c}$	=	22.06 OK

Length of Potential Plastic Hinges

Refer clause 17.2.1.4 of IRC:112-2011

$$n_k = \frac{\text{NED}}{A_k f_{ck}} = 0.0157 < 0.30$$

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1754.930 m

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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			815.073	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1			147.100	0.000	0.000	0.000	0.000
Wearing Course	1			123.338	0.000	0.000	0.000	0.000
Bearing Pedestal	1	25	0.902	22.560	0.000	0.000	-0.278	-6.272
				1108.070		0.000		-6.272
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	4.807	120.169	-0.800	-96.135	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.271	6.773	-0.800	-5.419	0.000	0.000
Bracket - Uniform portion	1	25	0.765	19.125	-1.100	-21.038	0.000	0.000
Bracket - Tapered portion	1	25	0.383	9.563	-1.050	-10.041	0.000	0.000
Cap - (uniform portion)	1	25	4.718	117.938	-0.025	-2.948	0.000	0.000
Cap - (corbel portion)	1	25	3.579	89.466	-0.025	-2.237	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.800	-22.400	0.000	0.000
Approach Slab	1	25	4.463	111.563	-1.100	-122.719	0.000	0.000
				502.595		-282.936		0.000
Substructure-Portion 2								
Abutment Shaft	1	25	42.666	1066.644	0.000	0.000	0.000	0.000
Total				2677.309		-282.936		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		55.509	1761.102	342.601
due to Earth pressure	1	1196.045		3619.902
				3962.503

Summary of Forces at Bottom of abutment shaft

P	2677.309	KN
ML	3679.568	kNm
MT	-6.272	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical Load**

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		2677.309		-282.936		-6.272
CWLL-Max. Reaction case	1	670.996	0.000	0.000	1.155	775.001
Total		3348.306		-282.936		768.729

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		185.809	1761.102	1146.814
due to Earth pressure	1	1196.045		3619.902
due to Live load surcharge	0.8	282.248		1080.588
				5847.304

Summary of Forces at Bottom of abutment shaft

P	3348.306	KN
ML	5564.368	kNm
MT	768.729	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1108.070		0.000		-6.272
Substructure-Portion 1				502.595		-282.936		0.000

Substructure-Portion 2								
Shaft above HFL	1.000	25.000	0.000	0.000	0.00	0.00	0.00	0.00
Shaft below HFL	1.000	23.500	42.666	1002.645	0.00	0.00	0.00	0.00
				2613.311		-282.936		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		55.509	1761.102	342.601
due to Earth pressure	1	1196.045		3052.705
				3395.306

Summary of Forces at Bottom of abutment shaft

P	2613.311	KN
ML	3112.370	kNm
MT	-6.272	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical Load**

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		2613.311		-282.936		-6.272
CWLL-Max. Reaction case	1	329.004	0.000	0.000	1.155	379.999
Total		2942.314		-282.936		373.727

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		202.909	1761.102	1252.353
due to Earth pressure	1	1196.045		3052.705
due to Live load surcharge	0.8	308.007		1179.206
				5484.263

Summary of Forces at Bottom of abutment shaft

P	2942.314	KN
ML	5201.328	kNm
MT	373.727	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 KN
Transverse Moment due to C.F.	=	0.000	x (	1763.787 - 1754.930)	=	0.000 kNm

Summary of SLS Forces for Design of Abutment Shaft

LOAD CASES	Total forces at bottom of abutment shaft		
Normal Dry Case	P	ML	MT
	kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case	2677.309	3679.568	-6.272
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	3348.306	5564.368	768.729
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	2613.311	3112.370	-6.272
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	2942.314	5201.328	373.727

IN RARE COMBINATION

Max SLS Moment	=	5564.368	kNm
Max Moment per meter	=	654.632	kNm/m

IN QUASI-PERMANENT

Max SLS Moment	=	3679.568	kNm
Max Moment per meter	=	432.890	kNm/m

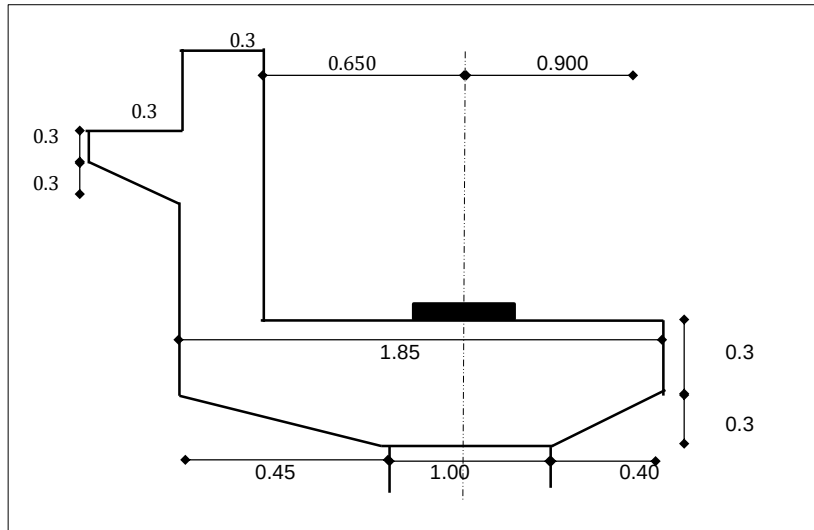
Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff = 2

		Rare Combination		Quasi permanent	
		Short term	Long Term		
Working bending moment, M	=	654.63	654.63	432.89	kNm/m
Dx (unit width of shaft)	=	1.00	1.00	1.00	m
Dy (Thickness of shaft)	=	1.00	1.00	1.00	m
Section Modulus (ZL) of uncracked	=	0.17	0.17	0.17	m ³
Bending Stress (M/ZL)	=	3.928	3.928	2.597	N/mm ²
Tensile stress of concrete , fctm	=	2.771	2.771	2.771	N/mm ²
Cracked or Uncracked Section	=	Cracked	Cracked	Uncracked	
Section properties of Cracked section:					
Note: Stresses under Service load are usually within Linear Elastic Range hence such analysis involved use of Modulus ratio.					
Es	=	200000.00	200000.00	200000.00	N/mm ²
Ecm	=	32308.25	32308.25	32308.25	N/mm ²
Eceff	=	32308.25	10769.42	10769.42	N/mm ²
Modular Ratio (m)	=	6.19	18.57	18.57	
Clear Cover, c	=	75.000	75.000	75.00	mm
Maximum dia used, ϕ	=	32.000	32.000	32.00	mm
Effective Depth deff (dy)	=	893.000	893.000	893.00	mm
Ast provided	=	6475.608	6475.608	6475.61	mm ² /m
Percentage of steel , pt	=	0.0065	0.0065	0.0065	
$k = \sqrt{2 \cdot p_t \cdot m + (p_t \cdot m)^2} - p_t \cdot m$	=	0.246	0.384	0.384	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	219.306	343.161	343.161	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	673.694	549.839	549.839	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	582.694	458.839	458.839	mm
Cracked moment of Inertia (Icr)	=	$Dx \cdot (k \cdot dy)^3/3 + m \cdot Ast \cdot (dy - k \cdot dy)^2$			
Icr	=	2.171E+10	4.983E+10	4.983E+10	mm ⁴
Maximum compressive stress in concrete	=	6.613	4.508	2.981	< 16.8, SAFE
Maximum tensile stress in concrete	=	20.315	7.224	4.777	
Maximum Tensile stress in steel	=	108.768	111.951	74.030	< 300, SAFE

Check For Crack Width in Quasi-Permanent Case

Crack width , Wk	=	Sr max (εsm - εcm)	
Above Formula For Calculation of Sr max is applicable if the spacing between the reinf. is less or equal to 5*(c+φ/2)			
5*(c+φ/2)	=	455.000	mm
Provided Spacing	=	200.000	mm
Check for Applicability of Formula	=	OK	
Maximum crack spacing , $S_{r,max}$	=	$3.4 \cdot c + \frac{0.425 \cdot k_1 \cdot k_2 \cdot \phi}{\rho_{p,eff}}$	
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	343.161	mm
$\rho_{p,eff} = A_s/A_{c,eff}$	=	, where $A_{c,eff}$ =effective area of concrete in tension surrounding the reinf.	
$hc_{eff} = \text{Min of } 2.5 (Dy - dy) , Dy - yc/3 , Dy/2$	=	267.500	mm
$A_{c,eff} = Dx \cdot hc_{eff}$	=	267500.000	mm
$\rho_{p,eff} = A_s/A_{c,eff}$	=	0.024	
Maximum crack spacing , $S_{r,max}$	=	479.720	mm
$(\epsilon_{sm} - \epsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t \cdot f_{ct,eff} (1 + \alpha_e \cdot \rho_{p,eff})}{E_s}$	
tensile stress in steel , σ_{sc}	=	74.030	N/mm ²
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm ²
$\alpha_e = E_s/E_{cm}$	=	18.571	
$(\epsilon_{sm} - \epsilon_{cm})$	=	0.00022	
Crack width , Wk=Sr max (εsm - εcm)	=	0.107	mm
Check	=	< 0.3 ,SAFE	

DESIGN OF ABUTMENT CAP

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl. 710.8.7 of IRC : 78-2014 is 225 mm.

$$\begin{aligned} \text{Assuming a cap thickness of} &= 300 \text{ mm} \\ \text{Volume of abutment cap} &= 300 \times 1850 \times 8500 \\ &= 4.72\text{E}+09 \text{ mm}^3 \end{aligned}$$

as per cl. 710.8.7 of IRC : 78- 2014

$$\begin{aligned} \text{Quantity of steel} &= 1 \% \text{ of volume} \\ &= \frac{1}{100} \times 4.72\text{E}+09 = 4.72\text{E}+07 \text{ mm}^3 \end{aligned}$$

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction

$$= 2.36\text{E}+07 \text{ mm}^3$$

Clear cover

$$= 50 \text{ mm}$$

Length of bar

$$= 8500 - 100 = 8400 \text{ mm}$$

Area of steel required in longitudinal direc

$$= \frac{2.36\text{E}+07}{8400} = 2808.036 \text{ mm}^2 \quad (\text{top +Bottom})$$

Provide	10	Nos. of	16	mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

$$\text{Provided steel} = 2011 \text{ mm}^2$$

(b) Transverse steel

$$\text{Volume of steel to be provided in transverse direction} = 2.36\text{E}+07 \text{ mm}^3$$

$$\text{Volume of steel required per meter} = \frac{2.36\text{E}+07}{8.50} = 2.78\text{E}+06 \text{ mm}^3/\text{m}$$

Provide	2 L	16 mm dia bar @	200 mm c/c stirrups

$$\text{Length of each stirrups} = 1850 - 100 - 16 = 1734 \text{ mm}$$

$$\text{Volume of steel provided per meter} = 3.49\text{E}+06 \text{ mm}^3/\text{m} \quad \text{OK}$$

$$\text{Thickness of Abutment Cap (uniform) } s = 0.3 \text{ m}$$

$$\text{Thickness of Abutment Cap (tapered)} = 0.3 \text{ m}$$

$$\text{C.G. of dirt wall from face of abutment shaft (a)} = 0.300 \text{ m}$$

$$\text{Overall depth of Abutment cap at face of abutment shaft} = 0.600 \text{ m}$$

$$\text{Clear cover} = 50 \text{ mm}$$

$$\text{Diameter of the main bar} = 20 \text{ mm}$$

$$\text{Effective cover } d' = 60 \text{ mm}$$

$$\text{Effective depth of cap } d = 0.540 \text{ m}$$

For the section to be designed as corbel " a / d " shall be less than 1.

$$\begin{aligned} \text{Hence } a/d &= 0.3 / 0.54 \\ &= 0.556 < 1.0 \end{aligned}$$

For the section to be designed as corbel "s / d" shall be greater than 0.5.

$$\begin{aligned} \text{Hence } s/d &= 0.3 / 0.54 \\ &= 0.56 > 0.5 \quad \text{Proceed with the design} \end{aligned}$$

Note: THE ABUTMENT CAP HAS BEEN DESIGNED AS CORBEL FOR DIRT WALL AND LIVE LOAD ON DIRT WALL

1. Dead Load

Self Weight of Dirt Wall	=	14.9344 kN		
Self Weight of Bracket	=	0.135 m ³ /m	x	25
	=	3.375 kN		
Total Dead Load	=	18.3094 kN		
Load Factor	=	1.35		
Ultimate Dead Load	=	24.7177 kN		

2. Live Load

Assuming Class 70R Boggie load, One Axle is Directly over Dirt Wall

Vertical Load on Dirt Wall	=	200 kN
Load Factor	=	1.5
Factored Live load	=	300 kN

Actual horizontal force in normal case	=	40.00 kN
Effective width for this load is considered as (b)	=	1000 mm

Vertical Load (V_u)

Total maximum Vertical Load " V _u "	324.72 kN

Horizontal Load (H_u)

$$\begin{aligned} H_u &= 1.7 \times \text{actual horizontal force in working load condition} \\ &\quad \text{but not less than} \\ &= 0.2 \times V_u \end{aligned}$$

Total maximum Horizontal Load " H _u "	68.00 kN

Now the design for the corbel is carried out as per the following steps.

STEP I

Check for Nominal Shear Strength

Ensure $V_u / b d \leq 0.15 f_c'$

$$\begin{aligned} \text{where; } V_u &= 324.72 \text{ kN} \\ &= 324717.66 \text{ N} \\ bd &= 1000 \times 540 \text{ sqmm} \\ &= 540000.00 \text{ mm}^2 \\ V_u / b d &= 324717.66 / 540000 \\ V_u / b d &= 0.60 \text{ N / mm}^2 \\ f_c' &= 28 - \text{day standard cylinder strength of concrete used.} \\ &= 0.80 \text{ times the standard cube strength} \\ \text{Grade of Concrete} &= \text{M 35} \\ 0.15 f_c' &= 0.15 \times 0.8 \times 35 \\ &= 4.20 \text{ N / mm}^2 \quad \text{Ensured} \end{aligned}$$

Hence $V_u / b d \leq 0.15 f_c'$ is Ensured

STEP II

Calculation of Shear Friction Reinforcement " A_{vf} "

$$A_{vf} = \frac{V_u}{0.87 f_{sy} m}$$

$$\begin{aligned} \text{where; } f_{sy} &= \text{yield stress value of the reinforcement used.} \\ &= 500.00 \text{ N / mm}^2 \end{aligned}$$

Type of Surface		m
1	Concrete placed monolithically across interface.	1.40
2	Concrete placed against hardened concrete but with roughened surface	1.00
3	Concrete anchored to structural steel	0.70
4	Concrete placed against hardened concrete but surface not roughened	0.60

$$\text{Type of Surface (1 / 2 / 3 / 4) ?} = \mathbf{1.00}$$

$$m = 1.40$$

(Note: Only monolithic construction is recommended)

$$A_{vf} = \frac{324.72 \times 1000}{0.87 \times 500 \times 1.4} \text{ mm}^2$$

$$A_{vf} = \mathbf{545.74} \text{ mm}^2$$

STEP III

Calculation for Direct Tension Reinforcement " A_t ".

$$A_t = \frac{H_u}{0.87 f_{sy}}$$

$$H_u = 68.00 \text{ kn}$$

$$A_t = \frac{68 \times 1000}{0.87 \times 500}$$

$$= \mathbf{156.32} \text{ mm}^2$$

STEP IV

Calculation for Flexural Tension Reinforcement " A_f "

$$A_f = \frac{[V_u a + H_u (h - d')]}{0.87 f_{sy} d}$$

$$= \frac{324.72 \times 1000 \times 300 + 68 \times 1000 (600 - 60)}{0.87 \times 500 \times 540}$$

$$= \mathbf{571.03} \text{ mm}^2$$

STEP V

Total Primary Tensile Reinforcement " A_s "

$$A_s \geq (A_f + A_t)$$

$$\geq (2 / 3 A_{vf} + A_t)$$

$$\geq (0.04 f_c' / f_{sy}) b d$$

Provide the largest of these three magnitudes as A_s .

$$(A_f + A_t) = 571.04 + 156.33 \text{ mm}^2$$

$$= 727.35 \text{ mm}^2$$

$$(2 / 3 A_{vf} + A_t) = 2 / 3 \times 545.75 + 156.32 \text{ mm}^2$$

$$= 520.15 \text{ mm}^2$$

$$(0.04 f_c' / f_{sy}) b d = 0.04 \times 0.8 \times 35 / 500 \times 1000 \times 540$$

$$= 1209.60 \text{ mm}^2$$

$$\text{Hence } A_s = \mathbf{1209.60} \text{ mm}^2$$

STEP VI

The total sectional area of the stirrups is " A_h " (closed ties) to be provided horizontally, one below the other, and next to " A_s "

$$A_h \geq 0.25 * A_s$$

$$\geq 0.333 * A_{vf}$$

Provide the largest of these two magnitudes as A_h .

$$0.25 * A_s = 0.25 \times 1209.6$$

$$= 285.52 \text{ mm}^2$$

$$0.333 * A_{vf} = 0.333 \times 545.75$$

$$= 181.73 \text{ mm}^2$$

$$\text{Hence } A_h = 285.52 \text{ mm}^2$$

STEP VII

The total steel in vertical stirrups is " A_v "

$$\begin{aligned} V_c &= 10 \text{ bd in kgs} \quad \text{where } b \text{ \& } d \text{ are in cms} \\ &= 10 \times 100 \times 54 \\ &= 54000.00 \text{ kg} \\ &= 540.00 \text{ kN} \\ \text{Pitch} &= 400 \text{ mm} \\ A_v &= \frac{0.50 * (V_u - V_c) * p}{f_{sy} d} \\ &= \frac{0.5 \times (324.72 - 540) \times 400 \times 1000}{500 \times 540} \\ A_v &= 0.00 \text{ mm}^2 \end{aligned}$$

Reinforcement Details**Total Primary Tensile Reinforcement " A_s "**

$$\begin{aligned} A_s &= 1209.60 \text{ mm}^2 \\ \text{Diameter of Primary Steel} &= 20 \text{ mm} \\ \text{Area of one bar} &= 314.16 \text{ mm}^2 \\ \text{Spacing of Bar} &= 200 \text{ mm c/c} \\ A_{s \text{ provided}} &= 1570.796 \text{ mm}^2 > A_s \quad \text{R/F is adequate} \end{aligned}$$

provide	200 mm c/c 20 mm diameter bars as main reinforcement

Horizontal Steel (Closed Stirrups) " A_h "

$$\begin{aligned} A_h &= 285.52 \text{ mm}^2 \\ A_h / m &= 285.52 \text{ mm}^2 / m \end{aligned}$$

The stirrups shall be provided below A_s and within a depth of " $2 / 3 d$ " below A_s .

$$\begin{aligned} 2 / 3 d &= (2 / 3) \times 540 \\ &= 360.00 \text{ mm} \\ \text{Diameter of Stirrup Bar} &= 12.00 \text{ mm} \\ \text{Area of one bar} &= 113.10 \text{ mm}^2 \\ \text{No. of layers of Stirrups} &= 2.00 \text{ nos.} \\ \text{Spacing of Stirrups} &= 180.000 \text{ mm} \\ \text{No. of legs of stirrups} &= 2.00 \\ A_{h \text{ provided}} &= 113.1 \times 2 \times 2 \\ &= 452.389 \text{ mm}^2 > A_h \quad \text{R/F is adequate} \end{aligned}$$

provide	2 legged. 12 mm diameter bars as horizontal stirrups in 2 layers per meter width

Vertical Steel (Closed Stirrups) " A_v "

A _v	=	0.00	mm ²	
A _v	=	0.25 x A _s	=	302.4 mm ²
A _v max	=	302.40		
A _v /m	=	302.40	mm ² /m	
Diameter of Stirrup Bar	=	10.00	mm	
Area of one bar	=	78.54	mm ²	
Pitch	=	400	mm	
No. of legs of stirrups	=	2.00		
A _h provided	=			
	=	393	mm ²	> A _v R/F is adequate

provide 2 legged. 10 mm diameter bars as vertical stirrups at 400 mm spacing per meter width	

DESIGN OF DIRT WALL

Dirt wall will be designed as a vertical cantilever.

1.) NORMAL CASE

1a. Dead Load

$$\text{Self Weight of Dirt Wall} = 5.078 \text{ m}^3 \times 25.00 = 126.942 \text{ kN}$$

$$\text{Self Weight of Dirt Wall/ m} = 126.942 / 8.50 = 14.934 \text{ kN}$$

1b. Live Load

Assuming Class 70R Boggie load, One Axle is Directly over Dirt Wall

$$\text{Vertical Load on Dirt Wall} = 200 \text{ kN}$$

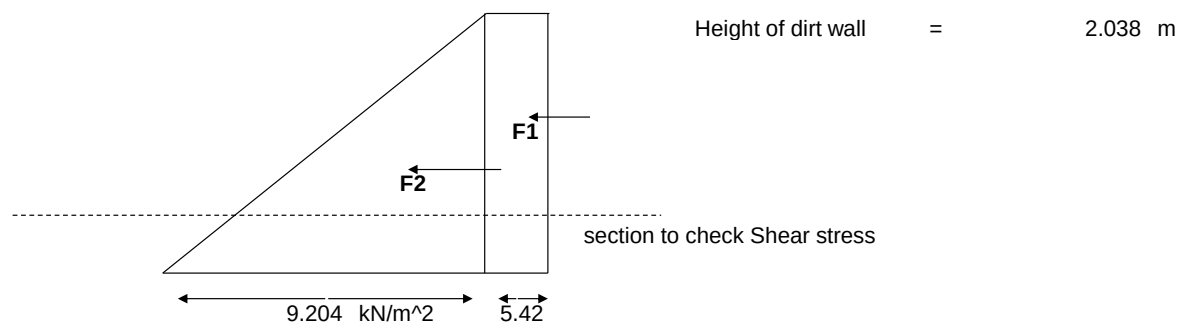
Braking Load

$$\text{Assuming 20\% braking Force i.e. } 0.2 \times 200 = 40.000 \text{ kN acting at 1.2 m above deck}$$

$$\text{Effective Width} = 2.79 \text{ m}$$

$$\text{Moment (due to Braking)} = \frac{40.000 \times 3.238}{2.79} = 46.416 \text{ kNm/m}$$

1c. EARTH PRESSURE



Normal Earth Pressure

Earth Pressure Diagram

$$\text{Intensity for rectangular portion} = 0.226 \times 20 \times 1.2 = 5.421 \text{ kN/m}^2$$

$$F1 = 5.421 \times 2.04 \times 1.00 = 11.045 \text{ kN/m}$$

$$\text{Intensity for triangular portion} = 0.2259 \times 20 \times 2.038 = 9.204 \text{ kN/m}^2$$

$$F2 = 0.50 \times 9.20 \times 2.038 \times 1.00 = 9.377 \text{ kN/m}$$

$$\text{Moment @ RL} = 1760.55 \text{ m (at dirt wall base)}$$

$$M1 = 11.045 \times 1.019 = 11.252 \text{ kN.m/m}$$

(Centre of pressure considered at an elevation of 0.42 x the height of the wall as per cl. 217.1 of IRC:6-2014)

$$M2 = 9.377 \times 0.856 = 8.024 \text{ kN.m/m}$$

Design Horizontal Forces (Normal Case):

$$\text{Load Factor For Live Load Surcharge} = 1.2$$

$$\text{Ultimate Moment due to Live Load Surcharge} = 13.502 \text{ kN.m/m}$$

$$\text{Load Factor For Earth Pressure} = 1.5$$

$$\text{Ultimate Moment due to Earth Pressure} = 12.036 \text{ kN.m/m}$$

$$\text{Load Factor For Braking Force} = 1.5$$

$$\text{Ultimate Moment due to Braking Force} = 69.624 \text{ kN.m/m}$$

Total Ultimate Moment	= 95.162 kN.m/m

Material Property:

Grade of Concrete	=	M 35
Characteristic Strength of Concrete, f_{ck}	=	35.00 Mpa at 28 days
Grade of Reinforcement	=	Fe 500
Yield Strength of Reinforcement, f_y or f_{yk}	=	500 N/mm ²
Design Yield Strength of Reinforcement, f_{yd}	=	434.783 N/mm ²
Modulus of Elasticity of Steel (E_s)	=	200000 N/mm ²

(a) Vertical steel on earth face

As per Clause 16.3.1 of IRC:112-2011

Adopting clear cover on either face	=	50 mm
Minimum Dia of Reinforcement	=	16 mm
Maximum Spacing of Steel	=	200 mm
Thickness of dirtwall	=	0.300 m
Available effective depth	=	300 - 50 - 8 = 242 mm

Check for Depth:

Mult	=	$0.165 \times f_{ck} \times b \times d^2$	=	95.16 kNm/m
Effective Depth of Cap Required (dreq)	=	$\text{SQRT}\left(\frac{95.16 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$	=	128.368 mm
Total Depth Required (Dreq)	=	186.37 mm		
Total Depth Provided (Dprov)	=	300.00 mm		OK
$R = M_u / (b \times d^2)$	=	1.625		

Area of Steel Required:

$\frac{p_t}{100} = \frac{A_{st_{req}}}{b \times d}$	=	$\frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$	=	0.004
$A_{st_{req}}$	=	958.246 mm ² /m		
As per Clause 16.3.1 of IRC:112-2011				
Minimum Reinforcement	=	$0.12/100 \times b \times D$	=	360 mm ² /m
Maximum ($A_{st_{req}}$, $A_{st_{min}}$)	=	958.246 mm ² /m		

Provide 16 mm dia bar @ 200 mm c/c as vertical steel at earth face.**Provide A_{st} = 1005 mm²/m) OK**

Percentage of Steel Provided	=	0.415 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , X_m	=	$\frac{0.0035 \cdot d}{(0.0035 + f_{yd} / E_s)}$	=	$\frac{0.0035 \times 242}{0.0035 + 0.00217}$
	=	149.2797 mm		

Depth of Neutral Axis ,	=	$\frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b}$		
	=	$\frac{435 \times 1005}{0.36 \times 35.00 \times 1000}$	=	34.707 mm OK

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

z	=	$d - 0.416 \cdot X$	=	242 - 14.438
	=	227.562 mm		

Moment of Resistance of Section w.r.t. Steel (MR)

MR	=	$f_{yd} \cdot A_{st} \cdot z$	=	434.78 x 1005 x 227.56
	=	9.9E+07 Nmm	=	99.465 kNm/m > 95.16 kNm/m
				SAFE

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

(b) Horizontal steel

Refer Clause 16.3.2 of IRC:112-2011

Adopting distribution steel bars Dia.	=	10 mm
Minimum Area of Steel	=	0.001x 0.5 x b x D OR 25% of Ast on Vertical Face
0.001x0.5xbxD	=	150 mm ² /m OR 239.561 mm ² /m
Governing Ast	=	239.561 mm ² /m
Maximum Spacing of Bars	=	300 mm

Provide	10 mm dia bar @	200 mm c/c horizontal steel at non earth face.

Provided Ast	=	393 mm ² /m)	OK

(c) Vertical steel on other face

As per Clause 16.3.1 of IRC:112-2011

Minimum Reinforcement	=	0.12/100 b x D	=	360 mm ² /m

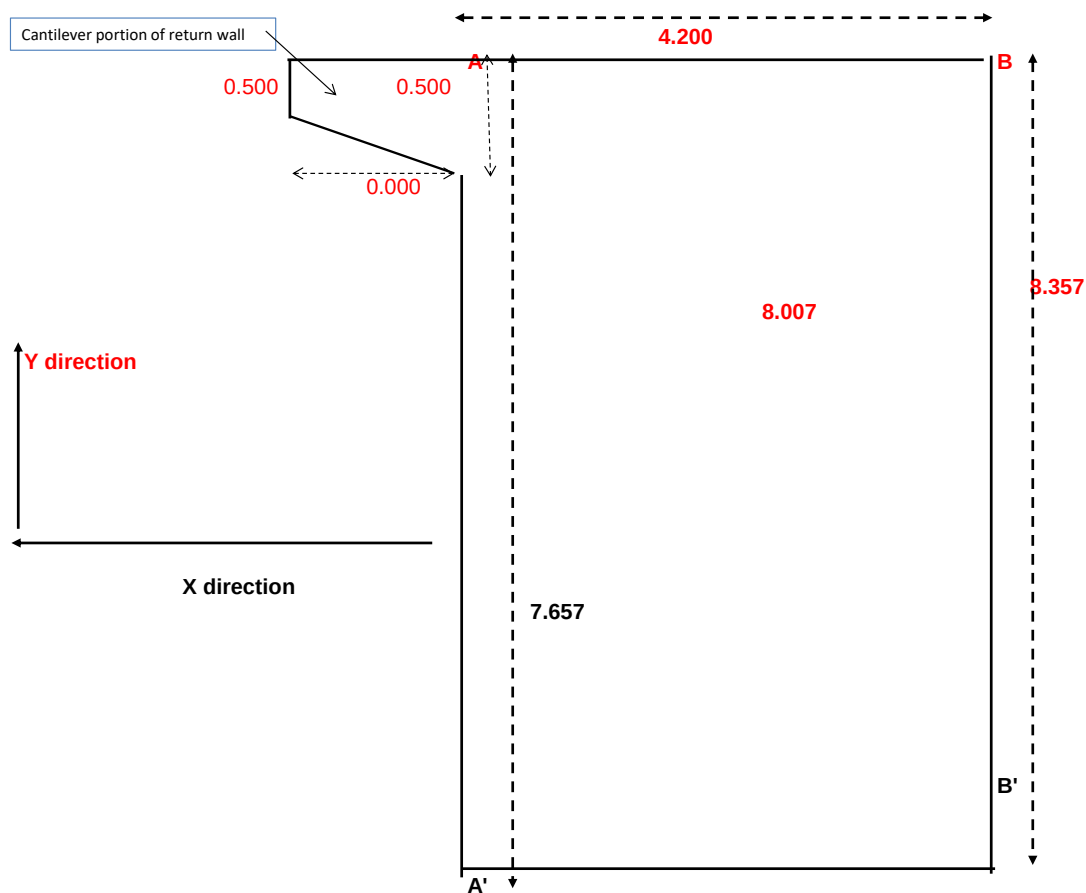
Provide	10 mm dia bar @	200 mm c/c as vertical steel at earth face.

Provided Ast=	393 mm ² /m)	OK

Design of Solid Return wall

THICKNESS OF SOLID RETURN WALL = 0.500 m

THICKNESS OF CANTILEVER RETURN WALL = 0.500 m



Width of Solid Return $a = 4.20$ m
 Avg. Height of Solid Return $b = 8.007$ m

a) Design of Solid Return wall

For design of return wall Load case 11.a & 11.d and their formulae given by Roark have been used.

Here,

a/b	=	0.525						
a/b	=	0.5	β_1	=	0.631	β_2	=	0.632
a/b	=	0.75	β_1	=	1.246	β_2	=	1.186

For uniformly distributed load over entire plate

For, $a/b = 0.525$ $\beta_1 = 0.691$ $\beta_2 = 0.686$

Live Load Surcharge Intensity:

$q = 0.2259 \times 20.00 \times 1.200 = 5.421$ kN/m²

$$\begin{aligned} \text{Max. } \sigma_b &= \frac{\beta_1 \times q \times b^2}{(t_1)^2} \\ \sigma_a &= \frac{\beta_2 \times q \times b^2}{(t_2)^2} \\ \sigma_b &= \frac{0.691 \times 5.421 \times 64.112}{0.250} \end{aligned}$$

At bottom edge = 961.114 kN/m² = 0.961 MPa

For 1000 mm of width, Z = $\frac{1000 \times 250000}{6} = 4.17E+07$ mm³

Hence Moment /m width along Y direction -

$$\begin{aligned}
 \text{My /m width} &= 0.961 \times 4.167\text{E}+07 \\
 &= 40046398 \text{ Nmm/m} = 40.046 \text{ kN.m/m} \\
 \sigma a &= \frac{0.686 \times 5.421 \times 64.112}{0.250} \\
 &= 954 = 0.9542 \text{ MPa} \\
 \text{For } 1000 \text{ mm of height, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence, Moment /m height along X direction -

$$\begin{aligned}
 \text{Mx /m height} &= 0.9542 \times 4.167\text{E}+07 = 3.976\text{E}+07 \text{ Nmm/m} \\
 &= 39.757 \text{ kN.m/m}
 \end{aligned}$$

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.500	β_1 =	0.328	β_2 =	0.200
a/b =	0.75	β_1 =	0.537	β_2 =	0.276

$$\begin{aligned}
 \text{For, } a/b &= 0.525 \quad \beta_1 = 0.349 \\
 &\quad \beta_2 = 0.207 \\
 q &= 0.226 \times 20.00 \times 8.01 \\
 &= 36.170 \text{ kN/m}^2 \\
 \text{Max. } \sigma b &= \frac{\beta_1 \times q \times b^2}{(t_1)^2} \\
 \sigma a &= \frac{\beta_2 \times q \times b^2}{(t_2)^2} \\
 \sigma b &= \frac{0.349 \times 36.170 \times 64.112}{0.25} \\
 &= 3232.77 \text{ kN/m}^2 \\
 &= 3.233 \text{ MPa}
 \end{aligned}$$

$$\begin{aligned}
 \text{For } 1000 \text{ mm of width, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence Moment /m width along Y direction -

$$\begin{aligned}
 \text{My /m width} &= 3.233 \times 4.167\text{E}+07 \\
 &= 134698891 \text{ Nmm/m} = 134.699 \text{ kN.m/m} \\
 \sigma a &= \frac{0.207 \times 36.170 \times 64.112}{0.25} \\
 &= 1924.4 \text{ kN/m}^2 = 1.924 \text{ MPa}
 \end{aligned}$$

$$\begin{aligned}
 \text{For } 1000 \text{ mm of height, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence Moment /m height along X direction -

$$\begin{aligned}
 \text{Mx /m height} &= 1.924 \times 4.167\text{E}+07 = 8.018\text{E}+07 \text{ Nmm/m} \\
 &= 80.182 \text{ kN.m/m}
 \end{aligned}$$

Total Moment in Solid Return Wall / m height = 119.939 kN.m/m

Total Moment in Solid Return Wall / m width = 174.745 kN.m/m

Final Design Moments:

Load Factor for Earth pressure	=	1.50
Load Factor for live load surcharge	=	1.20
Total Moment(Mx) in Solid Return Wall / m height	=	168 kN.m/m
Total Moment(My) in Solid Return Wall / m width	=	250 kN.m/m

Material Property:

- Refer Table No 6.5 of IRC : 112-2011

Grade of Concrete	=	M 35
Characteristic Strength of Concrete, fck	=	35.00 Mpa at 28 days
Grade of Reinforcement	=	Fe 500
Yield Strength of Reinforcement, fy or fyk	=	500.00 Mpa
Design Yield Strength of Reinforcement, fyd	=	434.78 Mpa (1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00 Mpa

1. Design of Face BB'

$$\begin{aligned} \text{Moment in Solid Return /m height (including cantilever moment) } &= \\ &= 167.982 + 0.00 \\ &= 167.98 \text{ kN.m / m} \end{aligned}$$

Adorting clear cover on either face	=	75 mm
Minimum Dia of Reinforcement	=	16 mm
Maximum Spacing of Steel	=	200 mm
Thickness of wall	=	0.500 m
Available effective depth	=	500 -75 -8
	=	417 mm

Check for Depth:

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 167.98 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \text{SQRT} \left(\frac{167.98 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$\text{Effective Depth of Cap Required (dreq)} = 170.551 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 253.55 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 500.00 \text{ mm}$$

OK

$$R = M_u / (b \times d^2) = 0.97$$

Area of Steel Required:

$$\frac{p_t}{100} = \frac{A_{st_{req}}}{b \times d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$$

$$A_{st_{req}} = 957.525 \text{ mm}^2/\text{m}$$

$$\begin{aligned} \text{Minimum Reinforcement} &= 0.12/100 \times b \times D \\ &= 600 \text{ mm}^2/\text{m} \end{aligned} \quad \text{As per Clause 16.3.1 of IRC:112-2011}$$

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 957.525 \text{ mm}^2/\text{m}$$

Provide 16 mm dia bar @ 200 mm c/c as Horizontal steel at earth face.

Provide Ast= 1005 mm²/m) OK

$$\text{Percentage of Steel Provided} = 0.241 \%$$

Check for Moment of Resistance of section due to steel

$$\text{Limiting Depth of Neutral Axis , } X_m = \frac{0.0035 \times d}{(0.0035 + f_{yd} / E_s)}$$

$$= \frac{0.0035 \times 417}{0.0035 + 0.00217}$$

$$= 257.230 \text{ mm}$$

$$\text{Depth of Neutral Axis , } X = \frac{f_{yd} \times A_{st}}{0.36 \times f_{ck} \times b}$$

$$= \frac{434.78 \times 1005}{0.36 \times 35.00 \times 1000}$$

$$= 34.690 \text{ mm} \quad \boxed{\text{OK}}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned} z &= d - 0.416 \cdot X \\ &= 417 - 14.431 \\ &= 402.569 \text{ mm} \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned} \text{MR} &= f_{yd} \cdot A_{st} \cdot z \\ &= 434.78 \times 1005 \times 402.569 \\ &= 1.76\text{E}+08 \text{ Nmm} \\ &= 175.959 \text{ kNm/m} > 167.98 \text{ kNm/m} \end{aligned}$$

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

Provide 12 mm dia bar @ 200 mm c/c as Horizontal steel at non earth face.

$$\text{Provided } A_{st} = 565 \text{ mm}^2/\text{m}$$

2. Design for Face A'B'

Moment in Solid Return /m width = 250.10 kN.m / m

$$\begin{aligned} \text{Adopting clear cover on either face} &= 75 \text{ mm} \\ \text{Minimum Dia of Reinforcement} &= 20 \text{ mm} \\ \text{Maximum Spacing of Steel} &= 150 \text{ mm} \\ \text{Thickness of wall} &= 0.500 \text{ m} \\ \text{Available effective depth} &= 500 \text{ mm} \quad -75 \quad -16 \quad -10 \\ &= 399 \text{ mm} \end{aligned}$$

Check for Depth:

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 250.10 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \text{SQRT} \left(\frac{250.10 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$\text{Effective Depth of Cap Required (dreq)} = 208.106 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 293.11 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 500.00 \text{ mm} \quad \boxed{\text{OK}}$$

$$R = M_u / (b \cdot d^2) = 1.57$$

Area of Steel Required:

$$\begin{aligned} \frac{p_t}{100} &= \frac{A_{st_{req}}}{b \cdot d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y} \\ &= 0.004 \\ A_{st_{req}} &= 1524.261 \text{ mm}^2/\text{m} \end{aligned}$$

$$\begin{aligned} \text{Minimum Reinforcement} &= 0.12/100 \cdot b \times D \\ &= 600 \text{ mm}^2/\text{m} \end{aligned} \quad \text{As per Clause 16.3.1 of IRC:112-2011}$$

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 1524.261 \text{ mm}^2/\text{m}$$

Provide 20 mm dia bar @ 150 mm c/c as vertical steel at earth face.

Provide A_{st} = 2094 mm²/m) OK

$$\text{Percentage of Steel Provided} = 0.5249 \%$$

Provide 12 mm dia bar @ 300 mm c/c as Vertical steel at non earth face.

Check for Moment of Resistance of section due to steel

$$\begin{aligned} \text{Limiting Depth of Neutral Axis , } X_m &= \frac{0.0035 \cdot d}{(0.0035 + f_{yd} / E_s)} \\ &= \frac{0.0035 \times 399}{0.0035 + 0.00217} \\ &= 246.13 \text{ mm} \end{aligned}$$

$$\begin{aligned}
 \text{Depth of Neutral Axis, } X &= \frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b} \\
 &= \frac{434.78}{0.36} \times \frac{2094}{35.00 \times 1000} \\
 &= 72.270 \text{ mm} \quad \boxed{\text{OK}}
 \end{aligned}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned}
 z &= d - 0.416 \cdot X \\
 &= 399 - 30.064 \\
 &= 368.94 \text{ mm}
 \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned}
 MR &= f_{yd} \cdot A_{st} \cdot z \\
 &= 434.78 \times 2094 \times 368.936 \\
 &= 3.36E+08 \text{ Nmm} \\
 &= 335.955 \text{ kNm/m} > 250.10 \text{ kNm/m}
 \end{aligned}$$

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

b) Cantilever Portion of Return Wall

$$\begin{aligned}
 \text{Self-weight of cantilever portion of return wall} &= 6 \text{ kN/m} \\
 \text{Crash Barrier weight} &= 10.0 \text{ kN/m} \\
 \text{Total Load} &= 16 \text{ kN/m} \\
 \text{Moment at Cantilever Face} &= 0 \text{ kNm} \\
 \text{Load Factor} &= 1.35 \\
 \text{Design Moment} &= 0 \text{ kNm} \\
 \text{Effective Depth} &= 444.000 \text{ mm}
 \end{aligned}$$

$$R = M_u / (b \cdot d^2) = 0.00$$

Area of Steel Required:

$$\begin{aligned}
 \frac{p_t}{100} &= \frac{A_{st_{req}}}{b \cdot d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y} \\
 &= \frac{0.000}{2} \\
 A_{st_{req}} &= 0.000 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 \text{Minimum Reinforcement} &= 0.12/100 \cdot b \times D \quad \text{As per Clause 16.3.1 of IRC:112-2011} \\
 &= 266.4 \text{ mm}^2
 \end{aligned}$$

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 266.4000 \text{ mm}^2$$

Provide	12	4	=	452 mm ²

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

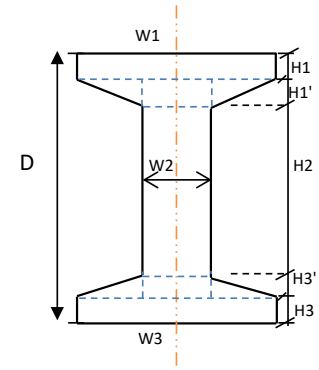
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1200
H1	=	150
H1'	=	100
H2	=	950
H3'	=	0
H3	=	0
W1	=	800
W2	=	350
W3	=	350

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	800	150	120000
1' (Tri.)	450	100	22500
1' (Rect.)	350	100	35000
2 (Rect.)	350	950	332500
3' (Tri.)	0	0	0
3' (Rect.)	350	0	0
3 (Rect.)	350	0	0
			510000



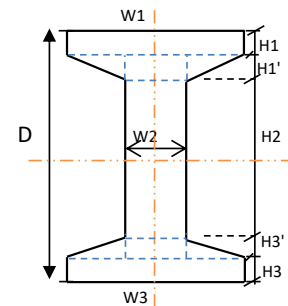
Total Area of Section A = 0.510 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1200
H1	=	150
H1'	=	67
H2	=	983
H3'	=	0
H3	=	0
W1	=	800
W2	=	500
W3	=	500

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	800	150	120000
1' (Tri.)	300	67	10000
1' (Rect.)	500	67	33333
2 (Rect.)	500	983	491667
3' (Tri.)	0	0	0
3' (Rect.)	500	0	0
3 (Rect.)	500	0	0
			655000



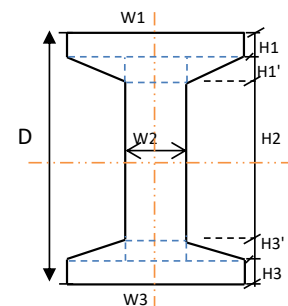
Total Area of Section A = 0.655 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

D	=	1200
H1	=	150
H1'	=	33
H2	=	1017
H3'	=	0.00
H3	=	0.00
W1	=	800
W2	=	650
W3	=	650

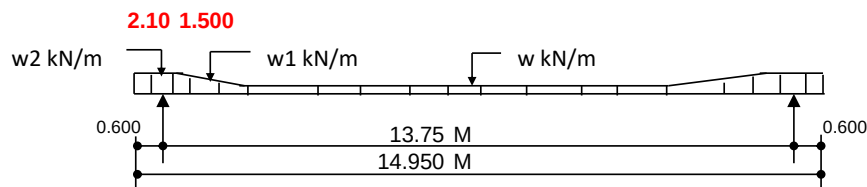
Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	800	150	120000
1' (Tri.)	150	33	2500
1' (Rect.)	650	33	21667
2 (Rect.)	650	1017	660833
3' (Tri.)	0	0	0
3' (Rect.)	650	0	0
3 (Rect.)	650	0	0
			805000



Total Area of Section A = 0.805 m²

1.0 Self weight of Inner Girder & Outer Girder

Length of the girder = 14.950 m



UDL , w = 12.7500 kN/m
 Avg. , w_1 = 16.3750 kN/m
 UDL , w_2 = 20.1250 kN/m
 Supp. Reaction , R = 116.23 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab = 2.850 m For Outer Girder
 Effective Width of Slab = 2.80 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/m³

UDL due to deck slab weight = 15.675 kN/m For Outer Girder
 on entire length of girder = 15.400 kN/m For Inner Girder

Supp. Reaction , R = 117.171 kN For Outer Girder
 = 115.115 kN For Inner Girder

weight of deck slab has been Increase by 5 %
 Supp. Reaction , R = 123.029 kN For Outer Girder
 = 120.871 kN For Inner Girder

3.0 Weight of end cross beam

Depth of cross beam = 1.17 m
 Thickness of cross beam = 0.4 m
 Support Reaction , R = 33.345 kN on outer bearings
 = 32.760 kN on Inner bearings

4.0 Weight due to wearing coat

Wearing coat width = 2.350 m For Outer Girder
 Wearing coat width = 2.800 m For Inner Girder
 Weight of wearing coat including future = 2.2 kN/m²

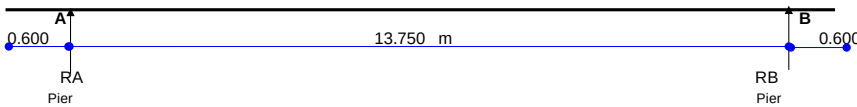
UDL due to Wearing coat = 5.170 kN/m For Outer Girder
 on entire length of girder = 6.160 kN/m For Inner Girder

Supp. Reaction , R = 38.646 kN For Outer Girder
 = 46.046 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 = **13.75 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 = **14.95 m**

Nos. of Lane for design purpose = **2 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

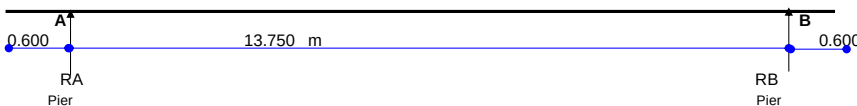
1st Train	Total Load	500.0kN				
					314.051	185.949
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

Reactions (without Impact) for 1 Lane

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	314.051	185.949

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

Length of the Span = **13.75 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 = **14.95 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

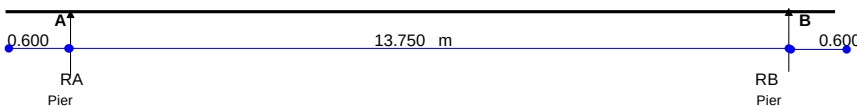
1st Train	Total Load	1000.0kN				
					670.996	329.004
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

Reactions (without Impact) for 70R Wheeled

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	670.996	329.004

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 = **13.75 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 = **14.95 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

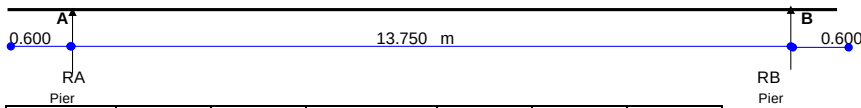
1st Train	Total Load	554.0kN				
					186.996	367.004
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
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	367.004	186.996

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

Length of the Span = 13.75 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 = 14.95 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	1801.964	1618.036
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		
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		
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		

Reactions for SV Loading

Description	At Supp-A RA	At Supp-B RB
Reaction in kN	1801.964	1618.036

Deck Width = 8.500 m

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

SV LOAD

transverse ecc.	=	0.3 m		
Maximum Reaction (RA)	=	1801.964 KN	Corr. MT	= 540.589 kNm
Minimum Reaction (RB)	=	1618.036 KN	Corr. MT	= 485.411 kNm

1-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-0.9	0	0	=	2.45 m
Maximum Reaction (RA)	=	314.051 KN		Corr. MT	=	769.425 kNm			
Minimum Reaction (RB)	=	185.949 KN		Corr. MT	=	455.575 kNm			

2-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-0.85	0	=	0.70 m
Maximum Reaction (RA)	=	628.102 KN		Corr. MT	=	439.671 kNm			
Minimum Reaction (RB)	=	371.898 KN		Corr. MT	=	260.329 kNm			

3-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	-1.05 m
Maximum Reaction (RA)	=	942.153 KN	Corr. MT	=	-989.260 kNm				
Minimum Reaction (RB)	=	557.847 KN	Corr. MT	=	-585.740 kNm				

4-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-2.8 m
Maximum Reaction (RA)	=	1256.204 KN	Corr. MT	=		-3517.370 kNm				
Minimum Reaction (RB)	=	743.796 KN	Corr. MT	=		-2082.630 kNm				

1- 70RW

Transverse Ecc.	for 70RW	=	4.25	-0.5	-1.2	-1.395	=	1.155 m
Maximum Reaction (RA)	=	670.996 KN	Corr. MT	=	775			
Minimum Reaction (RB)	=	329.004 KN	Corr. MT	=	380			

1-CLASS A + 1-70RW

CLASS A - 70RW		SEAL										
Transverse Ecc.	for Class A	=	4.25	-0.5	-0.4	-0.9	=	2.45 m				
	for 70RW	=	4.25	-0.5	-0.4	-1.8	-0.25	-1.2	-1.395	=	-1.295	
Maximum Reaction (RA)	=	1038.000 KN	Corr. MT	=	367.00 x	2.45 +	671.00 x	-1.295	=	30		
Minimum Reaction (RB)	=	516.000 KN	Corr. MT	=	186.996	2.45 +	329.004	-1.295	=	32		

2-CLASS A + 1-70RW

FORCES AT JOINT												
Transverse Ecc.	=	for 1st Class A	=	2.45 m			0	=	0.7 m			
	=	for 2nd Class A	=	2.45	-0.90	-0.85	-1.8	-0.25				
	=	for 70RW	=	0.7	-0.85				-1.2	-1.395	=	-4.795
Maximum Reaction (RA)	=	1405.004 KN		Corr. MT	=	367.00 x	2.45 +	367.00 x	0.700			-2061
							+	671.00 x	-4.795			
Minimum Reaction (RB)	=	702.996 KN		Corr. MT	=	186.996	2.45 +	187.00 x	0.700			-989
							+	329.00 x	-4.795			

1- 70RW + 1-CLASS A

1-70RW + 1-CLASS A		3 LANE								
Transverse Ecc.	for 70RW	=	4.25	-0.5	-1.2	-1.395	=	1.155 m		
	for Class A	=	4.25	-0.5	-7.25	-0.25	-0.9 =		-4.650 m	
Maximum Reaction (RA)	=	1038.000 KN	Corr. MT	=	671.00 x	1.155 +	367.00 x	-4.650	=	-932
Minimum Reaction (RB)	=	516.000 KN	Corr. MT	=	329.004	1.155 +	186.996	-4.650	=	-490

1- 70RW + 2-CLASS A

for 70RW		=	1.155 m							
for 1st Class		=	-4.650 m							
for 2nd Class		=	-4.650	-0.9	-0.85	=	-6.400 m			
Maximum Reaction (RA)	=	1405.004 KN	Corr. MT	=	671.00 x	1.155 +	367.00 x	-4.650	=	-3280
					+		367.00 x	-6.400		
Minimum Reaction (RB)	=	702.996 KN	Corr. MT	=	329.004	1.155 +	186.996	-4.650	=	-1686
					+		187.00 x	-6.400		

2- 70RW

Transverse Ecc.	=	1.16 m							
for 1-70RW	=	1.155	-1.395	-1.2	-1.40				
for 2-70RW	=								
Maximum Reaction (RA)	=	1341.993 KN	Corr. MT	=	671.00 x	1.155 +	671.00 x	-2.835	= -1127
Minimum Reaction (RB)	=	658.007 KN	Corr. MT	=	329.004	1.155 +	329.004	-2.835	= -553

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	314.051	769.425
2-CLASS A	628.102	439.671
3-CLASS A	0.000	0.000
4-CLASS A	0.000	0.000
1- 70RW	670.996	775.001
1-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 1-CLASS A	0.000	0.000
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)
185.949	455.575
371.898	260.329
0.000	0.000
0.000	0.000
329.004	379.999
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 670.996 KN
 Reaction RB = 329.004 KN

& Corresponding moment = 775.001 kNm
 & Corresponding moment = 379.999 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 670.996 KN
 Reaction RB = 329.004 KN

& Max moment = 775.001 kNm
 & Max moment = 379.999 kNm

Design of Precast RCC Girder and Deck
Span : 13.75m c/c of bearing (15.0m c/c of expansion
Gap)

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:6-2017 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: 6-2017
Density of Wearing Coat		22 KN/m ³	

SUPERSTRUCTURE and SPAN DETAILS

Type of Superstructure	-	RCC Precast Girder	pin / metallic guided	
Type of Bearing	-	Elastomeric with Pin / Metallic guided		
No of Spans	-	1		
Skew Angle	-	0 degree	contemporary angle	90
Span effective from E/J to E/J (SK)	-	15.000 m		
Span effective from Brg. to Brg. (SK)	-	13.750 m		
Span effective from Brg. to Brg. (SQ)	-	13.750 m		
Spacing of girders	-	2.800 m	number of girder	3
Slab overhang from girder centre	-	1.450 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	-	8.5 m	footpath	-	1.5
Thickness of Wearing Coat	-	0.065 m	crash barrier	-	0.5
Expansion Gap in Abutment	-	0.05 m			
Clear Cover Provided	-	40 mm			

SEISMIC DESIGN

Seismic Zone	-	v
Seismic Zone factor	-	0.36
Importance factor	-	1.2

CARRIAGEWAY LIVE LOAD (CWLL)

Superstructure meant for 2-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 (as mentioned below) so placed on the span to produce maximum stresses.

Impact Factor	=	1.228
Lane Reduction Factor with congestion factor	=	1.150

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 of class 70R Most Eccentric
- 2 Two Lane of Class A Placed Most Eccentrically
- 5 SV Loading

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 evaluate the effect on structure due to daily changes in temperature.

Coefficient of thermal expansion as per IRC: 6-2017	=	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

$$f_y = 500 \text{ N/mm}^2$$

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	=	50	days
open to traffic	assumed	=	78 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 $f_{cm} = f_{ck} + 10$ & $\beta_{cc}(t) = \exp \left\{ s \left[1 - \left(\frac{28}{t/t_1} \right)^{1/2} \right] \right\}$ for
 ordinary portland concrete $S = 0.25$
 Characteristic compressive strength with time, $f_{ck}(t) = f_{cm}(t) - 10$
 $f_{ck} = 35 \text{ Mpa}$ $f_{cm} = 45 \text{ Mpa}$

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

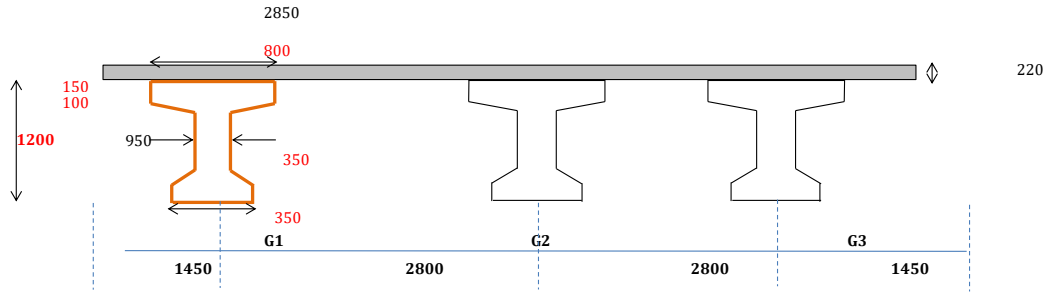
Superstructure has been modeled as an equivalent grillage mesh consisting of longitudinal and transverse members. Analysis of Self weight of girder and green concrete of slab has been carried out by single line model and the analysis of SIDL and CWLL has been carried out by grillage model. STAAD-Pro 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	1200 mm	1420 mm
Area	510000 mm ²	1137000 mm ²
I_z	6.91E+10 mm ⁴	1.80E+11 mm ⁴
Y_b	687.87 mm	1030.9 mm
Y_t	512.13 mm	389.1 mm
Z_b	1.005E+08 mm ³	1.751E+08 mm ³
Z_t	1.350E+08 mm ³	4.639E+08 mm ³
u	3.550E+00 m	8.890E+00 m

Deck slab 220
Eff. Width 2850

Moment of Inertia - I_y

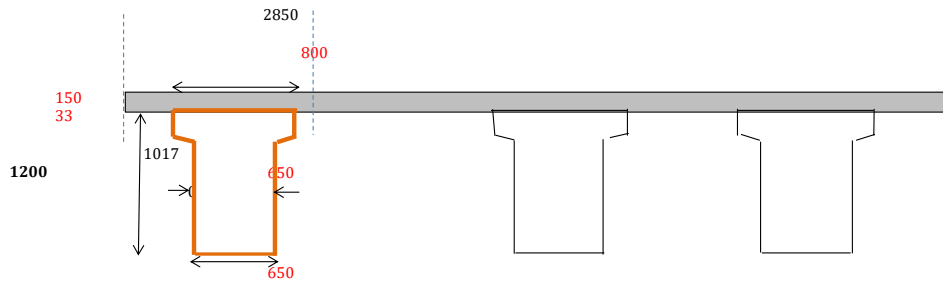
Isself = $bd^3/12$	1.02E+10 mm ⁴
A_z^2	1.41E+09 mm ⁴
I_y	1.16E+10 mm ⁴
Z_{Ly}	1436.2 mm
$I_y(\text{slab})$	4.24E+11 mm ⁴
$I_y(\text{comp})$	4.36E+11 mm ⁴

Moment of Inertia - I_x

T_{av} 200.0 mm
 B_{av} 0.0 mm

Portion no.	b	d	$\frac{3b^3d^3}{10(b^2+d^2)}$
Slab	2850 mm	220 mm	5.06E+09
Girder portion	800 mm	200 mm	1.81E+09
	350 mm	1000 mm	1.15E+10
	350 mm	0 mm	0.00E+00
$I_x \text{ composite(mm}^4\text{)}$			1.83E+10
$I_x \text{ Girder(mm}^4\text{)}$			1.33E+10

AT SUPPORT

Moment of Inertia - I_z

	Self	Composite
Depth	1200 mm	1420 mm
Area	804975 mm ²	1431975 mm ²
I_z	1.0E+11 mm ⁴	2.7E+11 mm ⁴
Y_b	616.52 mm	920.2 mm
Y_t	583.48 mm	499.8 mm
Z_b	1.6E+08 mm ³	3.0E+08 mm ³
Z_t	1.7E+08 mm ³	5.5E+08 mm ³
U		8.4E+00 m

Moment of Inertia - I_y

$I_{self} = bd^3/12$	3.043E+10	mm ⁴
A_z^2	3.032E+08	mm ⁴
I_y	3.073E+10	mm ⁴
Z_L	1425.0	mm
$I_y(\text{slab})$	4.244E+11	mm ⁴
$I_y(\text{comp})$	4.55134E+11	mm ⁴

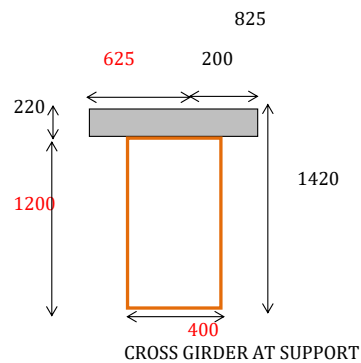
Moment of Inertia - I_x

T_{av}	166.5	mm
B_{av}	0.0	mm

Portion no.	b	d	$\frac{3b^3d^3}{10(b^2+d^2)}$
	mm	mm	
1	2850	220	5.06E+09
2	800	167	1.06E+09
3	650	1034	6.10E+10
4	650	0	0.00E+00
$I_{x \text{ composite}}$		=	6.71E+10
$I_{x \text{ girder}}$		=	6.21E+10

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

Area (A_x)	661500	mm ²
y _b	795	mm
y _t	625	mm
$I_{self} = bd^3/12$	5.83E+10	mm ⁴
A_z^2	6.64E+10	mm ⁴
I_z	1.247E+11	mm ⁴

**Moment of Inertia - I_y**

Area (A_x)	661500	mm ²
y _i	567	mm
y _r	258	mm
$I_{self} = bd^3/12$	1.67E+10	mm ⁴
A_z^2	5.95E+09	mm ⁴
$I_y =$	2.264E+10	mm ⁴

Moment of Inertia - I_x

Item	b	d	$\frac{3b^3d^3}{10*(b^2+d^2)}$
	mm	mm	
1	825	220	1.464E+09
2	400	1200	2.074E+10

$I_{x, \text{girder}}$	2.074E+10	mm ⁴
$I_{x, \text{composite}}$	2.220E+10	mm ⁴

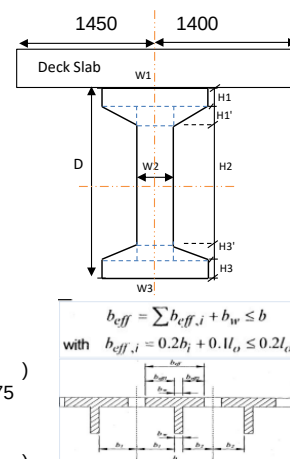
Effective Width of FlangeFOR **OUTER GIRDER**

Thickness of Deck Slab = 220 mm

bw	=	b	=	2.850 m
		w2	=	350 mm
		b1	=	1275 mm
		b2	=	1225 mm
		lo	=	13.75 m

effective width of cantilever on one end = $\min (0.2 \times b1 + 0.1 \times lo, 0.2 \times lo, b1)$
 $\min (1.63, 2.75, 1.275) = 1.275$
 beff, 1 = 1.275

effective width of cantilever on other end = $\min (0.2 \times b2 + 0.1 \times lo, 0.2 \times lo, b2)$
 $\min (1.62, 2.75, 1.225) = 1.225$
 beff, 2 = 1.225

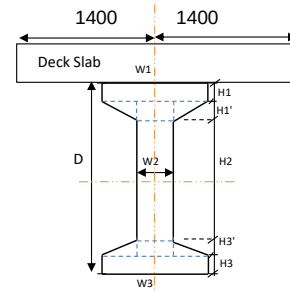


Total effective width of flange	beff	=	1.275	+	1.225	+	0.350
		=	2.850 m				

FOR **INNER GIRDER**

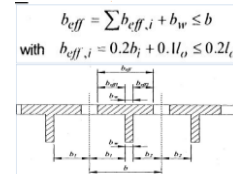
Thickness of Deck Slab	=	220 mm

bw	=	b	=	2.800 m
		w2	=	325.0 mm
		b1	=	1237.5 mm
		b2	=	1237.5 mm
		lo	=	13.75 m



effective width of cantilever on one end = min (0.2 x b1 + 0.1 x lo , 0.2 x lo, b1)	
min (1.6225 , 2.75 , 1.2375)	beff, 1 = 1.2375

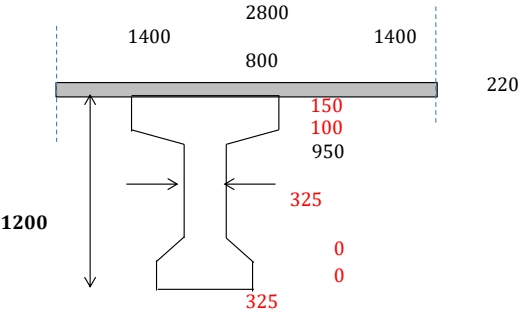
effective width of cantilever on other end = min (0.2 x b2 + 0.1 x lo , 0.2 x lo, b2)	
min (1.6225 , 2.75 , 1.2375)	beff, 2 = 1.238



Total effective width of flange	beff	=	1.238	+	1.2375	+	0.325
		=	2.800 m				

SECTION PROPERTIES (INNER GIRDER)

AT MID SPAN



Moment of Inertia - I_z

Parameter	Self	Composite
Depth	1200 mm	1420 mm
Area	485000 mm ²	1101000 mm ²
I_z	6.61E+10 mm ⁴	1.70E+11 mm ⁴
Y_b	697.53 mm	1040.2 mm
Y_t	502.47 mm	379.8 mm
Z_b	9.5E+07 mm ³	1.6E+08 mm ³
Z_t	1.3E+08 mm ³	4.5E+08 mm ³
u	3.5E+00 m	8.8E+00 m

Deck slab 220 mm
Eff. Width 2800 mm wide.

Moment of Inertia - I_y

$I_{self} = bd^3/12$	9.48E+09 mm ⁴
A_z^2	1.39E+09 mm ⁴
I_y	1.09E+10 mm ⁴
Z_i	1400.0 mm
$I_y(\text{slab})$	4.02E+11 mm ⁴
$I_y(\text{comp})$	4.13E+11 mm ⁴

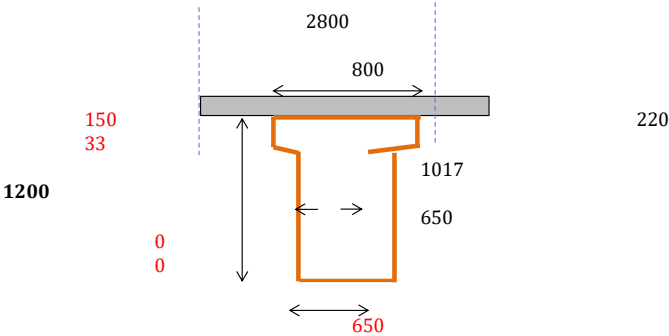
Moment of Inertia - I_x

T_{av}	200.0 mm
B_{av}	0.0 mm

Portion no.	b	d	$\frac{3b^3d^3}{10(b^2+d^2)}$
	mm	mm	
1	2800	220	4.97E+09
2	800	200	1.81E+09
3	325	1000	9.31E+09
4	325	0	0.00E+00

$I_x \text{ composite} = 1.61E+10 \text{ mm}^4$
 $I_x \text{ girder} = 1.11E+10 \text{ mm}^4$

AT SUPPORT



Moment of Inertia - I_z

Parameter	Self	Composite	
Depth	1200	1420	mm
Area	804975	1420975	mm ²
I_z	1.0E+11	2.7E+11	mm ⁴
Y_b	616.52	917.1	mm
Y_t	583.48	502.9	mm
Z_b	1.6E+08	3.0E+08	mm ³
Z_t	1.7E+08	5.4E+08	mm ³
u		7.9E+00	m

Moment of Inertia - I_y

$I_{self} = bd^3/12$	3.043E+10	mm ⁴
A_z^2	3.032E+08	mm ⁴
I_y	3.073E+10	mm ⁴
Z_b	1400.0	mm
$I_y(\text{slab})$	4.025E+11	mm ⁴
$I_y(\text{comp})$	4.33E+11	mm ⁴

Moment of Inertia - I_x

T_{av}	=	166.5	mm
B_{av}	=	0.0	mm

Portion no.	b	d	$\frac{3b^3d^3}{10(b^2+d^2)}$
	mm	mm	
1	2800	220	4.97E+09
2	800	167	1.06E+09
3	650	1034	6.10E+10
4	650	0	0.00E+00

$I_{x, \text{composite}}$		6.70E+10	mm ⁴
$I_{x, \text{girder}}$	=	6.21E+10	mm ⁴

GRILLAGE IDEALISATION

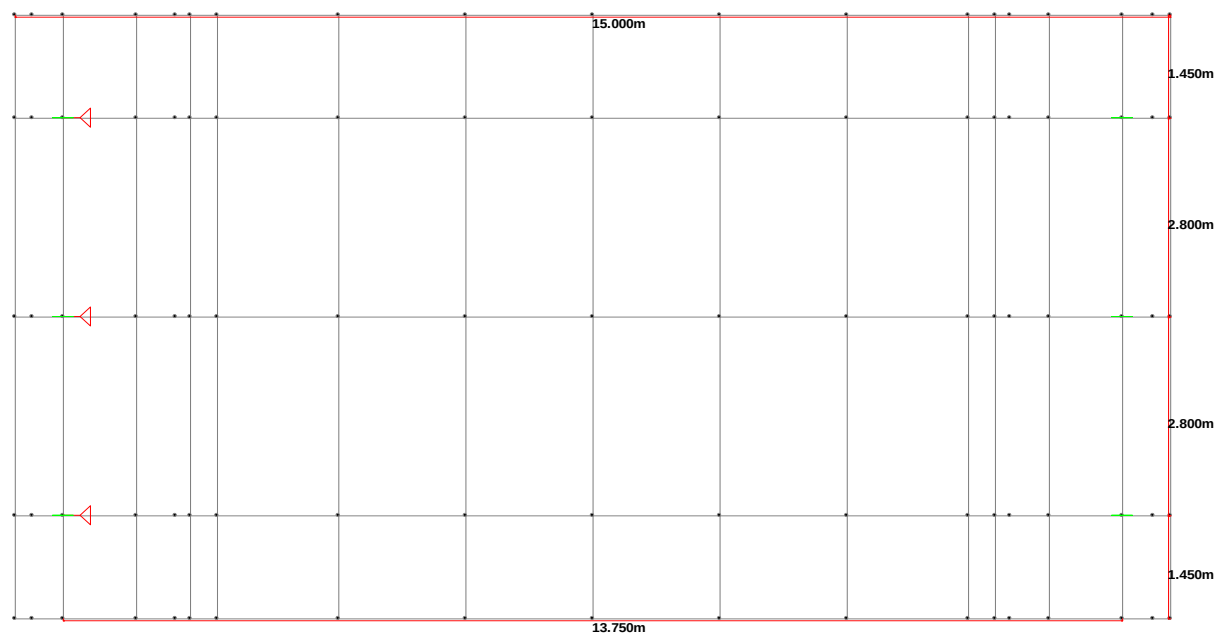


Fig. Showing the Grid Model of Deck Slab with Longitudinal and Cross Girder using Staad-Pro

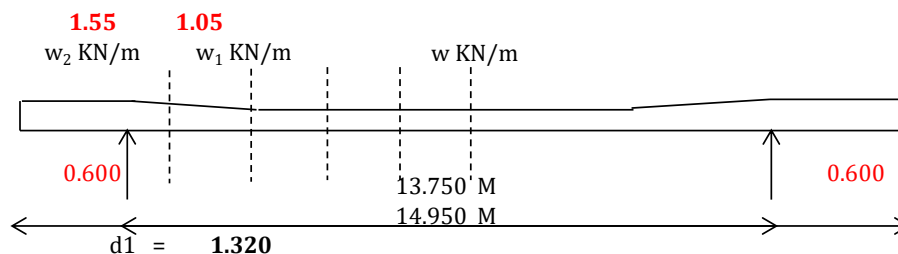
CALCULATION OF MOMENTS AND SHEARS (FOR OUTER GIRDERS)

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 = **14.950 m**



Distance Between Support, L = **13.75 m**
Overhang = **0.600 m**

UDL, w = 12.75 KN/m
Avg. , w_1 = 16.44 KN/m
UDL, w_2 = 20.12 KN/m
Supp. Reaction, R = **110.61 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	306.73	0.00
2	2	0.375 L	287.90	21.91
3	3	0.25 L	231.40	43.83
4	4	0.125 L	137.10	66.78
5	At D_{eff}	0.096 L	109.16	73.33
6	At web widening	0.95	80.90	79.42
7	At support	0	-3.622	98.53

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

UDL, w = 15.675 KN/m

Avg. , w_1 = 15.675 KN/m

UDL, w_2 = 15.675 KN/m

Supp. Reaction, R = **117.171 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	367.62	0.00
2	2	0.375 L	344.47	26.94
3	3	0.25 L	275.01	53.88
4	4	0.125 L	159.25	80.82
5	At D_{eff}	0.096 L	125.77	87.07
6	At web widening	0.95	92.48	92.87
7	At support	0.0	-2.82	107.77

3. 'Additional weight due to green concrete

$$\begin{aligned} \text{Weight of green concrete} &= 26 \text{ kN/m}^3 \\ \text{Additional weight due to green concrete} &= 26 \cdot 25 = 1 \text{ kN/m}^3 \\ &= 0.627 \text{ kN/m of girder} \end{aligned}$$

$$\text{UDL, } w = 0.627 \text{ KN/m (On entire length of girder)}$$

$$\text{Avg. , } w_1 = 0.627 \text{ KN/m}$$

$$\text{UDL, } w_2 = 0.627 \text{ KN/m}$$

$$\text{Supp. Reaction , } R = 4.687 \text{ KN}$$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	14.70	0.00
2	2	0.375 L	13.78	1.08
3	3	0.25 L	11.00	2.16
4	4	0.125 L	6.37	3.23
5	At D _{eff}	0.096 L	5.03	3.48
6	At web widening	0.95	3.70	3.71
7	At support	0.0	-0.11	4.31

4. 'Shuttering Load

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

$$= 3.6 \text{ kN/m}^2 = 10.26 \text{ kN/m}$$

$$\text{UDL, } w = 10.260 \text{ KN/m (On entire length of girder)}$$

$$\text{Avg. , } w_1 = 10.260$$

$$\text{UDL, } w_2 = 10.260 \text{ KN/m}$$

$$\text{Supp. Reaction , } R = 76.694 \text{ t}$$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	240.63	0.00
2	2	0.375 L	225.47	17.63
3	3	0.25 L	180.01	35.27
4	4	0.125 L	104.23	52.90
5	At D _{eff}	0.096 L	82.32	56.99
6	At web widening	0.95	60.53	60.79
7	At support	0	-1.85	70.54

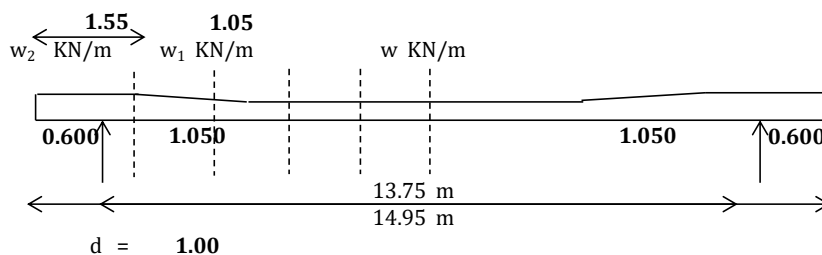
CALCULATION OF MOMENTS AND SHEARS(FOR INTERMEDIATE GIRDERS)

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 = **14.950 m**



Distance Between Support, L : **13.75 m**
Overhang **0.600 m**

UDL, w = 12.13 KN/m
Avg., w_1 = 16.12 KN/m
UDL, w_2 = 20.12 KN/m
Supp. Reaction, R = **107.23 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	292.73	0.00
2	2	0.375 L	274.82	20.84
3	3	0.25 L	221.09	41.68
4	4	0.125 L	131.39	63.64
5	At D_{eff}	0.096 L	104.73	70.07
6	At web widening	0.95	77.70	76.04
7	At support	0	-3.62	98.53

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

UDL, w = 15.400 KN/m (On entire length of girder)

Avg., w_1 = 15.400 KN/m

UDL, w_2 = 15.400 KN/m

Supp. Reaction, R = **115.115 KN**

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	361.17	0.00
2	2	0.375 L	338.43	26.47
3	3	0.25 L	270.19	52.94
4	4	0.125 L	156.45	79.41
5	At D_{eff}	0.096 L	123.57	85.55
6	At web widening	0.95	90.86	91.25
7	At support	0.0	-2.8	107.8

3. 'Additional weight due to green concrete

weight of green concrete = 26 kN/m^3
Additional weight due to green concrete = $26 - 25 = 1 \text{ kN/m}^3$
 = 0.616 kN/m of girder
 UDL, $w = 0.616 \text{ KN/m}$ (On entire length of girder)
 Avg. , $w_1 = 0.616 \text{ KN/m}$
 UDL, $w_2 = 0.616 \text{ KN/m}$
 Supp. Reaction , $R = 4.605 \text{ KN}$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	14.45	0.00
2	2	0.375 L	13.54	1.06
3	3	0.25 L	10.81	2.12
4	4	0.125 L	6.26	3.18
5	At D_{eff}	0.096 L	4.94	3.42
6	At web widening	0.95	3.63	3.65
7	At support	0	-0.113	4.311

4.'Shuttering Laod

As per clause 4.2.2.2.2 of IRC:87-1984 superimposed loads
 = 3.6 kN/m^2 = 10.08 kN/m
 UDL, $w = 10.080 \text{ KN/m}$ (On entire length of girder)
 Avg. , $w_1 = 10.080 \text{ KN/m}$
 UDL, $w_2 = 10.080 \text{ KN/m}$
 Supp. Reaction , $R = 75.348 \text{ KN}$

S.No	Section	Distance	Moment	Shear
		m	KN-m	KN
1	Mid Span	0.5 L	236.40	0.00
2	2	0.375 L	221.52	17.33
3	3	0.25 L	176.85	34.65
4	4	0.125 L	102.41	51.98
5	At D_{eff}	0.112 L	92.95	53.78
6	At web widening	0.95	59.47	59.72
7	At support	0	-1.85	70.54

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	KN-m
L/2	6.875	0.00	306.73	0.00	367.62	0.00	14.70	0.00	240.63	8.83	197.41	4.34	144.61	0.00	0.00	0.00	240.63	63.03	930.14	3.88	202.18	76.20	1946.51
3L/8	5.156	21.91	287.90	26.94	344.47	1.08	13.78	17.63	225.47	26.17	183.88	15.69	135.29	0.00	0.00	17.63	225.47	110.32	862.48	11.10	196.01	201.05	1814.01
L/4	3.438	43.83	231.40	53.88	275.01	2.16	11.00	35.27	180.01	40.57	141.68	26.21	107.17	0.00	0.00	35.27	180.01	174.06	697.41	17.67	172.52	338.55	1452.67
L/8	1.719	66.78	137.10	80.82	159.25	3.23	6.37	52.90	104.23	51.74	73.25	35.70	60.68	0.00	0.00	52.90	104.23	202.78	395.05	24.84	127.82	437.82	825.32
At D _{eff}	1.320	73.33	109.16	87.07	125.77	3.48	5.03	56.99	82.32	51.74	63.32	35.77	53.82	0.00	0.00	56.99	82.32	202.78	347.86	24.84	119.88	450.70	699.93
At Web widening	0.950	79.42	80.90	92.87	92.48	3.71	3.70	60.79	60.53	58.476	38.841	40.904	35.67	0.00	0.00	60.79	60.53	290.90	223.95	27.96	107.35	562.57	471.84
At support	0.000	98.53	-3.62	107.77	-2.82	4.31	-0.11	70.54	-1.85	58.476	0	41.807	0.00	0.00	0.00	70.54	-1.85	290.90	0.00	27.96	72.99	597.49	66.54

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	6.875	4.716	0.524	0	12.206	7.608
3L/8	5.156	15.125	1.7	0	18.244	22.138
L/4	3.438	27.579	3.115	0	28.728	34.789
L/8	1.719	44.023	4.921	0	38.684	47.096
At D _{eff}	1.320	44.023	4.921	0	37.997	47.096
At Web widening	0.950	54.547	6.089	0	68.575	52.551
At support	0.000	54.547	6.089	0	68.575	52.551

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		
		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	KN-m
L/2	6.875	0.00	292.73	0.00	361.17	0.00	14.45	0.00	236.40	2.23	24.45	3.25	123.04	0.00	0.00	0.00	236.40	80.95	1048.95	59.00	1285.45	108.66	2086.8
3L/8	5.156	20.84	274.82	26.47	338.43	1.06	13.54	17.33	221.52	6.38	21.81	12.52	115.42	0.00	0.00	17.33	221.52	134.55	970.53	163.81	1198.13	245.44	1948.6
L/4	3.438	41.68	221.09	52.94	270.19	2.12	10.81	34.65	176.85	8.97	12.97	21.48	92.43	0.00	0.00	34.65	176.85	213.18	779.74	258.23	942.89	387.29	1539.6
L/8	1.719	63.64	131.39	79.41	156.45	3.18	6.26	51.98	102.41	9.34	0.84	30.06	53.66	0.00	0.00	51.98	102.41	245.86	417.83	355.66	514.67	529.60	857.0
At D _{eff}	1.320	70.07	104.73	85.55	123.57	3.42	4.94	53.78	92.95	9.34	2.00	30.13	47.84	0.00	0.00	53.78	92.95	245.86	359.54	355.66	450.51	543.41	728.6
At Web widening	0.950	76.04	77.70	91.25	90.86	3.65	3.63	59.72	59.47	9.476	6.75	34.756	32.4	0.00	0.00	59.72	59.47	365.17	213.06	418.31	290.42	627.10	498.1
At support	0.000	98.53	-3.62	107.77	-2.82	4.31	-0.11	70.54	-1.85	9.476	14.22	35.66	0	0.00	0.00	70.54	-1.85	365.17	0.00	418.31	0.00	674.48	7.8

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	6.875	0.081	0.037	0	0.585	7.984
3L/8	5.156	0.210	0.114	0	2.097	24.925
L/4	3.438	0.557	0.159	0	3.728	49.371
L/8	1.719	1.928	0.110	0	6.25	86.224
At D _{eff}	1.320	1.928	0.084	0	6.25	86.224
At Web widening	0.950	2.632	0.084	0	7.329	107.08
At support	0.000	2.632	0.040	0	7.329	107.08



ANALYSIS OF LONG GIRDER FOR TEMPERATURE

Effective Bridge Temperature

INPUT DATA

Location	-				
Maximum Air shade Temperature	-	50 °C			
Minimum Air shade Temperature	-	-2.2 °C			
Mean of Max and Min Temp	-	23.9 °C			
Bridge Temperature assumed	-	33.9 °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	As per Clause 6.4.2.5 Pt.4 of IRC 112:2011
Coefficient of Thermal expansion	α	=	1.20E-05		

Sectional Properties of Composite Section

Area of Composite Section	A	=	1.101 m ²
MOI of Composite Section	I_z	=	0.170 m ⁴

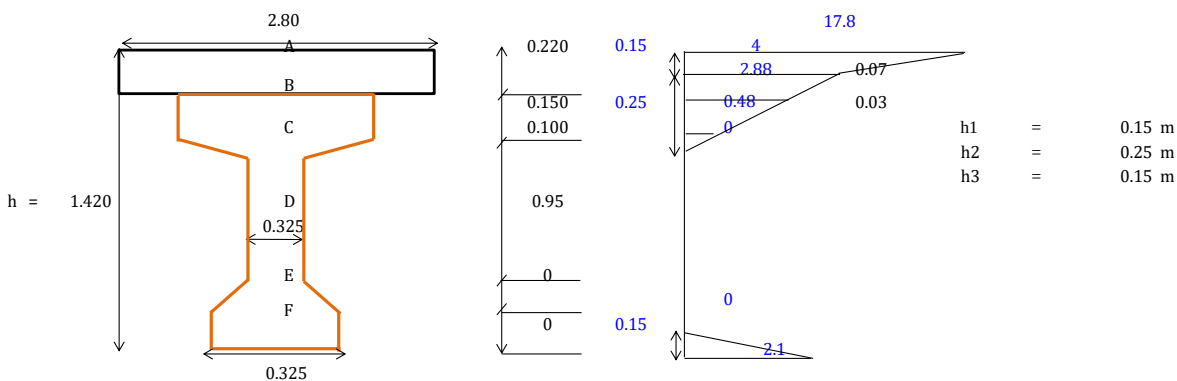
CG of the Section

From top of Slab	y_{ts}	=	0.38 m
From top of Girder	y_{tg}	=	0.16 m
From bottom of Girder	y_{bg}	=	1.04 m

Moduli of Section

	Z_{ts}	=	0.449 m ³
	Z_{tg}	=	1.066 m ³
	Z_{bg}	=	0.164 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	2.80	0.22	0.616	0.110	0.068	0.007	10.340	6.369	0.701
B	0.80	0.15	0.120	0.295	0.035	0.010	1.680	0.202	0.059
C	0.5625	0.1	0.056	0.403	0.023	0.009	0.240	0.014	0.005
D	0.325	0.95	0.309	0.945	0.292	0.276	0.000	0.000	0.000
E	0.325	0	0.000	1.420	0.000	0.000	0.000	0.000	0.000
F	0.325	0	0.000	1.345	0.000	0.000	1.050	0.000	0.000
SUM			1.101		0.418	0.303		6.585	0.766

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: 6-2017 Cl 215.4
α	=	1.20E-05

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

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

Eigen Stress

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

Above Equation Becomes

$$1.101 \varepsilon^o - 0.418 \theta = 7.9E-05 \quad \text{----- Eq.1}$$

$$0.418 \varepsilon^o - 0.303 \theta = 9.2E-06 \quad \text{----- Eq.2}$$

Multiplying eq. 1 by 0.418 & eq. 2 by 1.101 and subtract 1 and 2

$$\begin{array}{rclcl} 0.46 \varepsilon^o & - & 0.174 \theta & = & 3.3E-05 \\ 0.460 \varepsilon^o & - & 0.333 \theta & = & 1.0E-05 \\ \hline 0 & & 0.159 \theta & = & 2.3E-05 \end{array}$$

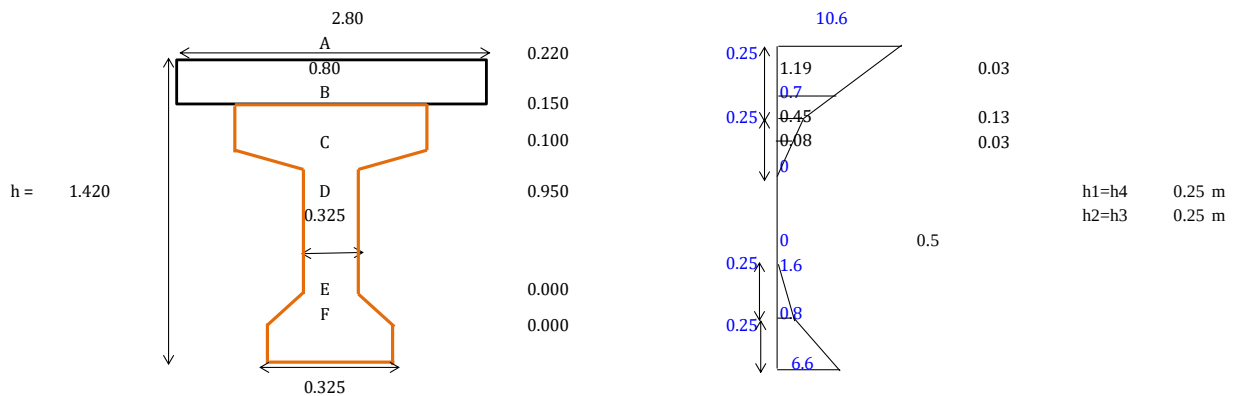
By solving the above two equations

$$\begin{array}{rcl} \text{Rotation of plane} & \theta & = 1.44E-04 \\ \text{Strain at Top Fibre} & \varepsilon^o & = 1.26E-04 \\ f_{ck} & & = 35 \text{ N/mm}^2 \\ E_c & & = 3.20E+04 \text{ N/mm}^2 \end{array}$$

S.No	y	y. θ	t	α. t	f _{ci}
1	0	0.00E+00	17.8	2.14E-04	-2.791
2	0.22	3.17E-05	2.88	3.46E-05	1.925
3	1.42	2.04E-04	2.1	2.52E-05	-0.003

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

Falling Gradient at Mid Span



Portion	B (m)	D	A	y from top	Ay	Ay ²	t	At	Aty
	m	m	m ²	m	m ³	m ⁴	°C	m ² °C	m ³ °C
A	2.80	0.22	0.616	0.110	0.068	0.007	5.894	3.631	0.399
B	0.800	0.15	0.120	0.295	0.035	0.010	0.574	0.069	0.020
C	0.5625	0.1	0.056	0.403	0.023	0.009	0.266	0.015	0.006
D	0.325	0.95	0.309	0.945	0.292	0.276	0.000	0.000	0.000
E	0.325	0	0.000	1.420	0.000	0.000	1.200	0.000	0.000
F	0.325	0	0.000	1.345	0.000	0.000	3.700	0.000	0.000
SUM			1.101		0.418	0.303		3.715	0.426

Equation 1 and 2 becomes

$$1.101 \epsilon^{\circ} - 0.418 \theta = 4.5E-05 \quad \text{Eq.1}$$

$$0.418 \epsilon^{\circ} - 0.303 \theta = 5.1E-06 \quad \text{Eq.2}$$

Multiplying eq. 1 by 0.418 & eq. 2 by 1.101 and subtract 1 and 2

$$\begin{array}{rclclcl} 0.46 \epsilon^{\circ} & - & 0.174 & \theta & = & 1.9E-05 \\ 0.460 \epsilon^{\circ} & - & 0.333 & \theta & = & 5.6E-06 \\ \hline 0 & & 0.159 & \theta & = & 1.3E-05 \end{array}$$

$$\text{Rotation of plane } \theta = 8E-05$$

$$\text{Strain at Top Fibre } \epsilon^{\circ} = 7E-05$$

S.No	y	y. θ	t	$\alpha. t$	f_{cl}
1	0	0.00E+00	10.6	1.27E-04	-1.455
1	0.22	1.80E-05	0.448	5.38E-06	1.540
2	1.42	1.16E-04	6.6	7.92E-05	-3.961

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

$$\text{Max permissible tensile stress due to temperature} = 2.80 \text{ N/mm}^2$$

Revise Temperature Calculation based on Cracked Section

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

$$F_{cs} = 304.515 \text{ KN}$$

$$M_{cs} = (\varepsilon_{diff})(E_{cf})A_{cf}(\delta_{cent})\alpha = 101.67 \text{ KN-m}$$

$$\begin{aligned} \text{Area of Composite Section } A &= 1.101 \text{ m}^2 \\ \text{MOI of Composite Section } I_z &= 0.170369 \text{ m}^4 \end{aligned}$$

CG of the Section

$$\begin{aligned} \text{From top of Slab } y_{ts} &= 0.380 \text{ m} \\ \text{From top of Girder } y_{tg} &= 0.160 \text{ m} \\ \text{From bottom of Girder } y_{bg} &= 1.040 \text{ m} \end{aligned}$$

Moduli of Section

$$\begin{aligned} Z_{ts} &= 0.44858 \text{ m}^3 \\ Z_{tg} &= 1.06615 \text{ m}^3 \\ Z_{bg} &= 0.16378 \text{ m}^3 \end{aligned}$$

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	-0.4943	0.2266	0.2766	0.0089
2	Top of Girder	0.0000	0.0954	0.2766	0.3719
3	Bottom of Girder	0.0000	-0.6208	0.2766	-0.3442

$$\alpha = \frac{1 - e^{-\phi}}{\phi} = 0.41$$

$$\phi = 2.147911$$

			Precast Girder	In-situ Slab	
Compressive strength	f_{ck}	=	35	35	N/mm ²
Cross sectional area of concrete	A_c	=	485000	616000	mm ²
Perimeter of the cross section	u	=	2941	2040	mm
Notional size of the cross section(h_o)		=	329.8	603.9	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

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

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

Time Dependent Shrinkage of Precast Girder

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

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

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

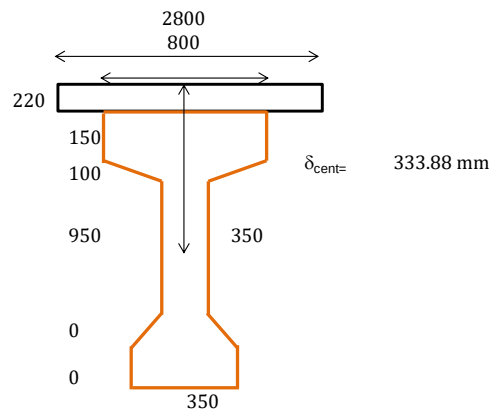
Drying Shrinkage with time

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd} = 1.93\text{E-}05$$

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

$$\text{Total Shrinkage at time} = 4.87\text{E-}05$$

$$\varepsilon_{diff} = \varepsilon_{slab, \infty} - (\varepsilon_{girder, \infty} - \varepsilon_{girder, t}) = 3.76\text{E-}05$$



DESIGN OF OUTER (OG1) GIRDER

ULS DESIGN OF LONGITUDINAL MEMBERS FOR TORSION

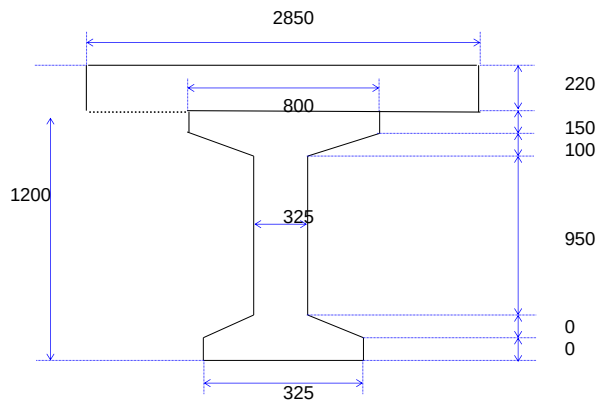
ULTIMATE BENDING MOMENT AND SHEAR FORCE

Bending Moment (kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	306.73	287.90	231.40	137.10	109.16	80.90
DL of Slab	1.35	367.62	344.47	275.01	159.25	125.77	92.48
SIDL	1.35	197.41	183.88	141.68	73.25	63.32	38.84
Surfacing	1.75	144.61	135.29	107.17	60.68	53.82	35.67
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	1142.07	1058.99	856.31	485.06	427.12	274.98
	TOTAL M_u	3143.05	2927.17	2346.94	1332.72	1137.50	761.39

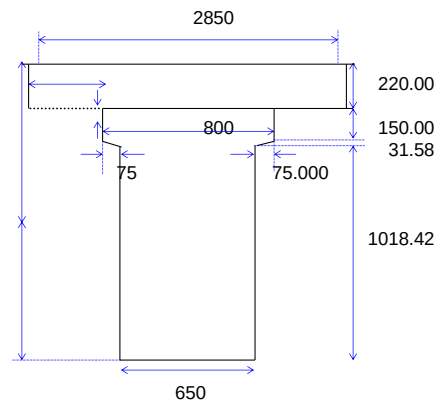
Shear Force (KN)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.00	21.91	43.83	66.78	73.33	79.42
DL of Slab	1.35	0.00	26.94	53.88	80.82	87.07	92.87
SIDL	1.35	8.83	26.17	40.57	51.74	51.74	58.48
Surfacing	1.75	4.34	15.69	26.21	35.70	35.77	40.90
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	77.39	135.46	213.72	248.99	248.99	357.19
	TOTAL V_u	135.59	331.94	553.12	705.06	722.48	918.89

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	4.72	15.13	27.58	44.02	44.02	54.55
Surfacing	1.75	0.52	1.70	3.12	4.92	4.92	6.09
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load with impact	1.5	14.99	22.40	35.27	35.27	46.65	84.20
	TOTAL T_u	29.76	57.00	95.59	120.95	138.02	210.59

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	2.850	0.220	0.627	5.459	0.116	0.104	0.284	4.995
Top Flange	0.681	0.250	0.170	0.856	0.199	0.051	0.025	0.585
Web	0.325	0.950	0.309	2.225	0.139	0.811	0.151	1.809
Bottom Flange	0.325	0.000	0.000	0.000	0.000	0.000	0.000	0.325
					0.128		0.460	7.713

Total Torsional inertia of section at mid-span = 0.0183 m⁴
 A for composite section = 1.137 m²
 perimeter u for composite section = 8.890 m
 Thickness of web = 0.325 m

Calculation of Section at Support:

	width	depth	area	perimeter	tef.i	eff.depth	Ak	uk
Deck Slab	2.850	0.220	0.627	5.378	0.117	0.103	0.283	4.911
Top Flange	0.763	0.182	0.138	0.476	0.291	0.000	0.000	0.000
Web	0.650	1.018	0.662	2.687	0.246	0.772	0.312	1.948
					0.185		0.594	6.859

Total Torsional inertia of section at support	=	0.067 m ⁴
A for comosite section	=	1.432 m ²
perimeter u for composite section	=	8.390 m
Thickness of web	=	0.650 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 = 35.00 N/mm ²		fcd = 15.63 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	29.76	57.00	95.59	120.95	138.02	210.59
effective cover	m	0.100	0.100	0.100	0.068	0.068	0.068
t _{ef,i} = max(A / u, 2*eff. Cover)	m	0.128	0.128	0.128	0.147	0.157	0.185
Area of section , Ak	m ²	0.460	0.460	0.460	0.505	0.529	0.594
Perimeter of section , uk	m	7.713	7.713	7.713	7.428	7.272	6.859
Shear stress due to pure Torsion, τ _{t,i} = T _{Ed,i} / (2 .Ak. t _{ef,i})	N/mm ²	0.253	0.484	0.812	0.816	0.829	0.960
Length of section , Zi	m	0.325	0.325	0.325	0.433	0.493	0.488
Shear force due to torsion =V _{Ed,i} = τ _{t,i} . t _{ef} . Zi	kN	10.51	20.13	33.76	51.92	64.24	86.38
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]		0.53	0.53	0.53	0.53	0.53	0.53
cot θ		2.5	2.5	2.5	2.5	2.5	2.5
θ = smallest angle before concrete crushing starts hence θ	deg	21.8	21.8	21.8	21.8	21.8	21.8
Maximum Torsional Moment T _{Rd,max} = 2 . v . α _{cw} . fcd .Ak. tef. sinθ . cosθ	kNm	337.71	337.71	337.71	425.33	477.52	629.70
Torsional Long. Reinf. Asl = T _{Ed} . cot θ/(2 . Ak. fyd)	mm ²	1434.63	2747.14	4607.56	5117.15	5451.60	6988.08
Torsional reinf. In web on each face	mm ²	168.21	322.10	540.24	622.99	763.47	1037.61
Torsional reinf. In bottom flange	mm ²	60.45	115.76	194.15	223.88	487.28	662.25
Torsional reinf. In top flange	mm ²	108.72	208.19	349.19	402.67	0.00	0.00
Total torsional steel in girder		337.39	646.05	1083.58	1249.54	1250.75	1699.86
Total torsional moment, T _{Rd,max}	kNm	337.71	337.71	337.71	425.33	477.52	629.70
Total shear force due to torsion, V _{Edi}	kN	10.51	20.13	33.76	51.92	64.24	86.38
Design shear force, Vu	kN	135.6	331.9	553.1	705.1	722.5	918.9
Total shear Force , V _{Ed} =V _u +V _{Edi}	kN	146.11	352.07	586.89	756.98	786.72	1005.27
Maximum shear capacity V _{Rd,max}	kN	1248.83	1248.83	1248.83	1705.48	1939.37	1918.66
T _{Ed} /T _{Rd,max} + V _{Ed} / V _{Rd,max} ≤ 1.0		0.205	0.451	0.753	0.728	0.695	0.858
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	1420.0	1420.0	1420.0	1420.0	1420.0	1420.0
Effective cover	mm	100.0	100.0	100.0	68.0	68.0	68.0
Effective depth , d	mm	1320.0	1320.0	1320.0	1352.0	1352.0	1352.0
Width of web , b_w	mm	325.00	325.00	325.00	433.33	492.76	487.50
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	1188.00	1188.00	1188.00	1216.80	1216.80	1216.80
$1 \geq \cot \theta \geq 2.5$		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	1248.83	1248.83	1248.83	1705.48	1939.37	1918.66

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

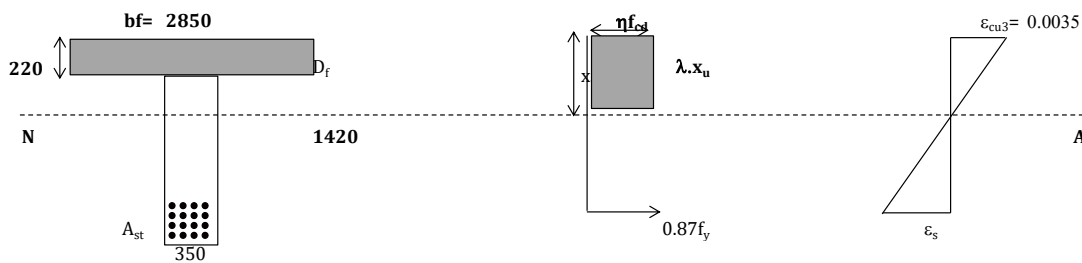
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	306.73	287.90	231.40	137.10	109.16	80.90
2	DL of Slab	1.35	367.62	344.47	275.01	159.25	125.77	92.48
3	SIDL	1.35	197.41	183.88	141.68	73.25	63.32	38.84
4	Surfacing	1.75	144.61	135.29	107.17	60.68	53.82	35.667
5	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
6	Live Load with impact	1.5	1142.07	1058.99	856.31	485.06	427.12	274.98
Total Design Moment M_{ED}			3143.05	2927.17	2346.94	1332.72	1137.50	761.39

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

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

Area of reinforcement provided A_{st} = 9651.0 mm² Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = 100.00 mm

Therefore, d_{eff} = 1.320 m

Asssuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 94.22 \text{ mm} && \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 117.78 \text{ mm} \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned}
 M_{ED} &= M_{u,lim} \\
 3.14E+09 &= \eta \cdot f_{cd} \cdot \frac{x_{u,max}}{d} \cdot (1 - 0.4 \cdot \frac{x_{u,max}}{d}) b_f \cdot d^2 \\
 d_{reqd} &= 389.6648 \text{ mm} \quad \text{OK}
 \end{aligned}$$

Area of Reinforcement required

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

$$\begin{aligned}
 \text{due to flexure} \quad A_{st,Required} &= 5589.3 \text{ mm}^2 \\
 \text{min steel required} \quad A_{s,min} &= 0.26 \cdot \frac{f_{cm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 672.67 \text{ mm}^2 \\
 &= 0.0013 \cdot b_t \cdot d_{eff} = 600.60 \text{ mm}^2 \\
 \text{due to torsion} \quad A_{st,torsion} &= 337.39 \text{ mm}^2 \\
 \text{Total Ast required due to flexure+torsion} &= 5927 \text{ mm}^2 \\
 b_t &- \text{Mean width of tension Zone}
 \end{aligned}$$

$$\text{Moment Resisting capacity } M_R = 5343.8031 \text{ KN-m} \quad \text{Safe}$$

At SECTION 2-2 (3L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

$$\text{Area of reinforcement provided } A_{st} = 9651.0 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 100.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1320 \text{ mm}$$

Asssuming N.A lies in the flange

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

$$\begin{aligned}
 \eta &= 1.00 \\
 \lambda x_u &= 94.22 \text{ mm} \quad \text{N.A lies inside the flange} \\
 \lambda &= 0.8 \\
 x_u &= 117.78 \text{ mm}
 \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}})} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned}
 M_{ED} &= M_{u,lim} \\
 2.93E+09 &= \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max}) \\
 d_{reqd} &= 406.3392 \text{ mm} \quad \text{OK}
 \end{aligned}$$

Area of Reinforcement required

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

$$A_{st, Required} = 5197.7 \text{ mm}^2$$

$$A_{s, min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 672.67 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 600.60 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, torsion} = 646.05 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 5843.80 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5343.8031 \text{ KN-m} \quad \text{Safe}$$

At SECTION 3-3 (L/4)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

$$\text{Area of reinforcement provided } A_{st} = 9651.0 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 100.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1320 \text{ mm}$$

Assuming N.A lies in the flange

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

$$\eta = 1.00$$

$$\lambda \cdot x_u = 94.22 \text{ mm}$$

$$\lambda = 0.8$$

$$x_u = 117.78 \text{ mm}$$

$$x_{u, max} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

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

Check for Depth Required

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

$$2.35E+09 = \eta f_{cd} \cdot b_f \cdot x_{u, max} \cdot (d - 0.4 \cdot x_{u, max})$$

$$d_{reqd} = 390.34 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st, Required} = 4151.04 \text{ mm}^2$$

$$A_{s, min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 672.67 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 600.60 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, torsion} = 1083.58 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 5234.62 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5343.8031 \text{ KN-m} \quad \text{Safe}$$

At SECTION 4-4 (L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

Area of reinforcement provided A_{st} = **4825.5** mm² Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = **68.00** mm

Therefore, d_{eff} = **1.352** m

Asssuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 47.11 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 58.89 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u,max} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} \cdot \mu_{u,lim} \\ 1.33E+09 &= \eta f_{cd} \cdot b_f \cdot x_{u,max} \cdot (d - 0.4 \cdot x_{u,max}) \\ d_{reqd} &= 369.41 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

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

$$A_{st,Required} = 2284.92 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 688.98 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 615.16 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 1249.54 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 3534.47 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2788.5187 \text{ KN-m} \quad \text{Safe}$$

At Deff

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

Area of reinforcement provided A_{st} = 4825.5 mm² Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = 68.00 mm

Therefore, d_{eff} = 1.352 m

Asssuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 47.11 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 58.89 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$1.14E+09 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 364.15 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st,Required} = 1947.83 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 688.98 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 615.16 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 1250.75 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 3198.58 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2788.5187 \text{ KN-m} \quad \text{Safe}$$

At Web Widening

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

Area of reinforcement provided A_{st} = 4825.5 mm² Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = 68.00 mm

Therefore, d_{eff} = 1.352 m

Asssuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 47.11 \text{ mm} & \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 58.89 \text{ mm} \end{aligned}$$

$$x_{u,max} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$7.61E+08 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 354.03 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st,Required} = 1300.72 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{cm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 688.98 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 615.16 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 1699.86 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 3000.59 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2788.5187 \text{ KN-m} \quad \text{Safe}$$

Summary

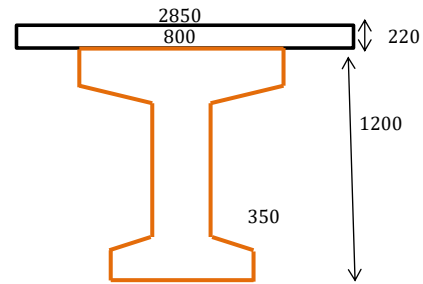
SNO	Description	unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Design Bending Moment M_{ED}	KN-m	3143.05	2927.17	2346.94	1332.72	1137.50	761.39
2	Area of Steel Required A_{st} (flexure+torsion)	mm ²	5926.71	5843.80	5234.62	3534.47	3198.58	3000.59
3	Area of Steel Provided	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
	Check for steel provided		OK	OK	OK	OK	OK	OK
3	Depth of Nuetral Axis x_u	mm	117.78	100.00	100.00	68.00	68.00	58.89
4	Critical Depth of N.A $x_{u,max}$	mm	814.10	814.10	814.10	833.83	833.83	833.83
5	Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
6	Total Depth required D_{reqd}	mm	509.66	526.34	510.34	489.41	484.15	474.03
7	Total Depth Provided	mm	1420.00	1420.00	1420.00	1420.00	1420.00	1420.00
9	Moment of Resistance M_R	KN-m	5343.80	5343.80	5343.80	2788.52	2788.52	2788.52
	Check for Moment Capacity		OK	OK	OK	OK	OK	OK
10	Additional tensile Force ΔF_d	KN	38.10	100.52	169.27	218.91	225.35	281.29
11	$M_{ED}/Z + \Delta F_d$	KN	2645.665	2463.952	1975.5372	1121.8211	957.49289	640.89921
12	M_{RD}/Z	KN	4498.151	4498.151	4498.1507	2347.2379	2347.2379	2347.2379
	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

* γ - Partial Safety factor

Refer 10.3.2 - IRC: 112-2011



$$\text{Shear force due to flexure + torsion} = 786.72 \text{ KN}$$

$$\text{Bending Moment at critical section} = 1137.50 \text{ KN-m}$$

$$\tan \beta = 0.00$$

$$\text{Effective Depth } d_{eff} = 1352.00 \text{ m}$$

$$\text{Net shear force } S - M \cdot \tan \beta / d' = 786.72 \text{ KN}$$

$$\text{Hence, Shear stress} = 1.583 \text{ N/mm}^2$$

$$1582.94 \text{ KN/m}^2$$

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

$$= 234 \text{ KN}$$

$$\text{Minimum Shear resistance} \quad v_{Rd,c} = (v_{min} + 0.15\sigma_{cp}) b_w d_{eff}$$

$$141 \text{ KN}$$

$$\text{Mean Compressive stress} \quad \sigma_{cp} = 0.2f_{cd} = 3.13 \text{ Mpa}$$

$$\text{(Limited to this value)}$$

$$N_{Ed}/A_c < 0.2f_{cd}$$

$$0.00$$

$$\text{Applied Longitudinal Force} \quad N_{Ed} = 0.00$$

$$\text{Design value of concrete compressive strength} \quad f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_m} = 15.633$$

$$v_{min} = 0.031 K^{3/2} \cdot f_{ck}^{1/2}$$

$$K = 1 + \sqrt{\frac{200}{d_{eff}}} = 1.385$$

$$\rho_1 = \frac{A_s}{b_w \cdot d_{eff}} < 0.02$$

$$\% \text{ of reinforcement} = \frac{0.00971}{0.971} \%$$

$$\text{Check for Shear resistance} = V_{Rd,Max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}$$

$$\alpha_{cw} = 1.00 \quad \text{for} \quad \sigma_{cp} = 0$$

$$b_w = 350.00$$

$$z = 1.22$$

$$v_1 = 0.60$$

$$V_{Rd,Max} = 3994.75 \text{ KN}$$

$$\text{Design Shear resistance} = 233.82 \text{ KN}$$

$$\text{Design Shear Force} = 786.72 \text{ KN}$$

Provide Shear Reinforcement

SHEAR REINFORCEMENT

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot Z \cdot f_{ywd} \cdot \cot \theta \quad \longrightarrow \quad \frac{A_{sw}}{s} = 1.616374 \text{ mm}^2/\text{mm}$$

$$V_{Rd,s} = 786.72 \text{ KN}$$

$$Z = 1216.8$$

$$f_{ywd} = 0.8 \cdot f_y = 400 \text{ N/mm}^2$$

$$\cot \theta = 2.5$$

$$S = \text{Spacing of stirrups}$$

$$\text{For combined Shear and Torsion } A_{swt,Reqd} = 1.61637 \text{ mm}^2/\text{mm}$$

$$\text{Using } 4 \text{ L- } 12 @ \quad A_{sw} = 3.02 \text{ mm}^2/\text{mm} \quad \text{OK}$$

$$\text{Spacing to be Provided } S = 150 \text{ mm}$$

Minimum Shear Reinforcement

$$\text{Shear Reinforcement ratio } \rho_w = \frac{A_{sw}}{(s \cdot b_w \cdot \sin \alpha)} = 0.0574 \quad \text{OK}$$

$$\text{Minimum Shear Reinforcement ratio } \rho_{w,min} = \frac{(0.072 \cdot \sqrt{f_{ck}})}{f_{yk}} = 0.00085 \quad \text{Refer 10.3.2 IRC: 112-2011}$$

INTERFACE SHEAR

Refer 10.3.4 IRC: 112-2011

Design Shear at Interface

Transverse Shear	V_{Ed}	=	786.72 KN	
Inner lever arm of composite section	Z	=	1.217 m	
Interface width	b_i	=	350.00 mm	
Interface area		=	350000 mm ²	
Proportion of Longitudinal tensile force carried in top slab	β	=	1.00	
Interface Shear Stress	V_{Edi}	=	$\frac{\beta \cdot V_{Ed}}{Z \cdot b_i}$	= 1.847 N/mm ²

Design Shear Resistance at Interface

$$\text{Interface shear capacity at section } V_{Rdi} = \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot [\mu \sin \alpha + \cos \alpha] \geq 0.5 \cdot v \cdot f_{cd}$$

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}	=	4.16 N/mm ²	OK

$$\text{Minimum reinforcement } A_{ci} = 525.000 \text{ mm}^2$$

Provide 10 @ 150 523.6 mm²

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	3.62	137.10	231.40	287.90	306.73
2	Dead Load-Slab	1.35	2.82	159.25	275.01	344.47	367.62
3	SIDL	1.35	197.41	183.88	141.68	73.25	63.32
4	Surfacing	1.75	0.00	60.68	107.17	135.29	144.61
5	Live Load	1.5	0.00	395.05	697.41	862.48	930.14
TOTAL			275.20	1347.05	2108.58	2483.06	2644.13

* 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	=	1.217 m
h_f	-	Thickness of Flange	=	220 mm
ΔFd	-	Change in Normal Force (unbalanced in plane SF at interface Section)	=	1833.38 KN
ν	-	Strength reduction factor	=	0.53
f_{ctd}	-		=	1.87

Description	Support - L/8	L/8 - L/4	L/4 - 3L/8	3L/8 - L/2
ΔFd	880.88	625.85	307.75	132.37
Δx	1.71875	1.71875	1.71875	1.71875
$V_{edf} = \frac{\Delta Fd}{h_f \cdot \Delta x}$	2.32959	1.65513	0.81389	0.35007
$\cot \theta_f$	2.00	2.00	2.00	2.00
$\frac{A_{sw}}{s} \cdot \frac{V_{EDf} \cdot h_f}{f_{yd} \cdot \cot \theta_f}$	589.09	418.54	205.81	88.52
Provide	2L- 10 Dia	2L- 10 Dia	2L- 10 Dia	2L- 10 Dia
A_{sf}	1047.198	1047.198	1047.198	1047.198
S_f	150.00	150.00	150.00	150.00
$VEd \leq \nu \cdot f_{ctd} \cdot \sin \theta_f \cdot \cos \theta_f$	3.33 OK	3.33 OK	3.33 OK	3.33 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

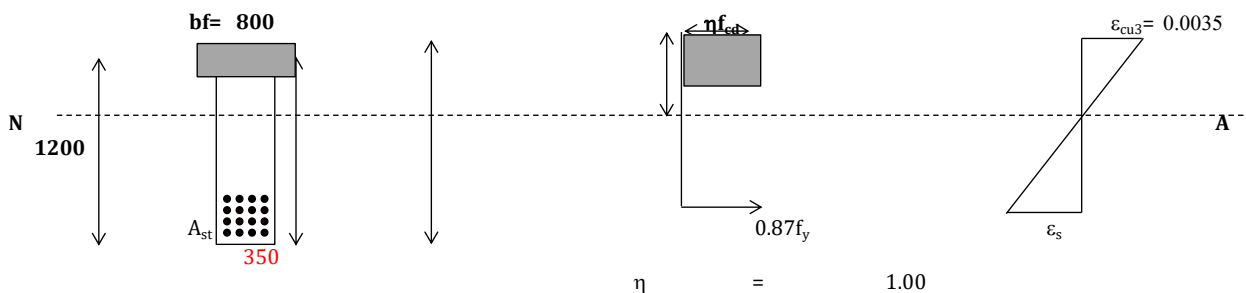
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	306.73	287.90	231.40	137.10	109.16	80.90
2	DL of Slab	1.35	367.62	344.47	275.01	159.25	125.77	92.48
Total Design Moment M_{ED}			910.38	853.70	683.66	400.07	317.16	234.07

* γ – Partial safety factor

DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	910.38	853.70	683.66	400.07	317.16	234.07
b	mm	650.11	650.11	650.11	845.67	854.71	854.71
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}	m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A	mm	516.33	516.33	516.33	198.47	196.37	196.37
Critical Depth of N.A	mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	1976.78	1849.05	1469.78	821.66	649.84	478.47
Moment Resisting capacity M_R	KN-m	3750.9255	3750.925	3750.925	2209.526	2211.288	2211.288

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	306.73	287.90	231.40	137.10	109.16	80.90
2	DL of Slab	1	367.62	344.47	275.01	159.25	125.77	92.48
Total Design Moment M_{ED}			674.36	632.37	506.42	296.34	234.93	173.39

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	674.36	632.37	506.42	296.34	234.93	173.39
b		mm	650.11	650.11	650.11	845.67	854.71	854.71
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		m ²	0.510	0.510	0.510	0.510	0.657	0.657
u		m	3.5	3.5	3.5	3.5	3.5	3.5
2Ac/u			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-to)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_o			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
$\beta(fcm)$			2.9	2.9	2.9	2.9	2.9	2.9
$\beta(to)$			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_o			3.0	3.0	3.0	3.0	3.0	3.0
$t-t_o$			4.0	4.0	4.0	4.0	4.0	4.0
$\beta(t,to)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			749.2	749.2	749.2	749.2	893.6	893.6
$E_{c,eff}$			19661.8	19661.8	19661.8	19661.8	20226.3	20226.3
Modular Ratio	m		10.17	10.17	10.17	10.17	9.89	9.89
Depth of N.A	xu	mm	444.82	444.82	444.82	309.08	304.04	304.04
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	1.01E+09	1.01E+09	1.01E+09	3.39E+08	3.32E+08	3.32E+08
Moment of Inertia of transformed area	I_S	mm ⁴	4.14E+09	4.14E+09	4.14E+09	3.27E+09	3.31E+09	3.31E+09
Modulus of Section of Concrete	Z_C	mm ³	2.27E+06	2.27E+06	2.27E+06	1.10E+06	1.09E+06	1.09E+06
Modulus of Section of Steel	Z_S	mm ³	6.32E+06	6.32E+06	6.32E+06	3.97E+06	4.00E+06	4.00E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.15E+09	5.15E+09	5.15E+09	3.61E+09	3.64E+09	3.64E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	5.72	5.37	4.30	2.50	1.98	1.46
At C.G of Steel	fy'	N/mm ²	85.75	80.41	64.40	67.62	53.45	39.44
At bottom fibre of Steel	fy''	N/mm ²	89.94	84.34	67.54	67.62	53.45	39.44

STRESS IN GIRDER DUE TO GREEN WEIGHT OF SLAB**SLS CHECK AT EACH SECTION****Material Properties**

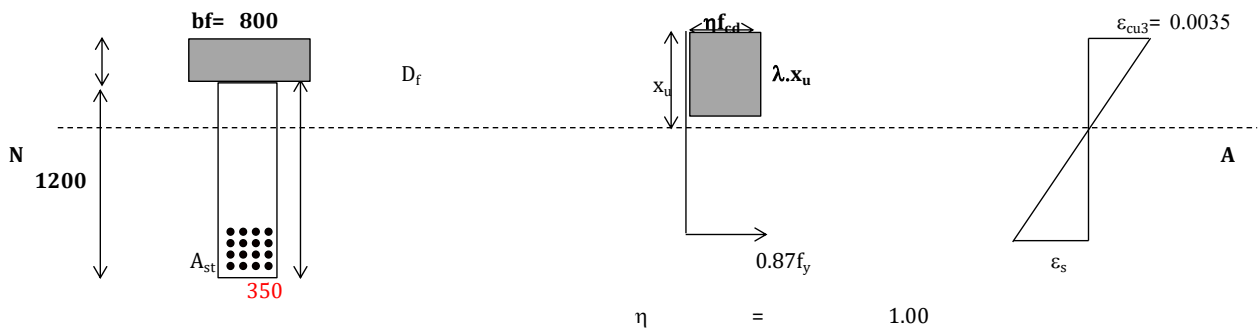
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	mm	

ULS Moments

S.No	Type of Loading	γ	BM(KN-m)					Web Widening
			L/2	3L/8	L/4	L/8	Deff	
1	Green weight of slab	1.35	14.45	13.54	10.81	6.26	4.94	3.63
Total Design Moment M_{ED}			19.50	18.28	14.59	8.45	6.67	4.91



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	19.50	18.28	14.59	8.45	6.67	4.91
b	mm	651.67	607.90	607.90	729.56	735.74	735.74
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of girder, d_{eff}	m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A	mm	515.10	552.19	552.19	230.05	228.12	228.12
Critical Depth of N.A	mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	40.79	38.22	30.51	17.16	13.55	9.97
Moment Resisting capacity M_R	KN-m	3753.003	3690.716	3690.7164	2183.004	2184.6285	2184.6285

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	14.45	13.54	10.81	6.26	4.94	3.63
Total Design Moment M_{ED}			14.45	13.54	10.81	6.26	4.94	3.63

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	14.45	13.54	10.81	6.26	4.94	3.63
b		mm	651.67	607.90	607.90	729.56	735.74	735.74
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	0.51	0.51	0.51	0.51	0.66	0.66
u		mm	3.5	3.5	3.5	3.5	3.5	3.5
2Ac/u			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-t_0)$			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
$\beta(t_0)$			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_o			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			749.2	749.2	749.2	749.2	893.6	893.6
$E_{c,eff}$			19985.3	19985.3	19985.3	19985.3	20542.2	20542.2
Modular Ratio	m		10.01	10.01	10.01	10.01	9.74	9.74
Depth of N.A	xu	mm	441.72	453.31	453.31	321.11	316.35	316.35
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_c	mm ⁴	1.00E+09	1.09E+09	1.09E+09	3.86E+08	3.79E+08	3.79E+08
Moment of Inertia of transformed area	I_s	mm ⁴	4.18E+09	4.04E+09	4.04E+09	3.17E+09	3.21E+09	3.21E+09
Modulus of Section of Concrete	Z_c	mm ³	2.27E+06	2.40E+06	2.40E+06	1.20E+06	1.20E+06	1.20E+06
Modulus of Section of Steel	Z_s	mm ³	6.35E+06	6.24E+06	6.24E+06	3.91E+06	3.94E+06	3.94E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.19E+09	5.12E+09	5.12E+09	3.56E+09	3.59E+09	3.59E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	0.12	0.12	0.10	0.06	0.04	0.03
At C.G of Steel	fy'	N/mm ²	1.83	1.71	1.36	1.43	1.12	0.83
At bottom fibre of Steel	fy''	N/mm ²	1.92	1.97	1.58	1.55	1.22	0.89

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

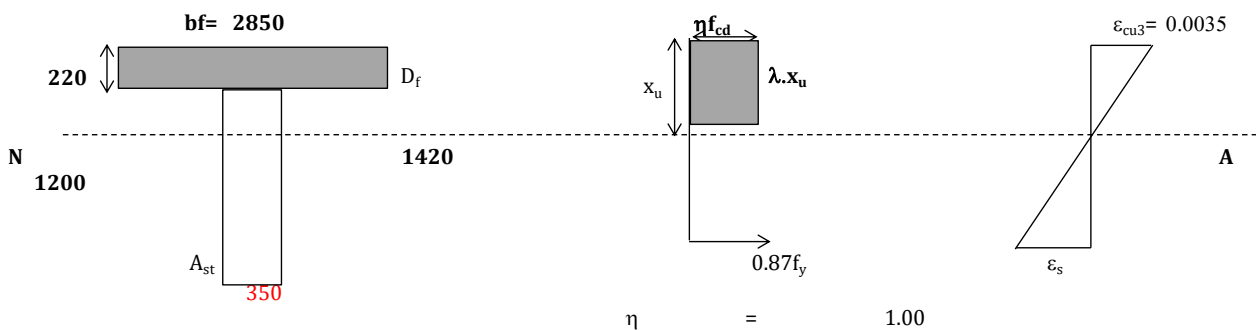
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	As per Clause 6.2.2, IRC: 112-2011
f_{yd}	-	435	N/mm ²	
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	=	40	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	197.41	183.88	141.68	73.25	63.32	38.84
2	Surfacing	1.75	144.61	135.29	107.17	60.68	53.82	35.67
Total Design Moment M_{ED}			519.57	484.99	378.81	205.07	179.65	114.85

* γ – Partial safety factor

DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	519.57	484.99	378.81	205.07	179.65	114.85
b	mm	2479.46	2479.46	2479.46	2144.41	2133.82	2144.41
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}	m	1.32	1.32	1.32	1.35	1.35	1.35
Depth of N.A	mm	135.38	135.38	135.38	78.27	78.66	78.27
Critical Depth of N.A	mm	814.10	814.10	814.10	833.83	833.83	833.83
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	908.36	847.69	661.58	349.27	305.92	195.47
Moment Resisting capacity M_R	KN-m	5314.245	5314.2451	5314.2451	2772.249	2771.9228	2772.2489

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	197.41	183.88	141.68	73.25	63.32	38.84
2	Surfacing	1.2	144.61	135.29	107.17	60.68	53.82	35.67
Total Design Moment M_{ED}			370.94	346.22	270.28	146.06	127.89	81.64

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	370.94	346.22	270.28	146.06	127.89	81.64
b		mm	2479.46	2479.46	2479.46	2144.41	2133.82	2144.41
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	1.1	1.1	1.1	1.1	1.3	1.1
u		mm	8.8	8.8	8.8	8.8	8.8	8.8
2Ac/u			259.4	259.4	259.4	259.4	293.1	259.4
$\phi(t-t_0)$			0.9	0.9	0.9	0.9	0.8	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			50.0	50.0	50.0	50.0	50.0	50.0
t_0			14.0	14.0	14.0	14.0	14.0	14.0
$t-t_0$			36.0	36.0	36.0	36.0	36.0	36.0
$\beta(t,t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
βH			697.6	697.6	697.6	697.6	755.6	697.6
$E_{c,eff}$			17028.5	17028.5	17028.5	17028.5	17300.1	17028.5
Modular Ratio	m		11.75	11.75	11.75	11.75	11.56	11.75
Depth of N.A	xu	mm	304.68	304.68	304.68	239.04	237.87	239.04
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.29E+09	2.29E+09	2.29E+09	1.10E+09	1.11E+09	1.10E+09
Moment of Inertia of transformed ar	I_S	mm ⁴	9.95E+09	9.95E+09	9.95E+09	5.98E+09	5.99E+09	5.98E+09
Modulus of Section of Concrete	Z_C	mm ³	7.51E+06	7.51E+06	7.51E+06	4.62E+06	4.65E+06	4.62E+06
Modulus of Section of Steel	Z_S	mm ³	9.80E+06	9.80E+06	9.80E+06	5.37E+06	5.38E+06	5.37E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.22E+10	1.22E+10	1.22E+10	7.08E+09	7.10E+09	7.08E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0.79	0.73	0.57	0.42	0.37	0.23
Top of Girder	fck''	N/mm ²	0.22	0.20	0.16	0.03	0.03	0.02
At C.G of Steel	fy'	N/mm ²	30.78	28.73	22.43	22.95	20.08	12.83
At bottom fibre of Steel	fy''	N/mm ²	31.75	31.56	24.63	24.36	21.31	13.61

STRESS IN GIRDER DUE TO CONSTRUCTION LOAD**SLS CHECK AT EACH SECTION****Material Properties**

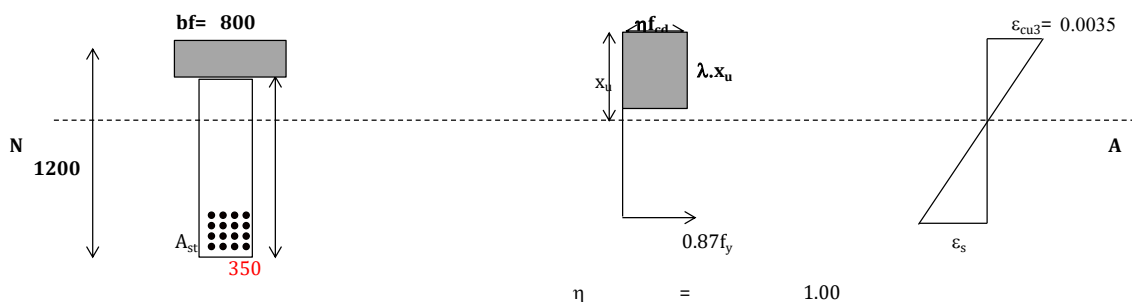
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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}	-	35.00	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	=	40	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	236.40	221.52	176.85	102.41	92.95
Total Design Moment M_{ED}			236.40	221.52	176.85	102.41	92.95



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	236.40	221.52	176.85	102.41	92.95	59.47
b_{eff}	mm	651.67	651.67	651.67	812.40	820.87	820.87
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}	m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A	mm	515.10	515.10	515.10	206.59	204.46	204.46
Critical Depth of N.A	mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	498.88	467.17	372.28	208.62	189.31	120.99
Moment Resisting capacity M_R	KN-m	2233.566	2073.302	1430.6029	1000.058	860.13671	6.6122284

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	236.40	221.52	176.85	102.41	92.95	59.47
Total Design Moment M_{ED}			236.40	221.52	176.85	102.41	92.95	59.47

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	236.40	221.52	176.85	102.41	92.95	59.47
b		mm	651.67	651.67	651.67	812.40	820.87	820.87
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
A_c		mm ²	0.5	0.5	0.5	0.5	0.7	0.7
u		mm	3.5	3.5	3.5	3.5	3.5	3.5
$2A_c/u$			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-t_0)$			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			749.2	749.2	749.2	749.2	893.6	893.6
$E_{c,eff}$			19985.3	19985.3	19985.3	19985.3	20542.2	20542.2
Modular Ratio	m		10.01	10.01	10.01	10.01	9.74	9.74
Depth of N.A	xu	mm	441.72	441.72	441.72	307.03	302.20	302.20
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_c	mm ⁴	1.00E+09	1.00E+09	1.00E+09	3.37E+08	3.31E+08	3.31E+08
Moment of Inertia of transformed area	I_s	mm ⁴	4.18E+09	4.18E+09	4.18E+09	3.28E+09	3.32E+09	3.32E+09
Modulus of Section of Concrete	Z_c	mm ³	2.27E+06	2.27E+06	2.27E+06	1.10E+06	1.09E+06	1.09E+06
Modulus of Section of Steel	Z_s	mm ³	6.35E+06	6.35E+06	6.35E+06	3.98E+06	4.00E+06	4.00E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.19E+09	5.19E+09	5.19E+09	3.62E+09	3.65E+09	3.65E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	2.01	1.89	1.50	0.87	0.79	0.51

At C.G of Steel	f_y'	N/mm ²	30.00	28.11	22.44	23.33	21.11	13.51
At bottom fibre of Steel	f_y''	N/mm ²	31.46	29.48	23.54	23.33	21.11	13.51

STRESS IN GIRDER DUE TO CWLL**SLS CHECK AT EACH SECTION****Material Properties**

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

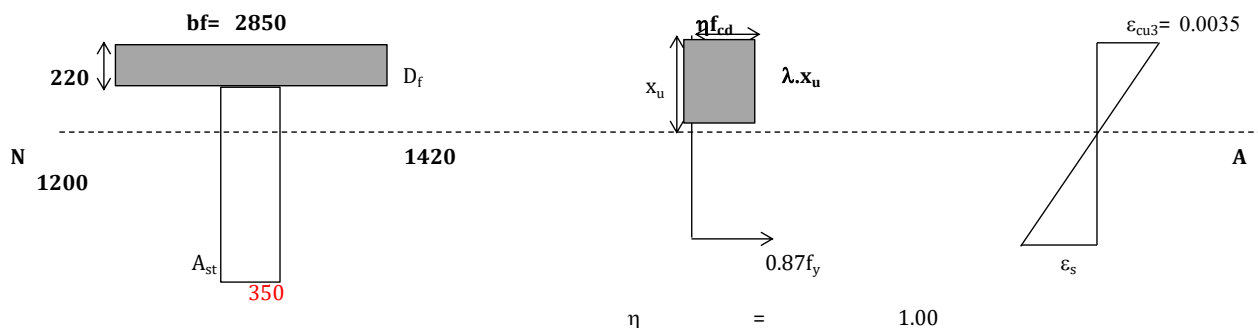
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	930.14	862.48	697.41	395.05	347.86	223.95
2	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
	Impact factor	1.228	1142.07	1058.99	856.31	485.06	427.12	274.98
	Lane Reduction factor	1.150	1313.38	1217.84	984.75	557.82	491.19	316.22
	Total Design Moment M_{ED}		1970.07	1826.75	1477.13	836.73	736.79	474.33

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	1970.07	1826.75	1477.13	836.73	736.79	474.33
b		mm	2451.86	2451.86	2451.86	2043.49	2033.52	2043.49
Area of reinforcement provided A_{st}		mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}		m	1.32	1.32	1.32	1.35	1.35	1.35
Depth of N.A		mm	136.91	136.91	136.91	82.13	82.54	82.13
Critical Depth of N.A		mm	814.10	814.10	814.10	833.83	833.83	833.83
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	3483.13	3226.13	2601.60	1433.06	1260.83	809.83
Moment Resisting capacity M_R		KN-m	5311.686	5311.68604	5311.686	2769.004	2768.6655	2769.0036

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	930.14	862.48	697.41	395.05	347.86	223.95
2	FPLL	1	0.00	0.00	0.00	0.00	0.00	0.00
Impact factor		1.228	1142.07	1058.99	856.31	485.06	427.12	274.98
Lane Reduction factor		1.150	1313.38	1217.84	984.75	557.82	491.19	316.22
Total Design Moment M_{ED}			1313.38	1217.84	984.75	557.82	491.19	316.22

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	1313.38	1217.84	984.75	557.82	491.19	316.22
b		mm	2451.86	2451.86	2451.86	2043.49	2033.52	2043.49
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	1.1	1.1	1.1	1.1	1.3	1.1
u		mm	8.8	8.8	8.8	8.8	8.8	8.8
2Ac/u			259.4	259.4	259.4	259.4	293.1	259.4
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_o			1.7	1.7	1.7	1.7	1.7	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			78.0	78.0	78.0	78.0	78.0	78.0
t_o			50.0	50.0	50.0	50.0	50.0	50.0
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			697.6	697.6	697.6	697.6	755.6	697.6
$E_{c,eff}$			19485.9	19485.9	19485.9	19485.9	19746.9	19485.9
Modular Ratio	m		10.26	10.26	10.26	10.26	10.13	10.26
Depth of N.A	x_u	mm	288.67	288.67	288.67	229.87	229.00	229.87
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.23E+09	2.23E+09	2.23E+09	1.12E+09	1.13E+09	1.12E+09
Moment of Inertia of transformed area	I_S	mm ⁴	1.03E+10	1.03E+10	1.03E+10	6.08E+09	6.09E+09	6.08E+09
Modulus of Section of Concrete	Z_C	mm ³	7.71E+06	7.71E+06	7.71E+06	4.89E+06	4.92E+06	4.89E+06
Modulus of Section of Steel	Z_S	mm ³	9.95E+06	9.95E+06	9.95E+06	5.41E+06	5.42E+06	5.41E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.25E+10	1.25E+10	1.25E+10	7.20E+09	7.21E+09	7.20E+09
Actual Stresses								
Top of Slab	f _{ck} '	N/mm ²	2.96	2.74	2.22	1.74	1.54	0.98
Top of Girder	f _{ck} ''	N/mm ²	0.70	0.65	0.53	0.07	0.06	0.04
At C.G of Steel	f _y '	N/mm ²	108.43	100.55	81.30	86.93	76.49	49.28
At bottom fibre of Steel	f _y ''	N/mm ²	111.80	110.30	89.19	92.20	81.12	52.27

CHECK FOR DEFLECTION

Refer 12.4.1 IRC 112-2011

$$\text{Allowable deflection due to Vehicular load} = \frac{\text{Span}}{800} = 17.1875 \text{ mm}$$

$$\text{Deflection due to LL (70R W)} = 2.003 \text{ mm} < 17.2 \text{ mm}$$

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	5.72	85.75	89.94
DL-Green Weight	1	0.12	1.83	1.92
Construction Load	1	2.01	30.00	31.46
Resultant Stress		7.86	117.59	123.33
CHECK		SAFE		

3L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
5.37	80.41	84.34
0.12	1.71	1.97
1.89	28.11	29.48
7.37	110.24	115.80
SAFE		

L/4		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
4.30	64.40	67.54
0.10	1.36	1.58
1.50	22.44	23.54
5.90	88.21	92.65
SAFE		

L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.50	67.62	67.62
0.06	1.43	1.55
0.87	23.33	23.33
3.42	92.37	92.49
SAFE		

Deff		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
1.98	53.45	53.45
0.04	1.12	1.22
0.79	21.11	21.11
2.82	75.68	75.77
SAFE		

Web Widening		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
1.46	39.44	39.44
0.03	0.83	0.89
0.51	13.51	13.51
2.00	53.78	53.85
SAFE		

LC-2 STAGE 2 DL - Construction Load + SIDL + LL AFTER 78 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	7.74	115.76	121.40	0	7.25	108.53	113.82	0	5.80	86.84	91.08	0	3.36	90.95	90.95	0	2.77	74.56	74.56	0	1.97	52.95	52.95
-ve Construction Load	1	0.00	-2.01	-30.00	-31.46	0.00	-1.89	-28.11	-29.48	0.00	-1.50	-22.44	-23.54	0.00	-0.87	-23.33	-23.33	0.00	-0.79	-21.11	-21.11	0.00	-0.51	-13.51	-13.51
SIDL	1	0.79	0.22	30.78	31.75	0.73	0.20	28.73	31.56	0.57	0.16	22.43	24.63	0.42	0.03	22.95	24.36	0.37	0.03	20.08	21.31	0.23	0.02	12.83	13.61
Live Load	1	2.96	0.70	108.43	111.80	2.74	0.65	100.55	110.30	2.22	0.53	81.30	89.19	1.74	0.07	86.93	92.20	1.54	0.06	76.49	81.12	0.98	0.04	49.28	52.27
Differential Shrinkage	1	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34
Resultant Stress		3.75	7.02	225.31	233.83	3.48	6.60	210.03	226.54	2.80	5.36	168.47	181.71	2.16	2.98	177.85	184.52	1.92	2.45	150.36	156.21	1.23	1.90	101.90	105.67
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
Temperature Rise	1	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		3.59	116.87	3.53	109.48	3.37	87.16	3.22	90.91	3.17	73.87	3.03	52.61
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
CWLL	1	2.96	108.43	2.74	100.55	2.22	81.30	1.74	86.93	1.54	76.49	0.98	49.28
Temperature Rise	0.6	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		5.43	225.31	5.16	210.03	4.47	168.47	3.84	177.85	3.59	150.35	2.90	101.90
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
Temperature fall	1	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		-0.66	120.84	-0.71	113.45	-0.87	91.13	-2.01	93.63	-2.06	76.58	-2.19	55.33
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
CWLL	1	2.96	108.43	2.74	100.55	2.22	81.30	1.74	86.93	1.54	76.49	0.98	49.28
Temperature fall	0.6	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		2.30	229.27	2.03	213.99	1.34	172.43	-0.27	180.56	-0.52	153.07	-1.21	104.61
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
Temperature Rise	0.5	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033	2.79	-0.0033
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		2.19	116.87	2.14	109.48	1.98	87.17	1.82	90.92	1.78	73.87	1.64	52.62
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	0.79	116.53	0.73	109.14	0.57	86.82	0.42	90.57	0.37	73.53	0.23	52.27
Temperature fall	0.5	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		0.07	118.86	0.02	111.47	-0.15	89.15	-0.79	92.27	-0.84	75.23	-0.98	53.97
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

$$S_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = \mathbf{0.09 \text{ mm}} \quad \mathbf{OK} \quad \text{Refer Cl. 12.3.4 IRC 112}$$

$S_{r,max}$	-	Maximum Crack spacing
ϵ_{sm}	-	Mean Strain in Reinforcement
ϵ_{cm}	-	Mean Strain in Concrete
σ_{sc}	-	118.86 N/mm ² (from Quasi permanent combination)
K_t	-	0.5
$f_{ct,eff}$	-	2.8
$\rho_{p,eff}$	-	0.175
α_e	-	6.25
E_s	-	200000
A_s	-	9651 mm ²
c	-	40
ϕ	-	32
h	-	1420 mm
d	-	1320
x	-	947.60

$$h_{c,eff} = \text{Min} \begin{cases} 2.5(h-d) \\ (h-x)/3 \\ h/2 \end{cases} = \begin{matrix} 250 \\ 157 \\ 710 \end{matrix} = 157.47 \text{ mm}$$

$$\phi_{eq} = \frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2} = 32$$

$$A_{c,eff} = 55113 \text{ mm}^2$$

$$\epsilon_{sm} - \epsilon_{cm} = \frac{\sigma_{sc} - K_t \frac{f_{ct,eff}}{\rho_{p,eff}} (1 + \alpha_e \cdot \rho_{p,eff})}{E_s} \geq 0.6 \cdot \frac{\sigma_{sc}}{E_s}$$

$$= 0.00051 \geq 0.0004$$

For the case of Deformed bars associated with pure bending

$$S_{r,max} = 3.4c + \frac{0.17 \cdot \phi}{\rho_{p,eff}} = 167.07$$

DESIGN OF INNER GIRDER

ULS DESIGN OF LONGITUDINAL MEMBERS FOR TORSION

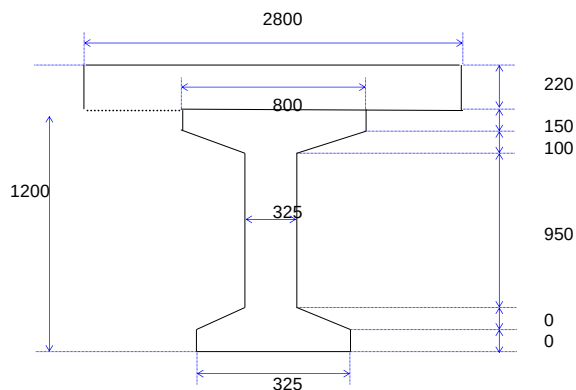
ULTIMATE BENDING MOMENT AND SHEAR FORCE

Bending Moment (kNm)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	292.73	274.82	221.09	131.39	104.73	77.70
DL of Slab	1.35	361.17	338.43	270.19	156.45	123.57	90.86
SIDL	1.35	24.45	21.81	12.97	0.84	2.00	6.75
Surfacing	1.75	123.04	115.42	92.43	53.66	47.84	32.40
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	1481.14	1370.41	1101.01	589.99	507.68	300.85
Live Load(SV Loading)	1.15	1285.45	1198.13	942.89	514.67	450.51	290.42
	TOTAL M_u	3352.80	218791.08	164235.1	76883.73	36179.38	21261.66

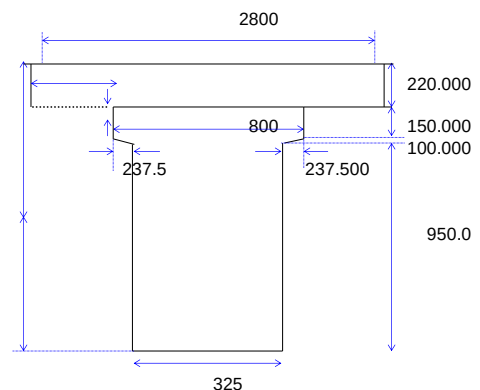
Shear Force (KN)		L/2	3L/8	L/4	L/8	Deff	Web Widening
DL of Precast Girder	1.35	0.00	20.84	41.68	63.64	70.07	76.04
DL of Slab	1.35	0.00	26.47	52.94	79.41	85.55	91.25
SIDL	1.35	2.23	6.38	8.97	9.34	9.34	9.48
Surfacing	1.75	3.25	12.52	21.48	30.06	30.13	34.76
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	83.31	231.30	364.62	502.20	502.20	590.66
Live Load(SV Loading)	1.15	59.00	163.81	258.23	355.66	355.66	418.31
	TOTAL V_u	133.66	441.33	724.36	1011.63	1028.72	1185.44

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	0.08	0.21	0.56	1.93	1.93	2.63
Surfacing	1.75	0.04	0.11	0.16	0.11	0.08	0.08
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(class A+ class 70 R)	1.5	0.83	2.10	4.58	7.67	7.67	9.00
Live Load(SV Loading)	1.15	7.98	30.60	49.37	86.22	86.22	107.08
	TOTAL T_u	1.41	3.63	7.90	14.31	14.26	17.20

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	2.800	0.220	0.616	5.359	0.116	0.104	0.279	4.895
Top Flange	0.681	0.250	0.170	0.856	0.199	0.051	0.025	0.260
Web	0.325	0.950	0.309	2.225	0.139	0.811	0.151	1.809
Bottom Flange	0.325	0.000	0.000	0.000	0.000	0.000	0.000	0.000
					0.128		0.455	6.963

Total Torsional inertia of section at mid-span = 0.0161 m⁴

Ak for comosite section = 1.101 m²

perimeter u for composite section = 8.765 m

Thickness of web = 0.3250 m

Calculation of Section at Support:

	width	depth	area	perimeter	tef.i	eff.depth	Ak	uk
Deck Slab	2.800	0.220	0.616	5.359	0.116	0.104	0.279	4.895
Top Flange	0.681	0.250	0.170	0.856	0.199	0.051	0.025	0.260
Web	0.325	0.950	0.309	2.225	0.139	0.811	0.151	1.809
					0.1281		0.455	6.963

Total Torsional inertia of section at support	=	0.067 m ⁴
Ak for comosite section	=	1.421 m ²
perimeter u for composite section	=	7.850 m
Thickness of web	=	0.650 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	=	35.00	N/mm ²	fcd	=	15.63	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	1.41	3.63	7.90	14.31	14.26	17.20
Total Area of section , A	m ²	1.101	1.101	1.101	1.208	1.266	1.421
effective cover	m	0.100	0.100	0.100	0.100	0.068	0.068
t _{ef,i}	m	0.128	0.128	0.128	0.128	0.128	0.128
Area of section , Ak	m ²	0.455	0.455	0.455	0.455	0.455	0.455
Perimeter of section , uk	m	6.963	6.963	6.963	6.963	6.963	6.963
Shear stress due to pure Torsion, τ _{t,i} = T _{Ed} / (2 . Ak . t _{ef,i})	N/mm ²	0.012	0.031	0.068	0.123	0.122	0.148
Length of section , Zi	m	0.325	0.325	0.325	0.325	0.325	0.325
Shear force due to torsion =V _{Ed,i} = τ _{t,i} . t _{ef} . Zi	kN	0.50	1.30	2.82	5.11	5.09	6.14
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]		0.53	0.53	0.53	0.53	0.53	0.53
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	334.25	334.25	334.25	334.25	334.25	334.25
Torsional Long. Reinf. Asl = T _{Ed} . cot θ / (2 . Ak . fyd)	mm ²	62.19	159.69	347.52	629.63	627.62	756.91
Torsional reinf. In web on each face	mm ²	8.08	20.74	45.14	81.78	85.63	103.27
Torsional reinf. In bottom flange	mm ²	0.00	0.00	0.00	0.00	29.29	35.33
Torsional reinf. In top flange	mm ²	2.32	5.95	12.95	23.47	23.39	28.21
Total torsional steel in girder		10.40	26.69	58.09	105.24	138.32	166.81
Total torsional moment, T _{Rd,max}	kNm	334.25	334.25	334.25	334.25	334.25	334.25
Total shear force due to torsion, V _{Edi}	kN	0.50	1.30	2.82	5.11	5.09	6.14
Design shear force, Vu	kN	133.7	441.3	724.4	1011.6	1028.7	1185.4
Total shear Force , V _{Ed} =V _u +V _{Edi}	kN	134.16	442.63	727.18	1016.74	1033.82	1191.59
Maximum shear capacity V _{Rd,max}	kN	1343.35	1343.35	1343.35	1343.35	1343.38	1343.38
T _{Ed} /T _{Rd,max} + V _{Ed} /V _{Rd,max} ≤ 1.0		0.104	0.340	0.565	0.800	0.812	0.938
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 , ϕ	=	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	1420.0	1420.0	1420.0	1420.0	1420.0	1420.0
Effective cover	mm	0.100	0.100	0.100	0.100	0.068	0.068
Effective depth , d	mm	1419.9	1419.9	1419.9	1419.9	1419.9	1419.9
Width of web , bw	mm	325.00	325.00	325.00	325.00	325.00	325.00
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	1277.91	1277.91	1277.91	1277.91	1277.94	1277.94
$1 \geq \cot \theta \geq 2.5$		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	1343.35	1343.35	1343.35	1343.35	1343.38	1343.38

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

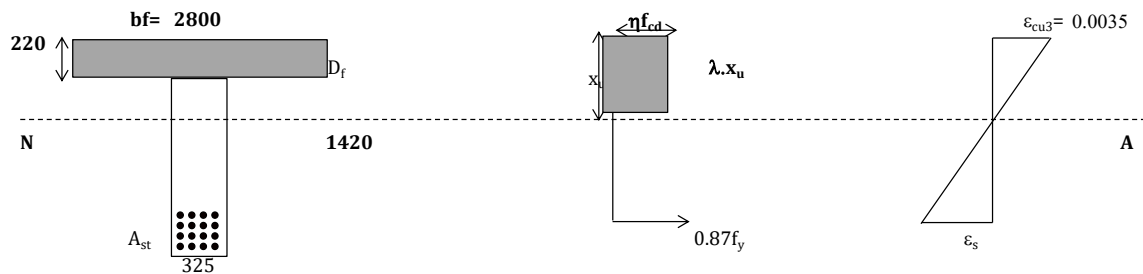
Design Constants

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	=	40	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	292.73	274.82	221.09	131.39	104.73	77.70
2	DL of Slab	1.35	361.17	338.43	270.19	156.45	123.57	90.86
3	SIDL	1.35	24.45	21.81	12.97	0.84	2.00	6.75
4	Surfacing	1.75	123.04	115.42	92.43	53.66	47.84	32.40
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	1481.14	1370.41	1101.01	589.99	507.68	300.85
7	Live Load (SV loading)	1	1285.45	1198.13	942.89	514.67	450.51	290.42
Total Design Moment M_{ED}			3352.80	3114.94	2494.00	1368.60	1156.13	744.63

* γ – Partial safety factor

**At Midspan(L/2)**

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

Area of reinforcement provided A_{st} = **9651.0 mm²** Ok (Check for Maximum Reinforcement)

C.G. of reinforcement from bottom of girder = **100.00 mm**

Therefore, d_{eff} = **1.320 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	=	95.91 mm	N.A lies inside the flange
λ	=	0.8	
x_u	=	119.88 mm	

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$3.35E+09 = \eta \cdot f_{cd} \cdot \frac{x_{u,max}}{d} \cdot \left(1 - 0.4 \cdot \frac{x_{u,max}}{d}\right) b_f \cdot d^2$$

$$d_{reqd} = 406.0344 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st,Required} = 5973.40 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 618.21 \text{ mm}^2$$

$$b_t = \text{Mean width of tension Zone}$$

$$0.0013 \cdot b_t \cdot d_{eff} = 557.70 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 10.40 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 5983.79 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5340.2712 \text{ KN-m} \quad \text{Safe}$$

At SECTION 2-2 (3L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

$$\text{Area of reinforcement provided } A_{st} = 9651.0 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 100.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.320 \text{ m}$$

Asssuming N.A lies in the flange

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

$$\eta = 1.00$$

$$\lambda x_u = 95.91 \text{ mm}$$

N.A lies inside the flange

$$\lambda = 0.8$$

$$x_u = 119.88 \text{ mm}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$3.11E+09 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 413.0492 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st, \text{Required}} = 5540.37 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 618.21 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 557.70 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, \text{torsion}} = 26.69 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 5567.07 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5340.2712 \text{ KN-m} \quad \text{Safe}$$

At SECTION 3-3 (L/4)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	6	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	12		

$$\text{Area of reinforcement provided } A_{st} = 9651.0 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 100.00 \text{ mm}$$

$$\text{Therefore } d_{eff} = 1.320 \text{ m}$$

$$\text{Asssuming N.A lies in the flange} \quad b_f \cdot \eta \cdot f_{cd} \cdot \lambda x = 0.87 \cdot f_y \cdot A_{st}$$

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 95.91 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 119.88 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u, \text{max}} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu, 3}} \right)} = 814.10 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$2.49\text{E}+09 = \eta f_{cd} \cdot b_f \cdot x_{u, \text{max}} \cdot (d - 0.4 \cdot x_{u, \text{max}})$$

$$d_{reqd} = 395.62 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st, \text{Required}} = 4416.86 \text{ mm}^2$$

$$A_{s, \text{min}} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 618.21 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 557.70 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st, \text{torsion}} = 58.09 \text{ mm}^2$$

$$\text{Total Ast required due to flexure+torsion} = 4474.95 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 5340.2712 \text{ KN-m} \quad \text{Safe}$$

At SECTION 4-4 (L/8)

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

$$\text{Area of reinforcement provided } A_{st} = 4825.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 68.00 \text{ mm}$$

$$\text{Therefore } d_{eff} = 1.352 \text{ m}$$

Assuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda \cdot x_u &= 47.95 \text{ mm} \quad \text{N.A lies inside the flange} \\ \lambda &= 0.8 \\ x_u &= 59.94 \text{ mm} \end{aligned}$$

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= M_{u,lim} \\ 1.37E+09 &= \eta f_{cd} \cdot b_f \cdot x_{u,max} \cdot (d - 0.4 \cdot x_{u,max}) \\ d_{reqd} &= 371.03 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

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

$$A_{st,Required} = 2347.33 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 633.20 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 571.22 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 105.24 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 2452.58 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2787.6357 \text{ KN-m} \quad \text{Safe}$$

At Deff

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

$$\text{Area of reinforcement provided } A_{st} = 4825.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 68.00 \text{ mm}$$

$$\text{Therefore, } d_{eff} = 1.352 \text{ m}$$

Assuming N.A lies in the flange

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

$$\eta = 1.00$$

$$\lambda x_u = 47.95 \text{ mm}$$

$$\lambda = 0.8$$

$$x_u = 59.94 \text{ mm}$$

N.A lies inside the flange

$$x_{u,max} = \frac{d_{eff}}{\left(1 + \frac{f_{yk}}{\gamma_s \cdot E_s \cdot \epsilon_{cu,3}}\right)} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

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

$$1.16 \times 10^9 = \eta f_{cd} b_f x_{u,max} (d - 0.4 x_{u,max})$$

$$d_{reqd} = 365.21 \text{ mm} \quad \text{OK}$$

Area of Reinforcement required

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

$$A_{st,Required} = 1980.22 \text{ mm}^2$$

$$A_{s,min} = 0.26 \cdot \frac{f_{cm}}{f_{yk}} \cdot b_t \cdot d_{eff} = 633.20 \text{ mm}^2$$

$$0.0013 \cdot b_t \cdot d_{eff} = 571.22 \text{ mm}^2$$

$$\text{due to torsion} \quad A_{st,torsion} = 138.32 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 2118.54 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2787.6357 \text{ KN-m} \quad \text{Safe}$$

At Web Widening

Reinforcement	No of bars	Dia(mm)	Dis. from bottom of girder(mm)
1st Layer	6	32	68
2nd Layer	0	32	132
3rd Layer	0	32	196
4th Layer	0	32	260
5th Layer	0	32	324
6th Layer	0	32	388
7th Layer	0	32	452
8th Layer	0	25	513
Total no. of bars	6		

$$\text{Area of reinforcement provided } A_{st} = 4825.5 \text{ mm}^2 \quad \text{Ok} \quad (\text{Check for Maximum Reinforcement})$$

$$\text{C.G. of reinforcement from bottom of girder} = 68.00 \text{ mm}$$

$$\text{Therefore, } d_{\text{eff}} = 1.352 \text{ m}$$

Assuming N.A lies in the flange

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

$$\begin{aligned} \eta &= 1.00 \\ \lambda x_u &= 47.95 \text{ mm} \\ \lambda &= 0.8 \\ x_u &= 59.94 \text{ mm} \end{aligned} \quad \text{N.A lies inside the flange}$$

$$x_{u,\text{max}} = 833.83 \text{ mm} \quad \text{under reinforced section}$$

Check for Depth Required

$$\begin{aligned} M_{ED} &= \frac{d_{\text{eff}}}{\left(1 + \frac{f_{yk}}{\gamma_s E_s \cdot \epsilon_{cu,3}}\right)} \\ 7.45\text{E}+08 &= \eta f_{cd} b_f x_{u,\text{max}} (d - 0.4 x_{u,\text{max}}) \\ d_{\text{reqd}} &= 353.93 \text{ mm} \quad \text{OK} \end{aligned}$$

Area of Reinforcement required

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

$$\begin{aligned} A_{st,\text{Required}} &= 1272.07 \text{ mm}^2 \\ A_{s,\text{min}} &= 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d_{\text{eff}} = 633.20 \text{ mm}^2 \\ &= 0.0013 \cdot b_t \cdot d_{\text{eff}} = 571.22 \text{ mm}^2 \end{aligned}$$

$$\text{due to torsion} \quad A_{st,\text{torsion}} = 166.81 \text{ mm}^2$$

$$\text{Total } A_{st} \text{ required due to flexure+torsion} = 1438.88 \text{ mm}^2$$

$$\text{Moment Resisting capacity } M_R = 2787.6357 \text{ KN-m} \quad \text{Safe}$$

Summary

SNO	Description	unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
1	Design Bending Moment M_{ED}	KN-m	3352.80	3114.94	2494.00	1368.60	1156.13	744.63
	Area of Steel required (flexure+torsion)	mm ²	5983.79	5567.07	4474.95	2452.58	2118.54	1438.88
2	Area of Steel Provided	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
	Check for steel provided		OK	OK	OK	OK	OK	OK
3	Depth of Neutral Axis x_u	mm	119.88	100.00	100.00	68.00	68.00	59.94
4	Critical Depth of N.A $x_{u,\text{max}}$	mm	814.10	814.10	814.10	833.83	833.83	833.83
5	Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
6	Total Depth required D_{reqd}	mm	526.03	533.05	515.62	491.03	485.21	473.93
7	Total Depth Provided	mm	1420.00	1420.00	1420.00	1420.00	1420.00	1420.00
8	Area of Steel Required A_{st}	mm ²	5973.40	5540.37	4416.86	2347.33	1980.22	1272.07
9	Moment of Resistance M_R	KN-m	5340.27	5340.27	5340.27	2787.64	2787.64	2787.64
Check for Moment Capacity			OK	OK	OK	OK	OK	OK
10	Additional tensile Force ΔF_d	KN	54.33	122.72	193.65	264.80	271.71	313.55
11	$M_{ED}/Z + \Delta F_d$	KN	2822.223	2621.99992	2099.3226	1152.024	973.17523	626.7929
12	M_{RD}/Z	KN	4495.178	4495.17778	4495.1778	2346.495	2346.4947	2346.4947
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

$$\text{Shear corresponding to torsion and flexure} = 1033.82 \text{ KN}$$

$$\text{Bending Moment at critical section} = 36179.38 \text{ KN-m}$$

$$\tan \beta = 0.00$$

$$\text{Effective Depth } d_{eff} = 1352.00 \text{ m}$$

$$\text{Net shear force } S - M \cdot \tan \beta / d' = 1033.82 \text{ KN}$$

$$\text{Hence, Shear stress} = \frac{2.240 \text{ N/mm}^2}{2240.13 \text{ KN/m}^2}$$

Design Shear resistance of the member (without Shear R/F)

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

$$= 222 \text{ KN}$$

Minimum Shear resistance

$$v_{Rd,c} = (v_{min} + 0.15\sigma_{cp}) b_w d_{eff}$$

$$131 \text{ KN}$$

Mean Compressive stress

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{0.00}{N_{Ed}/A_c} = 0.00 \text{ Mpa}$$

(Limited to this value)

$$N_{Ed}/A_c < 0.2f_{cd}$$

Applied Longitudinal Force

$$N_{Ed} = 0.00$$

Design value of concrete compressive strength

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_m} = 15.633$$

$$v_{min} = 0.031K^{3/2} \cdot f_{ck}^{1/2}$$

$$K = 1 + \sqrt{\frac{200}{d_{eff}}} = 1.3846$$

$$\rho_1 = \frac{A_s}{b_w \cdot d_{eff}} < 0.02$$

$$\% \text{ of reinforcement} = \frac{0.01046}{1.046} \%$$

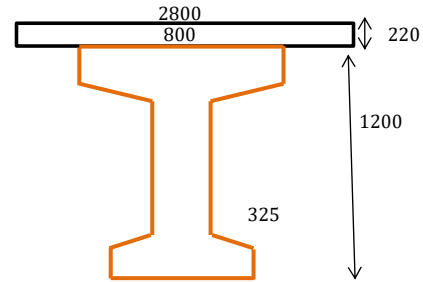
Check for Shear resistance

$$= V_{Rd,Max} = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}$$

$$\alpha_{cw} = 1.00 \text{ for } \sigma_{cp} = 0$$

$$b_w = 325.00$$

Refer 10.3.2 - IRC: 112-2011



$$\begin{aligned}
 z &= 1.22 \\
 v_l &= 0.60 \\
 V_{Rd,Max} &= 3709.41 \text{ KN}
 \end{aligned}$$

$$\begin{aligned}
 \text{Design Shear resistance} &= 222.50 \text{ KN} \\
 \text{Design Shear Force} &= 1033.82 \text{ KN}
 \end{aligned}$$

Provide Shear Reinforcement

SHEAR REINFORCEMENT

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot \cot \theta \quad \Rightarrow \quad \frac{A_{sw}}{s} = 2.124052 \text{ mm}^2/\text{m}$$

$$V_{Rd,s} = 1033.82 \text{ KN}$$

$$Z = 1216.8$$

$$f_{ywd} = 0.8 \cdot f_y = 400 \text{ N/mm}^2$$

$$\cot \theta = 2.5$$

$$S = \text{Spacing of stirrups}$$

$$\text{For combined Shear and Torsion} \quad A_{swt,Reqd} = 2.12405 \text{ mm}^2/\text{mm}$$

$$\text{Using } 4 \text{ L- } 12 @ \quad A_{sw} = 3.02 \text{ mm}^2/\text{mm} \quad \text{OK}$$

$$\text{Spacing to be Provided} \quad S = 150 \text{ mm}$$

Minimum Shear Reinforcement

$$\text{Shear Reinforcement ratio} \quad \rho_w = \frac{A_{sw}}{(s \cdot b_w \cdot \sin \alpha)} = 0.0619 \quad \text{OK}$$

$$\text{Minimum Shear Reinforcement ratio} \quad \rho_{w,min} = \frac{(0.072 \cdot \sqrt{f_{ck}})}{f_{yk}} = 0.00085 \quad \text{Refer 10.3.2 IRC: 112-2011}$$

INTERFACE SHEAR

Refer 10.3.4 IRC: 112-2011

Design Shear at Interface

$$\begin{aligned}
 \text{Transverse Shear} \quad V_{Ed} &= 1033.82 \text{ KN} \\
 \text{Inner lever arm of composite section} \quad Z &= 1.217 \text{ m} \\
 \text{Interface width} \quad b_i &= 325.00 \text{ mm} \\
 \text{Interface area} &= 325000 \text{ mm}^2 \\
 \text{Proportion of Longitudinal tensile force carried in top sla} \quad \beta &= 1.00 \\
 \text{Interface Shear Stress} \quad V_{Edi} &= \frac{\beta \cdot V_{ED}}{Z \cdot b_i} = 2.614 \text{ N/mm}^2
 \end{aligned}$$

Design Shear Resistance at Interface

$$\begin{aligned}
 \text{Interface shear capacity at section} \quad V_{Rdi} &= \mu \cdot \sigma_n + \rho \cdot f_{yd} \cdot [\mu \sin \alpha + \cos \alpha] \geq 0.5 \cdot v \cdot f_{cd} \\
 \text{Factor dependent on interface roughness} \quad \mu &= 0.6 \\
 \text{Stress resulting from forces normal to interface} \quad \sigma_n &= 0.00 \text{ Mpa} \\
 \text{Angle of reinforcement to interface} \quad \alpha &= 90 \text{ degree} \\
 \text{Strength reduction factor for concrete cracked in shear} \quad v &= 0.5 \\
 V_{Rdi} &= 4.16 \text{ N/mm}^2 \quad \text{OK}
 \end{aligned}$$

$$\begin{aligned}
 \text{Minimum reinforcement} \quad A_{ci} &= 487.500 \text{ mm}^2 \\
 \text{Provide } 10 @ 150 &= 523.6 \text{ mm}^2
 \end{aligned}$$

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
			BM(KN-m)				
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	24.45	21.81	12.97	0.84	2.00
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			82.12	2588.10	4406.27	5366.63	5712.41

* γ - Partial Safety factorLongitudinal 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	=	1.217 m
h_f	-	Thickness of Flange	=	220 mm
ΔFd	-	Change in Normal Force (unbalanced in plane SF at interface Section)	=	4324.15 KN
ψ	-	Strength reduction factor	=	0.53
f_{ctd}	-		=	1.85

Description	Support - L/8	L/8 - L/4	L/4 - 3L/8	3L/8 - L/2
ΔFd	2059.48	1494.22	789.25	284.17
Δx	3.125	3.125	3.125	3.125
$V_{edf} = \frac{\Delta Fd}{h_f \cdot \Delta x}$	2.99561	2.17341	1.14801	0.41334
$\cot \theta_f$	2.00	2.00	2.00	2.00
$\frac{A_{sw}}{s} \cdot \frac{V_{EDf} \cdot h_f}{f_{yd} \cdot \cot \theta_f}$	757.51	549.60	290.30	104.52
Provide	2L- 10 Dia	2L- 10 Dia	2L- 10 Dia	2L- 10 Dia
A_{sf}	1047.198	1047.198	1047.198	1047.198
S_f	150	150	150	150
$VEd \leq \psi \cdot f_{cd} \cdot \sin \theta_f \cdot \cos \theta_f$	3.33 OK	3.33 OK	3.33 OK	3.33 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

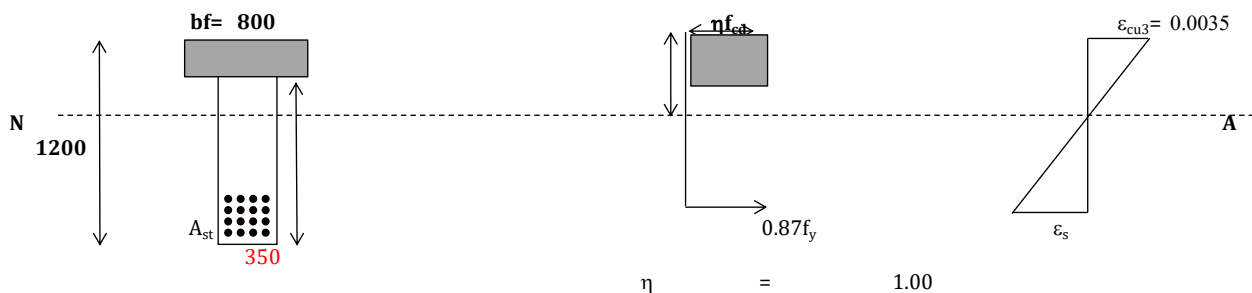
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

Design Constants

f_y	-	500	N/mm ²	As per Clause 6.2.2, IRC: 112-2011 at 14 day
f_{yd}	-	435	N/mm ²	
f_{ck}	-	30.57	N/mm ²	
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	=	40	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	292.73	274.82	221.09	131.39	104.73	77.70
2	DL of Slab	1.35	361.17	338.43	270.19	156.45	123.57	90.86
Total Design Moment M_{ED}			882.77	827.88	663.23	388.59	308.20	227.55

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	882.77	827.88	663.23	388.59	308.20	227.55
beff		mm	650.11	650.11	650.11	845.67	854.71	854.71
Area of reinforcement provided A_{st}		mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}		m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A		mm	516.34	516.34	516.34	198.47	196.37	196.37
Critical Depth of N.A		mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	1916.31	1792.69	1425.59	798.40	631.62	465.22
Moment Resisting capacity M_R		KN-m	3750.9248	3750.925	3750.925	2209.526	2211.288	2211.288

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	292.73	274.82	221.09	131.39	104.73	77.70
2	DL of Slab	1	361.17	338.43	270.19	156.45	123.57	90.86
Total Design Moment M_{ED}			653.90	613.25	491.28	287.84	228.29	168.56

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	653.90	613.25	491.28	287.84	228.29	168.56
b		mm	650.11	650.11	650.11	845.67	854.71	854.71
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		m ²	0.5	0.5	0.5	0.5	0.7	0.7
u		m	3.5	3.5	3.5	3.5	3.5	3.5
2Ac/u			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-to)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_o			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
$\beta(fcm)$			2.9	2.9	2.9	2.9	2.9	2.9
$\beta(to)$			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_o			3.0	3.0	3.0	3.0	3.0	3.0
$t-t_o$			4.0	4.0	4.0	4.0	4.0	4.0
$\beta(t,to)$			0.2	0.2	0.2	0.2	0.2	0.2
βH			749.2	749.2	749.2	749.2	893.6	893.6
$E_{c,eff}$			19661.8	19661.8	19661.8	19661.8	20226.3	20226.3
Modular Ratio	m		10.17	10.17	10.17	10.17	9.89	9.89
Depth of N.A	xu	mm	444.83	444.83	444.83	309.08	304.04	304.04
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	1.01E+09	1.01E+09	1.01E+09	3.39E+08	3.32E+08	3.32E+08
Moment of Inertia of transformed area	I_S	mm ⁴	4.14E+09	4.14E+09	4.14E+09	3.27E+09	3.31E+09	3.31E+09
Modulus of Section of Concrete	Z_C	mm ³	2.27E+06	2.27E+06	2.27E+06	1.10E+06	1.09E+06	1.09E+06
Modulus of Section of Steel	Z_S	mm ³	6.32E+06	6.32E+06	6.32E+06	3.97E+06	4.00E+06	4.00E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.15E+09	5.15E+09	5.15E+09	3.61E+09	3.64E+09	3.64E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	5.55	5.21	4.17	2.43	1.93	1.42
At C.G of Steel	fy'	N/mm ²	83.15	77.98	62.47	65.68	51.93	38.34
At bottom fibre of Steel	fy''	N/mm ²	87.21	81.79	65.52	65.68	51.93	38.34

STRESS IN GIRDER DUE TO GREEN WEIGHT OF SLAB**SLS CHECK AT EACH SECTION****Material Properties**

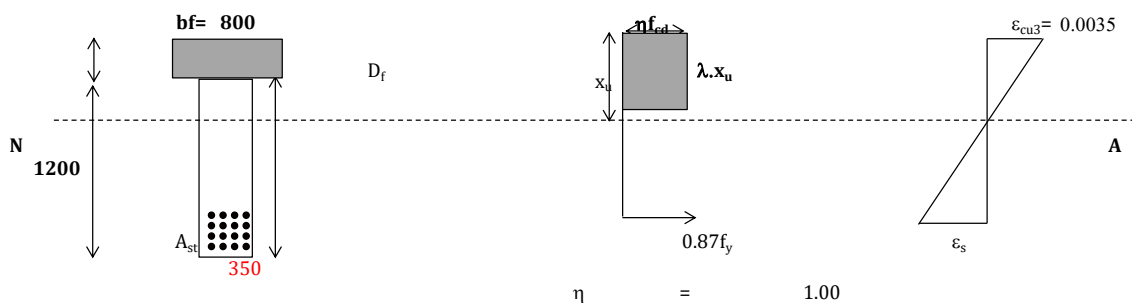
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	14.45	13.54	10.81	6.26	4.94	3.63
Total Design Moment M_{ED}			19.50	18.28	14.59	8.45	6.67	4.91



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	19.50	18.28	14.59	8.45	6.67	4.91
b_{eff}	mm	651.67	1097.13	1097.13	1484.79	1500.58	1500.58
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
$\phi(t, t_o) = \rho(t, t_o) \phi(\infty, t_o)$							
Effective depth of girder, d_{eff}	m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A	mm	515.10	305.96	305.96	113.04	111.85	111.85
Critical Depth of N.A	mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$ mm ²	40.79	38.21	30.50	17.16	13.55	9.96
Moment Resisting capacity M_R	KN-m	3753.003	4104.204	4104.2041	2281.255	2282.2541	2282.2541

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	14.45	13.54	10.81	6.26	4.94	3.63
Total Design Moment M_{ED}			14.45	13.54	10.81	6.26	4.94	3.63

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	14.45	13.54	10.81	6.26	4.94	3.63
b		mm	651.67	1097.13	1097.13	1484.79	1500.58	1500.58
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
A_c		mm ²	0.5	0.5	0.5	0.5	0.7	0.7
u		mm	3.5	3.5	3.5	3.5	3.5	3.5
$2A_c/u$			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-t_0)$			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			749.2	749.2	749.2	749.2	893.6	893.6
$E_{c,eff}$			19985.3	19985.3	19985.3	19985.3	20542.2	20542.2
Modular Ratio	m		10.01	10.01	10.01	10.01	9.74	9.74
Depth of N.A	xu	mm	441.72	360.76	360.76	236.94	233.00	233.00
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_c	mm ⁴	1.00E+09	5.47E+08	5.47E+08	1.55E+08	1.52E+08	1.52E+08
Moment of Inertia of transformed area	I_s	mm ⁴	4.18E+09	5.27E+09	5.27E+09	3.87E+09	3.90E+09	3.90E+09
Modulus of Section of Concrete	Z_c	mm ³	2.27E+06	1.52E+06	1.52E+06	6.54E+05	6.51E+05	6.51E+05
Modulus of Section of Steel	Z_s	mm ³	6.35E+06	7.13E+06	7.13E+06	4.32E+06	4.34E+06	4.34E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.19E+09	5.82E+09	5.82E+09	4.02E+09	4.05E+09	4.05E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	0.12	0.08	0.07	0.04	0.03	0.02
At C.G of Steel	fy'	N/mm ²	1.83	1.72	1.37	1.39	1.10	0.81
At bottom fibre of Steel	fy''	N/mm ²	1.92	1.79	1.43	1.39	1.10	0.81

STRESS IN GIRDER DUE TO CONSTRUCTION LOAD**SLS CHECK AT EACH SECTION****Material Properties**

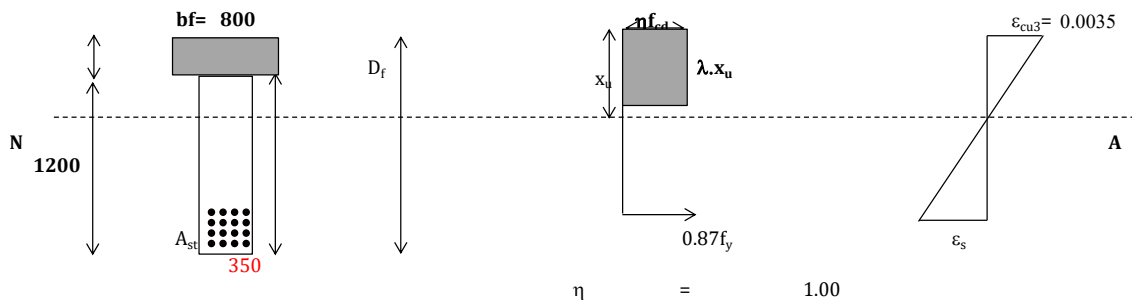
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	236.40	221.52	176.85	102.41	92.95
Total Design Moment M_{ED}			236.40	221.52	176.85	102.41	92.95



DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	236.40	221.52	176.85	102.41	92.95	59.47
b	mm	652.52	652.52	652.52	812.19	819.42	819.42
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}	m	1.10	1.10	1.10	1.13	1.13	1.13
Depth of N.A	mm	514.43	514.43	514.43	206.65	204.83	204.83
Critical Depth of N.A	mm	678.41	678.41	678.41	698.15	698.15	698.15
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	498.88	467.17	372.28	208.62	189.31	120.99
Moment Resisting capacity M_R	KN-m	3754.121	3754.121	3754.1215	2202.656	2204.1873	2204.1873

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	236.40	221.52	176.85	102.41	92.95	59.47
Total Design Moment M_{ED}			236.40	221.52	176.85	102.41	92.95	59.47

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	236.40	221.52	176.85	102.41	92.95	59.47
b		mm	652.52	652.52	652.52	812.19	819.42	819.42
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	0.5	0.5	0.5	0.5	0.7	0.7
u		mm	3.5	3.5	3.5	3.5	3.5	3.5
2Ac/u			289.4	289.4	289.4	289.4	373.0	373.0
$\phi(t-t_0)$			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	440.06	440.06	440.06	305.94	301.78	301.78
Goal Seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	1.00E+09	1.00E+09	1.00E+09	3.37E+08	3.31E+08	3.31E+08
Moment of Inertia of transformed area	I_S	mm ⁴	4.20E+09	4.20E+09	4.20E+09	3.29E+09	3.33E+09	3.33E+09
Modulus of Section of Concrete	Z_C	mm ³	2.28E+06	2.28E+06	2.28E+06	1.10E+06	1.10E+06	1.10E+06
Modulus of Section of Steel	Z_S	mm ³	6.37E+06	6.37E+06	6.37E+06	3.99E+06	4.01E+06	4.01E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	5.21E+09	5.21E+09	5.21E+09	3.63E+09	3.66E+09	3.66E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0	0	0	0	0	0
Top of Girder	fck''	N/mm ²	2.01	1.89	1.51	0.87	0.79	0.51
At C.G of Steel	fy'	N/mm ²	29.97	28.08	22.42	23.31	21.10	13.50
At bottom fibre of Steel	fy''	N/mm ²	31.42	29.45	23.51	23.31	21.10	13.50

STRESS IN GIRDER DUE TO SIDL AND SURFACING**SLS CHECK AT EACH SECTION****Material Properties**

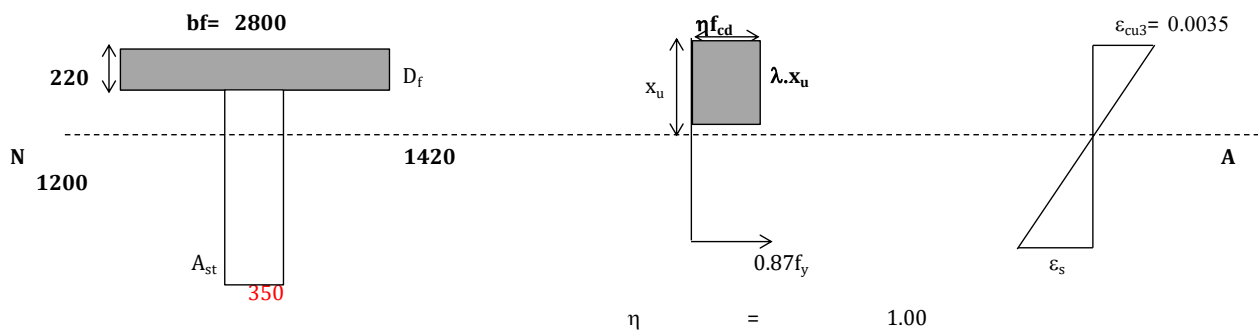
Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	24.45	21.81	12.97	0.84	2.00	6.75
2	Surfacing	1.75	123.04	115.42	92.43	53.66	47.84	32.40
Total Design Moment M_{ED}			248.32	231.43	179.26	95.04	86.42	65.81

* γ – Partial safety factor

DESCRIPTION	Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}	KN-m	248.32	231.43	179.26	95.04	86.42	65.81
b	mm	2479.46	2478.37	2478.37	2137.55	2125.84	2137.55
Area of reinforcement provided A_{st}	mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement		Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder	mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}	m	1.32	1.32	1.32	1.35	1.35	1.35
Depth of N.A	mm	135.38	135.44	135.44	78.52	78.95	78.52
Critical Depth of N.A	mm	814.10	814.10	814.10	833.83	833.83	833.83
Check for section		Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required $A_{st,Reqd}$	mm ²	433.26	403.75	312.61	161.72	147.04	111.95
Moment Resisting capacity M_R	KN-m	5314.245	5314.1454	5314.1454	2772.038	2771.675	2772.038

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	24.45	21.81	12.97	0.84	2.00	6.75
2	Surfacing	1.2	324.14	303.98	243.45	134.02	88.35	71.72
Total Design Moment M_{ED}			413.42	386.58	305.11	161.66	108.03	92.81

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	413.42	386.58	305.11	161.66	108.03	92.81
b		mm	2479.46	2478.37	2478.37	2137.55	2125.84	2137.55
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	1.1	1.1	1.1	1.1	1.3	1.1
u		mm	8.8	8.8	8.8	8.8	8.8	8.8
2Ac/u			259.4	259.4	259.4	259.4	293.1	259.4
$\phi(t-t_0)$			0.9	0.9	0.9	0.9	0.8	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			50.0	50.0	50.0	50.0	50.0	50.0
t_0			14.0	14.0	14.0	14.0	14.0	14.0
$t-t_0$			36.0	36.0	36.0	36.0	36.0	36.0
$\beta(t,t_0)$			0.4	0.4	0.4	0.4	0.4	0.4
βH			697.6	753.7	753.7	753.7	826.7	753.7
$E_{c,eff}$			17028.5	17204.5	17204.5	17204.5	17504.8	17204.5
Modular Ratio	m		11.75	11.62	11.62	11.62	11.43	11.62
Depth of N.A	xu	mm	304.68	303.38	303.38	238.28	237.01	238.28
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	2.24E+09	2.24E+09	1.09E+09	1.09E+09	1.09E+09
Moment of Inertia of transformed ar	I_S	mm ⁴	9.95E+09	9.97E+09	9.97E+09	5.99E+09	6.00E+09	5.99E+09
Modulus of Section of Concrete	Z_C	mm ³	7.38E+06	7.39E+06	7.39E+06	4.56E+06	4.59E+06	4.56E+06
Modulus of Section of Steel	Z_S	mm ³	9.80E+06	9.81E+06	9.81E+06	5.37E+06	5.38E+06	5.37E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.22E+10	1.22E+10	1.22E+10	7.07E+09	7.09E+09	7.07E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	0.88	0.83	0.65	0.47	0.32	0.27
Top of Girder	fck''	N/mm ²	0.24	0.23	0.18	0.04	0.02	0.02
At C.G of Steel	fy'	N/mm ²	34.42	32.17	25.39	25.46	17.00	14.62
At bottom fibre of Steel	fy''	N/mm ²	35.50	35.33	27.89	27.02	18.03	15.51

STRESS IN GIRDER DUE TO CWLL**SLS CHECK AT EACH SECTION****Material Properties**

Grade of Concrete	-	M-	35
Grade of Reinforcement	-	Fe-	500 D

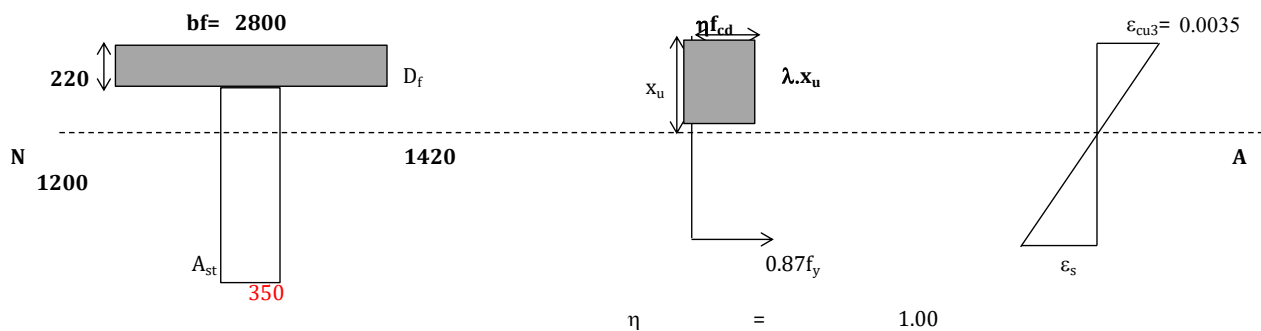
Elastic Modulus of Concrete	-	32000	N/mm ²
Elastic Modulus of Steel	-	200000	N/mm ²

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	=	40	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	1048.95	970.53	779.74	417.83	359.54	213.06
2	FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00
3	Live Load (SV Loading)	1	1285.45	1198.13	942.89	514.67	450.51	290.42
Impact factor		1.228	1287.95	1191.66	957.40	513.03	441.46	261.61
Lane Reduction factor		1.150	1481.14	1370.41	1101.01	589.99	507.68	300.85
Total Design Moment M_{ED}			2221.71	2055.62	1651.51	884.98	761.52	451.27

* γ – Partial safety factor

DESCRIPTION		Unit	L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Bending Moment M_{ED}		KN-m	2221.71	2055.62	1651.51	884.98	761.52	451.27
b		mm	2451.86	2449.81	2449.81	2037.00	2026.05	2037.00
Area of reinforcement provided A_{st}		mm ²	9651	9651	9651	4825	4825	4825
Check for Maximum Reinforcement			Ok	Ok	Ok	Ok	Ok	Ok
C.G. of reinforcement from bottom of girder		mm	100.00	100.00	100.00	68.00	68.00	68.00
Effective depth of Girder, d_{eff}		m	1.32	1.32	1.32	1.35	1.35	1.35
Depth of N.A		mm	136.91	137.02	137.02	82.39	82.84	82.39
Critical Depth of N.A		mm	814.10	814.10	814.10	833.83	833.83	833.83
Check for section			Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced	Under reinforced
Area of Reinforcement required	$A_{st,Reqd}$	mm ²	3935.82	3636.88	2912.69	1516.37	1303.46	770.31
Moment Resisting capacity M_R		KN-m	5311.686	5311.49341	5311.4934	2768.784	2768.41	2768.7838

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	1048.95	970.53	779.74	417.83	359.54	213.06
2	FPLL	1	0.00	0.00	0.00	0.00	0.00	0.00
3	Live Load(SV Loading)	1	1285.45	1198.13	942.89	514.67	450.51	290.42
Impact factor		1.228	1287.95	1191.66	957.40	513.03	441.46	261.61
Lane Reduction factor		1.150	1481.14	1370.41	1101.01	589.99	507.68	300.85
Total Design Moment M_{ED}			1481.14	1370.41	1101.01	589.99	507.68	300.85

			L/2	3L/8	L/4	L/8	Deff	Web Widening
Design Moment	M_{ED}	KN-m	1481.14	1370.41	1101.01	589.99	507.68	300.85
b		mm	2451.86	2449.81	2449.81	2037.00	2026.05	2037.00
Area of steel Provided	A_{st}	mm ²	9651.0	9651.0	9651.0	4825.5	4825.5	4825.5
Ac		mm ²	1.1	1.1	1.1	1.1	1.3	1.1
u		mm	8.8	8.8	8.8	8.8	8.8	8.8
2Ac/u			259.4	259.4	259.4	259.4	293.1	259.4
$\phi(t-t_0)$			0.6	0.6	0.6	0.6	0.6	0.6
ϕ_0			1.7	1.7	1.7	1.7	1.7	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			78.0	78.0	78.0	78.0	78.0	78.0
t_0			50.0	50.0	50.0	50.0	50.0	50.0
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			697.6	753.7	753.7	753.7	826.7	753.7
$E_{c,eff}$			19485.9	19655.8	19655.8	19655.8	19943.1	19655.8
Modular Ratio	m		10.26	10.18	10.18	10.18	10.03	10.18
Depth of N.A	x_u	mm	288.67	287.68	287.68	229.30	228.36	229.30
Goal seek			0.00	0.00	0.00	0.00	0.00	0.00
Moment of Inertia of Concrete section	I_C	mm ⁴	2.19E+09	2.18E+09	2.18E+09	1.11E+09	1.11E+09	1.11E+09
Moment of Inertia of transformed ar	I_S	mm ⁴	1.03E+10	1.03E+10	1.03E+10	6.08E+09	6.09E+09	6.08E+09
Modulus of Section of Concrete	Z_C	mm ³	7.58E+06	7.59E+06	7.59E+06	4.82E+06	4.85E+06	4.82E+06
Modulus of Section of Steel	Z_S	mm ³	9.95E+06	9.96E+06	9.96E+06	5.42E+06	5.42E+06	5.42E+06
Cracked Moment of Inertia	I_{CR}	mm ⁴	1.25E+10	1.25E+10	1.25E+10	7.19E+09	7.20E+09	7.19E+09
Actual Stresses								
Top of Slab	fck'	N/mm ²	3.35	3.11	2.50	1.85	1.61	0.94
Top of Girder	fck''	N/mm ²	0.80	0.73	0.59	0.08	0.06	0.04
At C.G of Steel	fy'	N/mm ²	122.67	113.46	91.15	92.15	79.22	46.99
At bottom fibre of Steel	fy''	N/mm ²	126.47	124.45	99.99	97.73	84.01	49.83

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+ Class A)	=	2.279	mm	
Deflection due to LL(SV Loading)	=	2.849	mm	
Deflection due to LL (70R W + Class A/sv loading)	=	2.849	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	5.55	83.15	87.21
DL-Green Weight	1	0.12	1.83	1.92
Construction Load	1	2.01	29.97	31.42
Resultant Stress		7.69	114.96	120.56
CHECK		SAFE		

3L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
5.21	77.98	81.79
0.08	1.72	1.79
1.89	28.08	29.45
7.18	107.79	113.03
SAFE		

L/4		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
4.17	62.47	65.52
0.07	1.37	1.43
1.51	22.42	23.51
5.74	86.27	90.46
SAFE		

L/8		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
2.43	65.68	65.68
0.04	1.39	1.39
0.87	23.31	23.31
3.33	90.38	90.38
SAFE		

Deff		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
1.93	51.93	51.93
0.03	1.10	1.10
0.79	21.10	21.10
2.75	74.13	74.13
SAFE		

Web Widening		
f _{ck} ''	f _y '	f _y ''
N/mm ²		
1.42	38.34	38.34
0.02	0.81	0.81
0.51	13.50	13.50
1.95	52.65	52.65
SAFE		

LC-2 STAGE 2 DL - Construction Load + SIDL + LL AFTER 78 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	7.56	113.12	118.64	0	7.09	106.07	111.24	0	5.68	84.89	89.03	0	3.30	88.99	88.99	0	2.72	73.04	73.04	0	1.93	51.85	51.85
-ve'Construction Load	1	0.00	-2.01	-29.97	-31.42	0.00	-1.89	-28.08	-29.45	0.00	-1.51	-22.42	-23.51	0.00	-0.87	-23.31	-23.31	0.00	-0.79	-21.10	-21.10	0.00	-0.51	-13.50	-13.50
SIDL	1	0.88	0.24	34.42	35.50	0.83	0.23	32.17	35.33	0.65	0.18	25.39	27.89	0.47	0.04	25.46	27.02	0.32	0.02	17.00	18.03	0.27	0.02	14.62	15.51
Live Load	1	3.35	0.80	122.67	126.47	3.11	0.73	113.46	124.45	2.50	0.59	91.15	99.99	1.85	0.08	92.15	97.73	1.61	0.06	79.22	84.01	0.94	0.04	46.99	49.83
Differential Shrinkage	1	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34	0.01	0.37	0.34	0.34
Resultant Stress		4.23	6.96	240.58	249.53	3.94	6.54	223.96	241.92	3.16	5.31	179.36	193.74	2.33	2.91	183.63	190.77	1.93	2.38	148.50	154.33	1.22	1.85	100.29	104.03
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
Temperature Rise	1	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		3.68	117.91	3.63	110.49	3.45	88.20	3.27	91.48	3.12	69.27	3.07	53.30
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
CWLL	1	3.35	122.67	3.11	113.46	2.50	91.15	1.85	92.15	1.61	79.22	0.94	46.99
Temperature Rise	0.6	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		5.91	240.58	5.62	223.95	4.83	179.36	4.00	183.63	3.61	148.49	2.90	100.29
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
Temperature fall	1	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		-0.57	121.87	-0.62	114.46	-0.79	92.17	-1.96	94.20	-2.11	71.99	-2.16	56.02
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
CWLL	1	3.35	122.67	3.11	113.46	2.50	91.15	1.85	92.15	1.61	79.22	0.94	46.99
Temperature fall	0.6	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		2.78	244.54	2.49	227.92	1.70	183.32	-0.11	186.34	-0.51	151.21	-1.22	103.01
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
Temperature Rise	0.5	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003	2.79	-0.003
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		2.28	117.91	2.23	110.50	2.06	88.21	1.87	91.48	1.72	69.27	1.67	53.30
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	0.88	117.57	0.83	110.15	0.65	87.86	0.47	91.14	0.32	68.93	0.27	52.96
Temperature fall	0.5	-1.46	3.96	-1.46	3.96	-1.46	3.96	-2.44	2.71	-2.44	2.71	-2.44	2.71
Differential Shrinkage	1	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34	0.01	0.34
Resultant Stress		0.16	119.89	0.11	112.48	-0.07	90.19	-0.74	92.84	-0.89	70.63	-0.94	54.66
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

$$S_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) = \mathbf{0.08 \text{ mm}} \quad \mathbf{OK} \quad \text{Refer Cl. 12.3.4 IRC 112}$$

$S_{r,max}$	-	Maximum Crack spacing
ϵ_{sm}	-	Mean Strain in Reinforcement
ϵ_{cm}	-	Mean Strain in Concrete
σ_{sc}	-	119.89 N/mm ² (from Quasi permanant combination)
K_t	-	0.5
$f_{ct,eff}$	-	2.77127
$\rho_{p,eff}$	-	0.236
α_e	-	6.25
E_s	-	200000
A_s	-	12064 mm ²
c	-	40
ϕ	-	32
h	-	1420 mm
d	-	1320
x	-	947.60

$$h_{c,eff} = \text{Min} \begin{cases} 2.5(h-d) \\ (h-x)/3 \\ h/2 \end{cases} = \begin{matrix} 250 \\ 157.467 \\ 710 \end{matrix} = 157.47 \text{ mm}$$

$$\phi_{eq} = \frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2} = 32$$

$$A_{c,eff} = 51177 \text{ mm}^2$$

$$\epsilon_{sm} - \epsilon_{cm} = \frac{\sigma_{sc} - K_t \frac{f_{ct,eff}}{\rho_{p,eff}} (1 + \alpha_e \cdot \rho_{p,eff})}{E_s} \geq 0.6 \cdot \frac{\sigma_{sc}}{E_s}$$

$$= 0.00053 \geq 0.0004$$

For the case of Deformed bars associated with pure bending

$$S_{r,max} = 3.4c + \frac{0.17 \cdot \phi}{\rho_{p,eff}} = 159.08$$

ANNEXURE-I

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 6-April-2018

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0.225 0 0; 2 0.625 0 0; 3 1.575 0 0; 4 2.083 0 0; 5 2.275 0 0; 6 2.625 0 0;
 7 4.2 0 0; 8 5.85 0 0; 9 7.5 0 0; 10 9.15 0 0; 11 10.8 0 0; 12 12.375 0 0;
 13 12.725 0 0; 14 12.917 0 0; 15 13.425 0 0; 16 14.375 0 0; 17 14.775 0 0;
 18 0.225 0 1.45; 19 0.625 0 1.45; 20 1.575 0 1.45; 21 2.083 0 1.45;
 22 2.275 0 1.45; 23 2.625 0 1.45; 24 4.2 0 1.45; 25 5.85 0 1.45; 26 7.5 0 1.45;
 27 9.15 0 1.45; 28 10.8 0 1.45; 29 12.375 0 1.45; 30 12.725 0 1.45;
 31 12.917 0 1.45; 32 13.425 0 1.45; 33 14.375 0 1.45; 34 14.775 0 1.45;
 35 0.225 0 4.25; 36 0.625 0 4.25; 37 1.575 0 4.25; 38 2.083 0 4.25;
 39 2.275 0 4.25; 40 2.625 0 4.25; 41 4.2 0 4.25; 42 5.85 0 4.25; 43 7.5 0 4.25;
 44 9.15 0 4.25; 45 10.8 0 4.25; 46 12.375 0 4.25; 47 12.725 0 4.25;
 48 12.917 0 4.25; 49 13.425 0 4.25; 50 14.375 0 4.25; 51 14.775 0 4.25;
 52 0.225 0 7.05; 53 0.625 0 7.05; 54 1.575 0 7.05; 55 2.083 0 7.05;
 56 2.275 0 7.05; 57 2.625 0 7.05; 58 4.2 0 7.05; 59 5.85 0 7.05; 60 7.5 0 7.05;
 61 9.15 0 7.05; 62 10.8 0 7.05; 63 12.375 0 7.05; 64 12.725 0 7.05;
 65 12.917 0 7.05; 66 13.425 0 7.05; 67 14.375 0 7.05; 68 14.775 0 7.05;
 70 0.625 0 8.5; 71 1.575 0 8.5; 73 2.275 0 8.5; 74 2.625 0 8.5; 75 4.2 0 8.5;
 76 5.85 0 8.5; 77 7.5 0 8.5; 78 9.15 0 8.5; 79 10.8 0 8.5; 80 12.375 0 8.5;
 81 12.725 0 8.5; 83 13.425 0 8.5; 84 14.375 0 8.5; 86 0.225 0 8.5;
 89 2.083 0 8.5; 99 12.917 0 8.5; 102 14.775 0 8.5; 103 0 0 0; 104 0 0 1.45;
 105 0 0 4.25; 106 0 0 7.05; 107 0 0 8.5; 109 15 0 0; 110 15 0 1.45;
 111 15 0 4.25; 112 15 0 7.05; 113 15 0 8.5;

MEMBER INCIDENCES

1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
 11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 18 19; 18 19 20;
 19 20 21; 20 21 22; 21 22 23; 22 23 24; 23 24 25; 24 25 26; 25 26 27; 26 27 28;
 27 28 29; 28 29 30; 29 30 31; 30 31 32; 31 32 33; 32 33 34; 33 35 36; 34 36 37;
 35 37 38; 36 38 39; 37 39 40; 38 40 41; 39 41 42; 40 42 43; 41 43 44; 42 44 45;
 43 45 46; 44 46 47; 45 47 48; 46 48 49; 47 49 50; 48 50 51; 49 52 53; 50 53 54;
 51 54 55; 52 55 56; 53 56 57; 54 57 58; 55 58 59; 56 59 60; 57 60 61; 58 61 62;
 59 62 63; 60 63 64; 61 64 65; 62 65 66; 63 66 67; 64 67 68; 81 86 70; 82 70 71;
 83 71 89; 84 89 73; 85 73 74; 86 74 75; 87 75 76; 88 76 77; 89 77 78; 90 78 79;
 91 79 80; 92 80 81; 93 81 99; 94 99 83; 95 83 84; 96 84 102; 97 103 1;
 98 104 18; 99 105 35; 100 106 52; 102 107 86; 103 17 109; 104 34 110;
 105 51 111; 106 68 112; 108 102 113; 109 103 104; 110 104 105; 111 105 106;
 112 106 107; 114 2 19; 115 19 36; 116 36 53; 117 53 70; 119 3 20; 120 20 37;
 121 37 54; 122 54 71; 124 5 22; 125 22 39; 126 39 56; 127 56 73; 129 6 23;
 130 23 40; 131 40 57; 132 57 74; 134 7 24; 135 24 41; 136 41 58; 137 58 75;
 139 8 25; 140 25 42; 141 42 59; 142 59 76; 144 9 26; 145 26 43; 146 43 60;
 147 60 77; 149 10 27; 150 27 44; 151 44 61; 152 61 78; 154 11 28; 155 28 45;
 156 45 62; 157 62 79; 159 12 29; 160 29 46; 161 46 63; 162 63 80; 164 13 30;
 165 30 47; 166 47 64; 167 64 81; 169 15 32; 170 32 49; 171 49 66; 172 66 83;
 174 16 33; 175 33 50; 176 50 67; 177 67 84; 179 109 110; 180 110 111;
 181 111 112; 182 112 113;

START GROUP DEFINITION

MEMBER

_OUTERGIRDER 18 TO 31

_INNERGIRDER 34 TO 47 50 TO 63

JOINT

_LEFTSUPPORT 19 36 53 70

_RIGHTSUPPORT 33 50 67 84

END GROUP DEFINITION

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 3.2e+007

```

POISSON 0.2
DENSITY 25
ALPHA 5e-006
DAMP 0.05
ISOTROPIC MATERIAL1
E 3.2e+007
POISSON 0.2
DENSITY 0
END DEFINE MATERIAL
MEMBER PROPERTY INDIAN
***DUMMY
1 TO 16 81 TO 100 102 TO 106 108 TO 112 179 TO 182 PRIS YD 0.02 ZD 0.02
****TRANSVERSE MEMBER
119 TO 122 PRIS YD 0.22 ZD 1.1
124 TO 127 PRIS YD 0.22 ZD 0.525
129 TO 132 PRIS YD 0.22 ZD 0.825
134 TO 137 PRIS YD 0.22 ZD 1.475
139 TO 142 PRIS YD 0.22 ZD 1.65
144 TO 147 PRIS YD 0.22 ZD 1.65
149 TO 152 PRIS YD 0.22 ZD 1.65
154 TO 157 PRIS YD 0.22 ZD 1.475
159 TO 162 PRIS YD 0.22 ZD 0.825
164 TO 167 PRIS YD 0.22 ZD 0.525
169 TO 172 PRIS YD 0.22 ZD 1.1
**OUTTER GIRDER-END
17 18 31 32 49 50 63 64 PRIS AX 1.432 IX 0.067 IY 0.455 IZ 0.273
**OUTTER GIRDER-TAPPERED
19 TO 21 28 TO 30 51 TO 53 60 TO 61 -
62 PRIS AX 1.284 IX 0.0427686 IY 0.446 IZ 0.227
**OUTTER GIRDER-MID
22 TO 27 54 TO 59 PRIS AX 1.137 IX 0.018 IY 0.436 IZ 0.18
**INNER GIRDER-END
33 34 47 48 PRIS AX 1.421 IX 0.0676897 IY 0.433 IZ 0.271
**INNER GIRDER-TAPPERED
35 TO 37 44 TO 46 PRIS AX 1.261 IX 0.042 IY 0.423 IZ 0.221
**INNER GIRDER-MID
38 TO 43 PRIS AX 1.101 IX 0.016 IY 0.413 IZ 0.17
****END CROSS GIRDER
115 TO 117 175 TO 177 PRIS AX 0.662 IX 0.022 IY 0.023 IZ 0.125
MEMBER PROPERTY INDIAN
114 174 PRIS YD 0.22 ZD 1.08
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 16 81 TO 100 102 TO 106 108 TO 112 114 -
119 TO 122 124 TO 127 129 TO 132 134 TO 137 139 TO 142 144 TO 147 -
149 TO 152 154 TO 157 159 TO 162 164 TO 167 169 TO 172 174 179 TO 182
MATERIAL CONCRETE MEMB 17 TO 64 115 TO 117 175 TO 177
SUPPORTS
19 36 53 FIXED BUT FZ MX MY MZ
33 50 67 FIXED BUT FX FZ MX MY MZ
DEFINE MOVING LOAD
***70R Wheel
TYPE 1 LOAD 40 60 60 85 85 85 85
DIST 3.96 1.52 2.13 1.37 3.05 1.37
***Class A
TYPE 2 LOAD 13.5 13.5 57 57 34 34 34 34
DIST 1.1 3.2 1.2 4.3 3 3 3
**SV
TYPE 3 LOAD 30 47.5 47.5 90 90 90 90 90 90 90 90 90 90 90 90 90 90 90 -
90 90
DIST 3.2 1.37 5.389 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 1.5 -
1.5 1.5 1.5 1.5

```


LOAD 1 LOADTYPE None TITLE DL
SELFWEIGHT Y -1
LOAD 2 LOADTYPE None TITLE SIDL
*CB
MEMBER LOAD
1 TO 16 81 TO 97 102 103 108 UNI GY -8
LOAD 3 LOADTYPE None TITLE WEARING COAT
*Surfacing
FLOOR LOAD
YRANGE 0 0 FLOAD -2 GY
** 2 Lane Class A(Eccentric to Outer Girder)
LOAD GENERATION 339
TYPE 2 -18.8 0 1.75 XINC 0.1
TYPE 2 -18.8 0 3.55 XINC 0.1
TYPE 2 -18.8 0 5.25 XINC 0.1
TYPE 2 -18.8 0 7.05 XINC 0.1
** 1 Lane 70R(Eccentric to Carriageway)
LOAD GENERATION 339
TYPE 1 -13.4 0 2.13 XINC 0.1
TYPE 1 -13.4 0 4.06 XINC 0.1
** SV (Concentric Carriageway)
LOAD GENERATION 536
TYPE 3 -38.459 0 3.35 XINC 0.1
TYPE 3 -38.459 0 5.15 XINC 0.1
PERFORM ANALYSIS
FINISH

DESIGN OF ABUTMENT(A1 and
A2) WITH OPEN FOUNDATION
FOR MINOR BRIDGE AT CH-
18+203

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 = 40 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)	= 0.500m	
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	= 0.000m	
Railing/kerb on footpath edge	= 0.000m	
Total Width of Superstructure	= 8.500m	
Median Width minus 20mm gap	= 0.000m	

Bearings

No of Bearings = 3
Type of Bearing = SPHERICAL BEARING
Coeff. Of Friction for POT-PTFE Bearing = 0.05
C/C Distance of Bearing in Transverse Direction = 2.800m (Right) 2.800 (Skew)

Type of Soil = 2 Medium Soil Strata

NBC of soil -Normal Case = 300 kN/m² Assumed
SBC of soil-Normal Case = 350 kN/m²
SBC of soil-Seismic Case = 438 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	= 1.885m	
Height of Trapering portion	= 0.106m	
Length of Dirt Wall at top (Uniform portion)	= 8.500m	8.500m
Length of Dirt Wall at bottom (Tapering Portion)	= 8.500m	8.500m

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	

Abutmentcap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	8.500m	8.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	8.500m	8.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	8.500m	8.500m
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	=	4.300m
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	=	0.000m
Thickness of Return wall at Bottom	=	0.000m

Foundation**Along Traffic Direction:**

Total Width of Footing	=	7.900m
abutment pedestal width	=	1.000m
abutment pedestal Height	=	1.000m
Width of Toe Slab	=	2.600m
Width of Heel Slab	=	4.300m
Thickness of Toe slab at tip	=	0.300m
Thickness of Toe slab near shaft	=	1.000m
Thickness of heel slab at tip	=	0.300m
Thickness of heel slab near shaft	=	1.000m
Width of backfill on heel slab	=	4.300m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	8.409m
Height of back fill at back fill edge of heel slab	=	7.709m

Across Traffic Direction:

Width of foundation -Uniform portion	=	8.500m (skew dimension)
Width of foundation -Tapering portion	=	8.500m (skew dimension)

Levels

Deck Level at Median Edge=	1771.003m	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	1770.911m	Height of Superstructure	=	1.420m
Deck level at center line =	1771.003m	Min. Height of Footpath Side Pedestal (1)	=	0.400m
Soffit Level at center of bridge =	1769.518m	Height of Pedestal (2)	=	0.470m
Abutment cap top level =	1768.966m	Height of Pedestal (3)	=	0.540m
Abutment cap bottom lvl (uniform portion ends)	1768.666m	Height of Pedestal (4)	=	0.000m
Abutment cap bottom lvl (corbel portion ends)	1768.366m	Height of Pedestal (5)	=	0.000m
Abutment shaft top level =	1768.366m	Height of Pedestal (6)	=	0.000m
Ground level/LBL =	1766.401m	Distance of nearest girder to c.l. of deck	=	0.000m
Abutment shaft bottom level =	1763.248m	Height (Avg.) of Dirt Wall	=	1.991m
Foundation level =	1762.248m	Abutment shaft Above G.L	=	1.964m
HFL =	1768.401m	Abutment Shaft below G.L	=	3.153m
		Height of abutment shaft	=	5.117m
		MSL	=	1764.248m
		Wedge over girder flange	=	0.0250m

Material Specification

Concrete 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 (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	
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	=	5	
Z = Zone factor	=	0.36	
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	=	492.63 2.80
DL Reaction on Bearing No.-2	=	488.24 0.00
DL Reaction on Bearing No.-3	=	492.63 -2.80
DL Reaction on Bearing No.-4	=	0.00 0.00

DL Reaction on Bearing No.-5	=	0.00 0.00
DL Reaction on Bearing No.-6	=	0.00 0.00
		1473.50 0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	217.402 2.80
SIDL Reaction on Bearing No.-2	=	-189.636 0.00
SIDL Reaction on Bearing No.-3	=	217.402 -2.80
SIDL Reaction on Bearing No.-4	=	0.000 0.00
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		245.17 0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	64.496 2.80
SIDL Reaction on Bearing No.-2	=	76.846 0.00
SIDL Reaction on Bearing No.-3	=	64.496 -2.80
SIDL Reaction on Bearing No.-4	=	0.000 0.00
SIDL Reaction on Bearing No.-5	=	0.000 0.00
SIDL Reaction on Bearing No.-6	=	0.000 0.00
		205.84 0.000

Carriageway Live Load Reactions**MAXIMUM REACTION CASE:****(1-70RW)**

Reduction Factor = 1 (for 2 Lane)

max	=	809.52	KN	Corr. Transv	=	935.000 kNm	ecc.	1.16
min	=	190.48	KN	Corr. Transv	=	220.000 kNm	ecc.	1.16

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	=	392.35	KN	Corr. Transv	=	961.246 kNm	ecc.	2.45
min	=	107.65	KN	Corr. Transv	=	263.754 kNm	ecc.	2.45

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	=	0.400 x	0.800 x	0.800 =	0.2560 m ³	2.800
Pedestal 2	=	0.470 x	0.800 x	0.800 =	0.3008 m ³	0.000
Pedestal 3	=	0.540 x	0.800 x	0.800 =	0.3456 m ³	-2.800
Pedestal 4	=	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
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m ³	0.000
					0.9024 m ³	-0.278

VOLUME CALCULATION

C.G. Of Footing	=	3.950 m
C.G. Of shaft from toe tip	=	3.100 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	8.50	0.300	1.885	4.807	-0.800	0.050	-3.900	0.000
-Trapering portion	1	8.50	0.300	0.106	0.271	-0.800	0.050	-3.900	0.000
Bracket (Rectangle)	1	8.50	0.300	0.300	0.765	-1.100	-0.250	-4.200	0.000
(Corbel)	0.5	1	8.50	0.300	0.383	-1.050	-0.200	-4.150	0.000
Cap (uniform portion)	1	8.50	1.850	0.300	4.718	-0.025	0.825	-3.125	0.000
Cap (Corbel Portion)	1	8.50	1.850	0.300	3.579	-0.025	0.825	-3.125	0.000
		8.50	1.000						
Shaft above HFL	1	8.50	1.000	0.000	0.000	0.000	0.850	-3.100	0.000
Shaft below HFL	1	8.50	1.000	5.117	43.499	0.000	0.850	-3.100	0.000
Solid Return Wall	2	4.30	0.500	8.105	34.851	-2.650	-1.800	-5.750	0.000
Cantilever Return wall(Rectangular portion)	2	0.00	0.000	0.000	0.000	-0.500	0.350	-3.600	0.000
Cantilever Return wall(Traingular portion)	2	0.00	0.000	0.000	0.000	-0.500	0.350	-3.600	0.000
Footing									
Heel Slab	1	8.50	4.300	0.650	23.758		-1.414	-5.364	0.000
Toe Slab	1	8.50	2.600	0.650	14.365		2.417	-1.533	0.000
Portion between Heel and Toe	1	8.50	1.000	1.000	8.500		0.850	-3.100	0.000
Back filling above HFL over Heel Slab	1	8.50	4.300	2.602	95.103		-1.800	-5.750	0.000
Back filling below HFL over Heel Slab	1	8.50	4.300	5.503	201.135		-1.846	-5.796	0.000
Backfill above Heel slab	1	8.50	4.300	8.059	294.547		-1.831	-5.781	0.000
Front Filling over Toe Slab	1	8.50	2.600	3.503	77.416		2.607	-1.343	0.000
Front Filling over Toe Slab in HFL Case	1	8.50	2.600	1.350	29.835		1.688	-2.262	0.000
Side filling between heel and toe	1	0.00	1.000	3.503	0.000		0.000	0.000	0.000
Approach Slab	1	8.500	1.750	0.300	4.463	-1.100	-0.250	-4.200	0.000
Back fill above HFL on flared portion of stem	1	8.50	0.000	2.602	0.000		0.000	0.000	0.000
Back fill below HFL on flared portion of stem	1	8.50	0.000	5.503	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	0.050	-3.900

SECTIONAL PROPERTIES

Width of Footing (B)	=	7.900 m
Length of Footing (L)	=	8.500 m

A	=	7.900	x	8.500	=	67.150	m ²
ZL	=	8.500	x	10.402	=	88.414	m ³
ZT	=	IT1	+	IT2			
		distance of extreme point from centre					
IT1	=	7.900	x	51.177	=	404.30	m ⁴
IT2 (moment of inertia of triangle)	=	7.900	x	0.000	+	0.500 x 7.900 x	0.000 x 18.063
from centre of footing	=	0.000	m ⁴				
Moment of inertia of two triangle	=	0.000	m ⁴				
Total moment of inertia	=	404.30	m ⁴				
Distance of extreme point from centre of footing	=	4.250	+	0.000	=	4.250	m
Total Section modulus (ZT)	=	95.129	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

5.0

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	=	0.36	
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

= 7.117 m

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

= 8.476 m

Abutment Cap Top Level - Foundation Level

= 6.717 m

Dimensions of Abutment Shaft

Length = 8.50 m

Width = 1.00 m

Moment of Inertia , $I_{\text{longitudinal}}$ = 0.531 m^4

Moment of Inertia , $I_{\text{transverse}}$ = 51.177 m^4

Ecm = 3.231E+07 kN/m^2

Longitudinal Direction

Force = 155.939 KN
 D = 1924.51 KN
 T = 0.2222 sec

Transverse Direction

Force = 11750.357 KN
 D = 2124.506 KN
 T = 0.0269 sec

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

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

$\frac{S_a}{g}$ = 1.403

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

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

$\frac{S_a}{g}$ = 1.403

For Soft Soil Strata, with $N < 10$

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

$\frac{S_a}{g}$ = 1.403

Medium Soil Strata

Sa/g = 2.5

Sa/g = 1.403

Seismic Coeff. In Longitudinal Direction = 0.18

Seismic Coeff. In Transverse Direction = 0.303

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.180	In Longitudinal direction
Ah	=	0.303	In Transverse direction
Av	=	0.202	In Vertical direction

For Design of Foundation

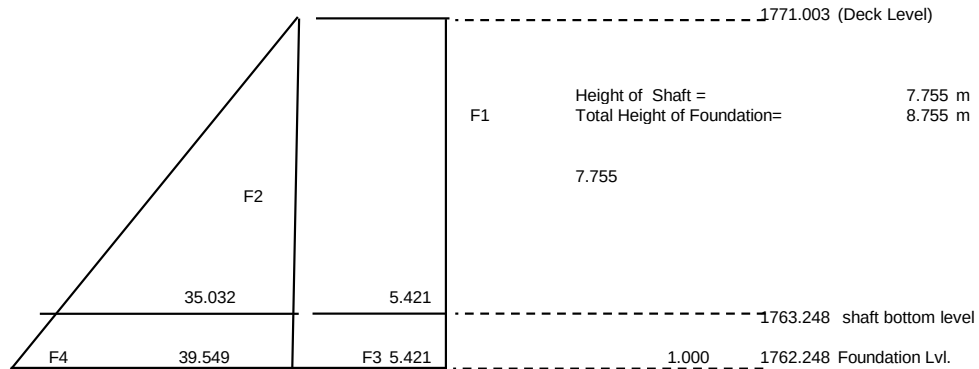
(35% increment in Seismic Coeff for Foundation as per IRC:6-2017, Clause No. 219.8)

Ah	=	0.243	In Longitudinal direction
Ah	=	0.303	In Transverse direction
Av	=	0.202	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					1763.248 m (at Shaft Base)		
@ RL					1762.248 m (at Foundation Level)		
Due to Live Load Surcharge							
Intensity for rectangular portion	=	0.226	x	20	x	1.2	= 5.421 kN/m^2
F1	=	5.421	x	7.755	x	8.500	= 357.326 kN
M1	=	357.33	x	3.88	=	1385.532 kN.m	at Shaft Bottom
F3	=	5.421	x	8.755	x	8.500	= 403.403 kN
M3	=	403.403	x	4.377	=	1765.896 kN.m	at Foundation

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)							
	=	0.226	x	20	x	7.755	= 35.032 kN/m ²
F2	=	0.5	x	35.03	x	7.755	x 8.50
	=	1154.610 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	=	1154.61	x	3.26	=	3760.679 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	0.226	x	20	x	8.755	= 39.549 kN/m ²
F4	=	0.5	x	39.55	x	8.755	x 8.50
	=	1471.580 kN					
M4	=	1471.58	x	3.68	=	5411.147 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	7.755	=	37.224 kN/m ²	
F	=	0.5	x	37.224	x	7.755	x 8.500
	=	1226.857 kN					
M	=	1226.86	x	2.585	=	3171.424 kN.m	at Shaft Bottom
Intensity for triangular portion (At Foundation level)							
	=	4.800	x	8.755	=	42.02 kN/m ²	
F	=	0.5	x	42.024	x	8.75	x 8.500
	=	1563.661 kN					
M	=	1563.66	x	2.918	=	4563.283 kN.m	at Foundation
Intensity of Passive pressure							
	=	5.249	x	20	x	0.000	= 0.000 kN/m ²
Force due to passive @ Foundation, F							
	=	0.5	x	0.000	x	0.000	x 8.50
	=	0.000 kN					

Moment due to passive @ Foundation, M
= 0.000 x

0.000 =

0.000 kN.m

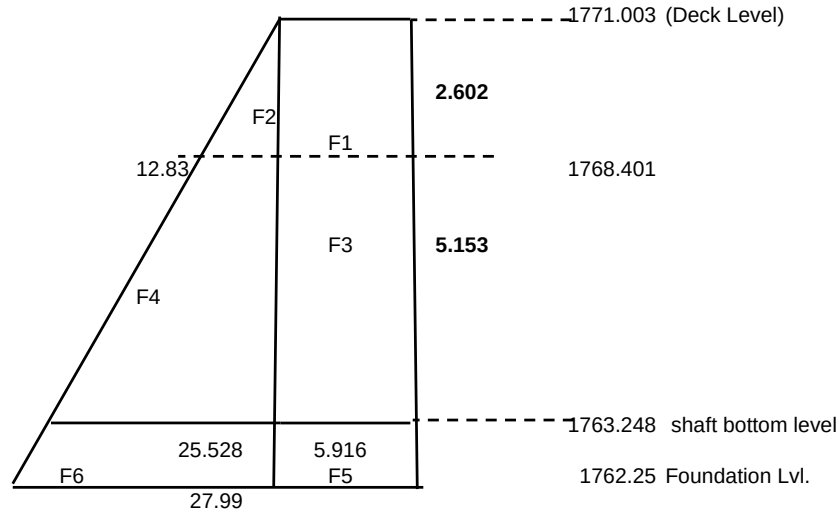
at Foundation

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	3760.679	5411.147	1154.610	1471.580
Due to Minimum Fluid Pressure	3171.424	4563.283	1226.857	1563.661
Governing of Two	3760.679	5411.147	1226.857	1563.661
Due to Live Load Surcharge	1385.532	1765.896	357.326	403.403
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****1763.2 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	7.755	x	8.500	=	389.937 kN
M1	=	389.94	x	3.88	=	1511.980 kN.m		at Shaft Bottom
F3	=	5.916	x	8.755	x	8.500	=	440.219 kN
M3	=	440.22	x	4.38	=	1927.057 kN.m		at Foundation Level

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	2.602	=	12.827 kN/m ²
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	5.153	=	12.701 kN/m ²
F2	=	0.5	x	12.83	x	2.602	x	8.50
	=	141.846 kN						
F4	=	$\left(\frac{12.83 + 25.53}{2} \right)$	x	5.15	x	8.50		
	=	839.980 kN						
Total Force	=	981.825 kN						
M2	=	141.85	x	6.25	=	885.944 kN.m		
M4	=	839.98	x	2.29	=	1925.316 kN.m		
Total Moment	=	2811.26 kN.m						at Shaft Bottom

Intensity for
triangular portion

$$\text{Upto HFL} = 0.246 \times 20 \times 2.602 = 12.827 \text{ kN/m}^2$$

$$\text{at Foundation level} = 0.246 \times 10 \times 6.153 = 15.166 \text{ kN/m}^2$$

$$F2 = 0.5 \times 12.83 \times 2.602 \times 8.50 = 141.846 \text{ kN}$$

$$F6 = \left(\frac{12.83 + 27.99}{2} \right) \times 6.15 \times 8.50 = 1067.443 \text{ kN}$$

$$\text{Total Force} = 1209.288 \text{ kN}$$

$$M2 = 141.85 \times 7.25 = 1027.790 \text{ kN.m}$$

$$M6 = 1067.44 \times 2.70 = 2877.281 \text{ kN.m}$$

$$\text{Total Moment} = 3905.07 \text{ kN.m} \quad \text{Foundation Lvl.}$$

Intensity of Passive pressure:

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

Force due to passive @ Foundation, F

$$= 0.5 \times 0.000 \times 8.50 = 0.000 \text{ kN}$$

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	2811.261	3905.071	981.825	1209.288
Due to Minimum Fluid Pressure	3171.424	4563.283	1226.857	1563.661
Governing of Two	3171.424	4563.283	1226.857	1563.661
Due to Live Load Surcharge	1511.980	1927.057	389.937	440.219
Due to Passive pressure		0.000		0.000

Earth Pressure : Seismic Dry Case**As per Clause 219.5.4 , IRC:6-2014****Seismic Zone = 5.0****Dynamic increment due to seismic force**

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

$$\begin{aligned} \alpha h &= 0.180 \\ \alpha v &= 0.202 \\ \phi &= 35.00 & 0.611 \\ \delta &= 22.50 & 0.393 \\ \alpha &= 0.00 & 0.000 \\ \beta &= 0.00 & 0.000 \end{aligned}$$

$$\begin{aligned} \alpha h &= \text{HORIZONTAL SEISMIC COEFFICIENT} \\ \alpha v &= \text{VERTICAL SEISMIC COEFFICIENT} \\ \phi &= \text{ANGLE OF INTERNAL FRICTION OF SOIL} \\ \delta &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL} \\ \alpha &= \text{ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,} \\ \beta &= \text{SLOPE OF EARTH FILL} \end{aligned}$$

$$\lambda = \tan^{-1} \frac{\alpha h}{(1 \pm \alpha v)} = \begin{matrix} 0.149 \\ 0.222 \end{matrix}$$

$$C_a = \begin{matrix} 1 & 2 \\ 0.380 & 0.300 \end{matrix}$$

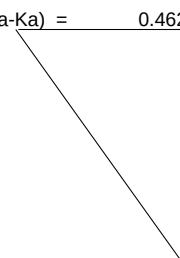
$$\begin{aligned} C_a &= 0.380 \\ K_a &= 0.226 \\ \text{Dynamic Increment} &= 0.380 \quad -0.226 \quad 0.154 \end{aligned}$$

3 Earth Pressure : DRY CASE (Seismic case)

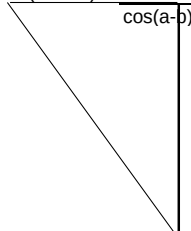
$$\begin{aligned} \text{Equivalent Live Load Surcharge height} &= 1.2 \text{ m} \\ \text{Assuming } \gamma_{\text{dry}} &= 20 \text{ kN/m}^3 \\ \gamma_{\text{water}} &= 10.00 \text{ kN/m}^3 \end{aligned}$$

Active Earth Pressure

$$3 (Ca - Ka) = 0.462$$

**Surcharge Earth Pressure**

$$2 (Ca - Ka) \frac{\cos a}{\cos(a-b)} = 0.31 \quad \text{---} \quad 1771.003 \text{ (Deck Level)}$$



--- 1763.248 (Shaft bottom level)

--- 1762.248 (Foundation Bottom level)

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

$$= 0.308 \quad \times \quad 20.00 \quad \times \quad 1.2 \quad = \quad 7.390 \text{ kN/m}^2$$

at Shaft Bottom Level

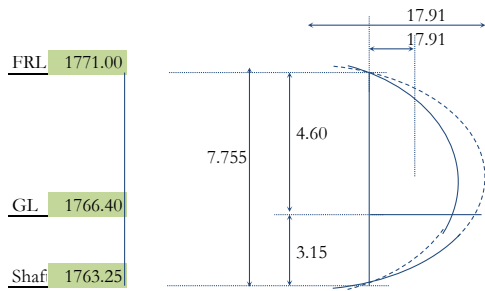
$$\begin{aligned} E1 &= 0.50 \times 7.390 \times 7.755 \times 8.500 \\ &= 243.556 \text{ kN} \end{aligned}$$

$$M1 = 243.556 \times 5.196 = 1265.479 \text{ kN.m}$$

at Foundation Bottom Level

$$\begin{aligned} E2 &= 0.50 \times 7.390 \times 7.755 \times 8.500 \\ &= 243.556 \text{ kN} \end{aligned}$$

$$M2 = 243.556 \times 6.196 = 1509.035 \text{ kN.m}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height	=	17.91 t/m ²
h	=	7.755 m
y	=	0.7245 m
L	=	8.500 m

Parabola below Water Level

p_mid_height	=	17.91 t/m ²
h	=	7.755 m
y	=	-0.7245 m
L	=	8.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	502.5	4.9
Parabola below Water Level	284.5	2.0
Total Dynamic Earth Pressure	787.0	3.9

$$E3 = 786.989 \text{ kN}$$

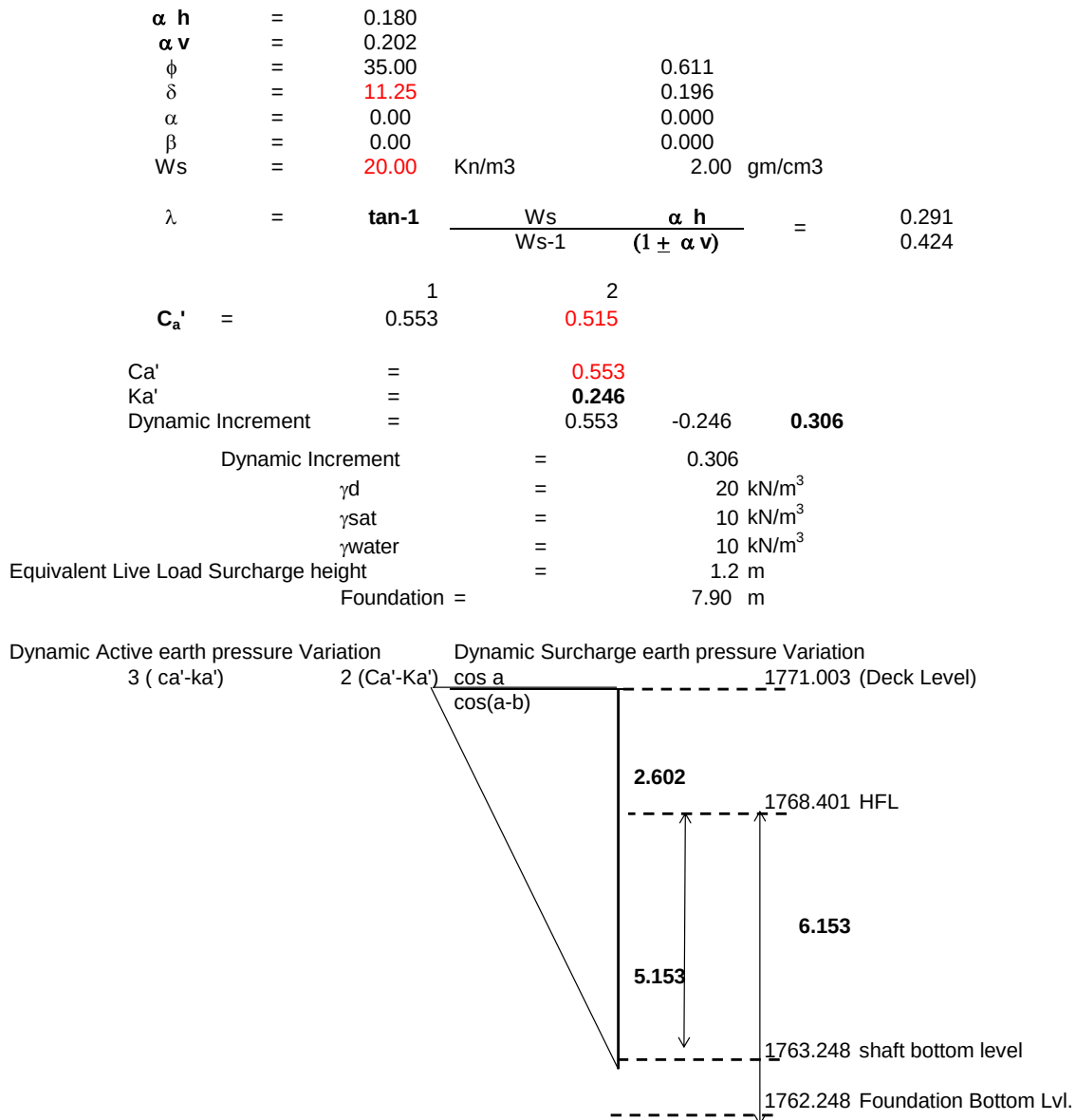
$$E4 = 786.989 \text{ kN}$$

$$M3 = 786.99 \times 3.88 = 3051.551 \text{ kN.m (Shaft bottom level)}$$

$$M4 = 786.99 \times 4.88 = 3838.541 \text{ kN.m (Foundation Bottom level)}$$

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)	3760.679	5411.147	1154.610	1471.580
Due to active Earth Pressure (dynamic Increment)	3051.551	3838.541	786.989	786.989
Total Earth Pressure	6812.231	9249.688	1941.599	2258.570
Due to Minimum Fluid Pressure	3171.424	4563.283	1226.857	1563.661
Governing of Two	3760.679	9249.688	1941.599	2258.570
Due to Live Load Surcharge (Static)	1385.532	1765.896	357.326	403.403
Due to Live Load Surcharge(Dynamic)	1265.479	1509.035	243.556	243.556
Due to Passive pressure		0.000		0.000

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

h'	=	5.153
h	=	7.755

Dynamic active earth pressure

3 (ca'-ka')	=	0.462 at deck level
3 (ca'-ka')	=	0.918 at deck level
3 (ca'-ka')h'/h	=	0.610 at HFL level

Dynamic Surcharge earth pressure

2 (Ca'-Ka') $\frac{\cos a}{\cos(a-b)}$	=	0.612 at deck level	Pressure	=	14.70 kN/m ²
	=	0.407 at HFL level	Pressure	=	9.77 kN/m ²

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	7.390 kN/m ²
Dynamic surcharge pressure in Submerge condition at HFL level	=	9.77 kN/m ²

at Shaft Bottom Level

F1	=	8.577	x	2.602	x	8.500	=	189.707 kN
M1	=	189.71	x	6.45	=	1224.366 kN.m		
F2	=	0.50	x	9.765	x	5.153	x	8.500
	=	213.858 kN						

$$M2 = 213.86 \times 3.45 = 738.348 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Abutment shaft level} &= 403.565 \text{ kN} \\ \text{Total Moment at Abutment shaft level} &= 1962.714 \text{ kN.m} \end{aligned}$$

at Foundation Bottom Level

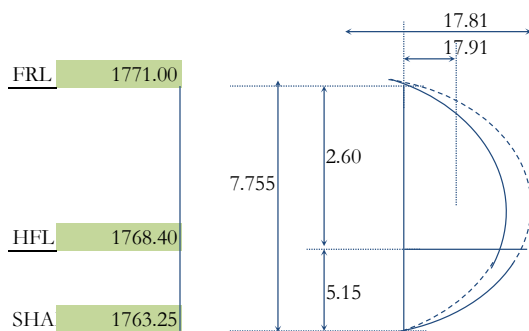
$$F3 = 8.577 \times 2.602 \times 8.500 = 189.707 \text{ kN}$$

$$M3 = 189.71 \times 7.45 = 1414.073 \text{ kN.m}$$

$$\begin{aligned} F4 &= 0.500 \times 9.765 \times 5.153 \times 8.500 \\ &= 213.858 \text{ kN} \end{aligned}$$

$$M4 = 213.86 \times 4.45 = 952.206 \text{ kN.m}$$

$$\begin{aligned} \text{Total Force at Founding level} &= 403.565 \text{ kN} \\ \text{Total Moment at Founding level} &= 2366.279 \text{ kN.m} \end{aligned}$$

Due to Dynamic Active Earth Pressure

Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 17.91 \text{ t/m}^2 \\ h &= 7.755 \text{ m} \\ y &= -1.2755 \text{ m} \\ L &= 8.500 \text{ m} \end{aligned}$$

Parabola below Water Level

$$\begin{aligned} p_{\text{mid\_height}} &= 17.81 \text{ t/m}^2 \\ h &= 7.755 \text{ m} \\ y &= 1.2755 \text{ m} \\ L &= 8.500 \text{ m} \end{aligned}$$

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	206.3	6.1
Parabola below Water Level	577.4	3.1
Total Dynamic Earth Pressure	783.7	3.9

Total Force (F1 + F2)	=	783.711 kN	at Shaft Bottom Level
Total Force (F1 + F2)	=	783.711 kN	at Foundation Bottom Level

$$M1 = 783.71 \times 3.88 = 3041.409 \text{ kN.m}$$

$$\text{Total Mome} = 3041.409 \text{ kN.m} \quad \text{at Shaft Bottom}$$

$$M1 = 783.71 \times 4.88 = 3825.120 \text{ kN.m}$$

$$\text{Total Mome} = 3825.120 \text{ kN.m} \quad \text{at Foundation Bottom Level}$$

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)	2811.261	3905.071	981.825	1209.288
Due to active Earth Pressure (Dynamic Increment)	3041.409	3825.120	783.711	783.711
Total Earth Pressure	5852.669	7730.191	1765.536	1992.999
Due to Minimum Fluid Pressure	3171.424	4563.283	1226.857	1563.661
Governing of Two	5852.669	7730.191	1765.536	1992.999
Due to Live Load Surcharge(Static)	1511.980	1927.057	389.937	440.219
Due to Live Load Surcharge (Dynamic Increment)	1962.714	2366.279	403.565	403.565
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	=	1473.50	1.35	1.35	1989.23	1989.23	
SIDL except wc	=	245.17	1.35	1.35	330.98	330.98	
WC	=	205.84	1.75	1.75	360.22	360.22	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	809.52	1.5	0.20	1214.29	161.90	(1-70RW)
CWLLmax-Reaction case	=	1926.19	1	0.20	1926.19	385.24	SV Loading
CWLLmin	=	190.48	1.5	0.20	285.71	38.10	(1-70RW)
CWLLmin	=	1493.81	1	0.20	1493.81	298.76	SV Loading
CWLLmax-Transv. Moment Case		392.35	1.5	0.20	588.52	78.47	(1-70RW)

$$\text{Braking Force} = 0.2 \times 1000 + 0.05 \times 554 = 227.7 \text{ KN}$$

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2680.42	0	134.021	134.021	-134.021	134.021	
DL+SIDL+LL-Max Reaction case	2966.13	341.55	148.307	319.082	193.243	319.082	(1-70RW)
	4459.94	0	222.997	222.997	-222.997	222.997	SV Loading
DL+SIDL+LL-Min Reaction case	3894.70	341.55	194.735	365.510	146.815	365.510	(1-70RW)
	4606.61	0	230.330	230.330	-230.330	230.330	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3268.94	341.55	163.447	334.222	178.103	334.222	

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	1924.51	2680.42	1039.23	134.021	653.637	905.212	1039.233	
DL+SIDL+LL-Max Reaction case		2718.51	1084.77	135.926	678.312	948.847	1084.773	Dry Case
DL+SIDL+LL-Min Reaction case		2842.32	1084.77	142.116	684.503	942.657	1084.773	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2758.89	1084.77	137.944	680.331	946.829	1084.773	

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	1924.51	2680.42	311.770	134.021	289.906	177.749	311.770	
DL+SIDL+LL-Max Reaction case		2718.51	357.310	135.926	314.581	221.384	357.310	Dry Case
DL+SIDL+LL-Min Reaction case		2842.32	357.310	142.116	320.771	215.194	357.310	HFL Case

DL+SIDL+LL-Max Transv. Moment case		2758.89	357.310	137.944	316.599	219.366	357.310

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	=	1473.50	1.35	1.35	1989.23	1989.23
SIDL except wc	=	245.17	1.35	1.35	330.98	330.98
WC	=	205.84	1.75	1.75	360.22	360.22
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	809.52	1.5	0.75	1214.29	607.14
CWLLmax-Transv. Moment Case	=	392.35	1.5	0.75	588.52	294.26
CWLLmin	=	190.48	1.5	0.75	285.71	142.86

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	2680.42	0.000	134.021	134.021	-134.021	134.021	
DL+SIDL+LL-Max Reaction case	2966.13	341.550	148.307	319.082	193.243	319.082	(1-70RW)
	4459.94	0.000	222.997	222.997	-222.997	222.997	SV Loading
DL+SIDL+LL-Min Reaction case	3894.70	341.550	194.735	365.510	146.815	365.510	(1-70RW)
	4606.61	0.000	230.330	230.330	-230.330	230.330	SV Loading
DL+SIDL+LL-Max Transv. Moment case	3268.94	341.550	163.447	334.222	178.103	334.222	

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	1924.51	2680.42	1402.96	134.021	835.503	1268.944	1402.965	
DL+SIDL+LL-Max Reaction case		2823.27	1448.50	141.164	865.416	1307.341	1448.505	Dry Case
DL+SIDL+LL-Min Reaction case		3287.56	1448.50	164.378	888.630	1284.127	1448.505	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2974.68	1448.50	148.734	872.986	1299.771	1448.505	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1924.51	2680.42	420.889	134.021	344.466	286.868	420.889	
DL+SIDL+LL-Max Reaction case		2823.27	466.429	141.164	374.378	325.266	466.429	Dry Case
DL+SIDL+LL-Min Reaction case		3287.56	466.429	164.378	397.593	302.051	466.429	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2974.68	466.429	148.734	381.949	317.696	466.429	

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	=	1473.50	1	1.00	1473.50	1473.50
SIDL except wc	=	245.17	1	1.00	245.17	245.17
WC	=	205.84	1	1.00	205.84	205.84
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax- Reaction case	=	809.52	1	0.20	809.52	161.90
CWLLmax- Transv. Moment Case		392.35	1	0.20	392.35	78.47
CWLLmin	=	190.48	1	0.20	190.48	38.10

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1924.51	0.000	96.225	96.225	-96.225	96.225	
DL+SIDL+LL-Max Reaction case	2114.98	227.700	105.749	219.599	121.951	219.599	(1-70RW) Dry Case
	3418.32	0.000	170.916	170.916	-170.916	170.916	SV Loading
DL+SIDL+LL-Min Reaction case	2734.03	227.700	136.701	250.551	90.999	250.551	(1-70RW) HFL Case
	3850.70	0.000	192.535	192.535	-192.535	192.535	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2316.85	227.700	115.843	229.693	111.857	229.693	

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1924.51	935.310	96.225	563.880	839.084	935.310	
DL+SIDL+LL-Max Reaction case	1962.60	980.850	98.130	588.555	882.720	980.850	Dry Case
DL+SIDL+LL-Min Reaction case	2086.41	980.850	104.321	594.745	876.529	980.850	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2002.97	980.850	100.149	590.574	880.701	980.850	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μR	Fh/2 + μR	Fh - μR	Governing Long. Force (kN)	
DL+SIDL	1924.51	280.593	96.225	236.522	184.368	280.593	
DL+SIDL+LL-Max Reaction case	1962.60	326.133	98.130	261.196	228.003	326.133	Dry Case
DL+SIDL+LL-Min Reaction case	2086.41	326.133	104.321	267.387	221.812	326.133	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2002.97	326.133	100.149	263.215	225.984	326.133	

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	=	1473.50	1.1	1.10	1620.85	1620.85
SIDL except wc	=	245.17	1.1	1.10	269.68	269.68
WC	=	205.84	1.35	1.35	277.88	277.88
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax- Reaction case	=	809.52	1.5	0.00	1214.29	0.00
CWLLmax- Transv. Moment Case		392.35	1.5	0.00	588.52	0.00
CWLLmin	=	190.48	1.5	0.00	285.71	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	2168.42	0.000	108.421	108.421	-108.421	108.421	
DL+SIDL+LL-Max Reaction case	2454.13	341.550	122.706	293.481	218.844	293.481	(1-70RW)
	3662.23	0.000	183.111	183.111	-183.111	183.111	SV Loading
DL+SIDL+LL-Min Reaction case	3382.70	341.550	169.135	339.910	172.415	339.910	(1-70RW)
	4094.60	0.000	204.730	204.730	-204.730	204.730	SV Loading
DL+SIDL+LL-Max Transv. Moment case	2756.93	341.550	137.847	308.622	203.703	308.622	

Dry Case

HFL Case

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1924.51	2168.42	1402.96	108.421	809.903	1294.544	1402.965	
DL+SIDL+LL-Max Reaction case		2168.42	1402.96	108.421	809.903	1294.544	1402.965	Dry Case
DL+SIDL+LL-Min Reaction case		2168.42	1402.96	108.421	809.903	1294.544	1402.965	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2168.42	1402.96	108.421	809.903	1294.544	1402.965	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1924.51	2168.42	420.889	108.421	318.865	312.469	420.889	
DL+SIDL+LL-Max Reaction case		2168.42	420.889	108.421	318.865	312.469	420.889	Dry Case
DL+SIDL+LL-Min Reaction case		2168.42	420.889	108.421	318.865	312.469	420.889	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2168.42	420.889	108.421	318.865	312.469	420.889	

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	=	1473.50	1	1	1	1473.50	1473.50	1473.50
SIDL except wc	=	245.17	1	1	1	245.17	245.17	245.17
WC	=	205.84	1.20	1.20	1.20	247.01	247.01	247.01
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	809.52	1	0.75	0	809.52	607.14	0.00
CWLLmax-Transv. Moment Case	=	392.35	1	0.75	0	392.35	294.26	0.00
CWLLmin	=	190.48	1	0.75	0	190.48	142.86	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)		
DL+SIDL	1965.67	0.000	98.284	98.284	-98.284	98.284		
DL+SIDL+LL-Max Reaction case	2156.15	227.700	107.807	221.657	119.893	221.657	(1-70RW)	Dry Case
	3459.48	0.000	172.974	172.974	-172.974	172.974	SV Loading	
DL+SIDL+LL-Min Reaction case	2775.20	227.700	138.760	252.610	88.940	252.610	(1-70RW)	HFL Case
	3891.86	0.000	194.593	194.593	-194.593	194.593	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	2358.02	227.700	117.901	231.751	109.799	231.751		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1965.67	0.000	98.284	98.284	-98.284	98.284	
DL+SIDL+LL-Max Reaction case	2108.53	170.775	105.426	190.814	65.349	190.814	Dry Case
DL+SIDL+LL-Min Reaction case	2572.82	170.775	128.641	214.028	42.134	214.028	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2259.93	170.775	112.997	198.384	57.778	198.384	

Normal Case: Quasi Permanent Combination

	Vertical Force (R)	Fh	μR	$Fh/2 + \mu R$	$Fh - \mu R$	Governing Long. Force (kN)	
DL+SIDL	1965.67	0.000	98.284	98.284	-98.284	98.284	
DL+SIDL+LL-Max Reaction case	1965.67	0.000	98.284	98.284	-98.284	98.284	Dry Case
DL+SIDL+LL-Min Reaction case	1965.67	0.000	98.284	98.284	-98.284	98.284	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1965.67	0.000	98.284	98.284	-98.284	98.284	

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	=	40.00	kmph	40.00	kmph
Live Load	W	=	809.52	kN	809.52	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

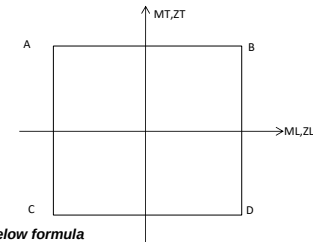
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum 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 = 1762.248 m

Properties of Footing Base:

A	=	67.150	m ²
ZL	=	88.414	m ³
ZT	=	95.129	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 = PxL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxT (kNm)
Superstructure								
Dead Load	1			1473.500	0.850	1252.475	0.000	0.000
SIDL except Wearing Course	1			245.168	0.850	208.393	0.000	0.000
Wearing Course	1			205.838	0.850	174.962	0.000	0.000
Bearing Pedestal	1	25	0.902	22.560	0.850	19.176	-0.278	-6.272
				1947.066		1655.006		-6.272
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	4.807	120.169	0.050	6.008	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.271	6.773	0.050	0.339	0.000	0.000
Bracket - Uniform portion	1	25	0.765	19.125	-0.250	-4.781	0.000	0.000
Bracket - Tapered portion	1	25	0.383	9.563	-0.200	-1.913	0.000	0.000
Cap - (uniform portion)	1	25	4.718	117.938	0.825	97.298	0.000	0.000
Cap - (corbel portion)	1	25	3.579	89.466	0.825	73.809	0.000	0.000
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.050	1.400	0.000	0.000
Approach Slab	1	25	4.463	111.563	-0.250	-27.891	0.000	0.000
				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	34.851	871.287	-1.800	-1568.317	0.000	0.000
Abutment Shaft	1	25	43.499	1087.469	0.850	924.348	0.000	0.000
Back filling over heel slab	1	20	294.547	5890.946	-1.831	-10787.062	0.000	0.000
Front Filling over toe slab	1	20	77.416	1548.326	2.607	4036.027	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	23.758	593.938	-1.414	-839.889	0.000	0.000
Toe slab	1	25	14.365	359.125	2.417	867.885	0.000	0.000
portion between heel & toe	1	25	8.500	212.500	0.850	180.625	0.000	0.000
Vertical Components of active earth pressure	1			647.689	-3.950	-2558.373	0.000	0.000
				11211.280		-9744.755		0.000
Total				13660.941		-7945.479		-6.272

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	96.225	1769.518	699.558
due to Earth pressure	1	1563.661		5411.147

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		108.421	1769.518	788.219
due to Earth pressure	1.5	2345.491		8116.721

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxL2 (kNm)
0.900	1326.150	-3.100	-4111.065
0.900	220.651	-3.100	-684.019
1.000	205.838	-3.100	-638.096
0.900	20.304	-3.100	-62.942
	1772.943		-5496.122
0.900	108.152	-3.900	-421.792
0.900	6.096	-3.900	-23.775
0.900	17.213	-4.200	-72.293
0.900	8.606	-4.150	-35.716
0.900	106.144	-3.125	-331.699
0.900	80.519	-3.125	-251.622
0.900	0.000	-3.600	0.000
0.900	0.000	-3.600	0.000
0.900	25.200	-3.900	-98.280
0.900	100.406	-4.200	-421.706
	452.336		-1656.883
0.900	784.159	-5.750	-4508.913
0.900	978.722	-3.100	-3034.038
0.900	5301.852	-5.781	-30650.670
0.900	1393.493	-1.343	-1871.874
0.900	0.000	0.000	0.000
0.900	534.544	-5.364	-2867.348
0.900	323.213	-1.533	-495.593
0.900	191.250	-3.100	-592.875
0.900	582.920	-7.900	-4605.072
	10090.152		-48626.382
	12315.431		-55779.387

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
96.23	699.56	0.00	0.00
1563.66	5411.15	0.00	0.00
1659.886	6110.705	0.000	0.000

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
108.42	788.22	0.00	0.00
2345.49	8116.72	0.00	0.00

2453.912 8904.940 2453.912 8904.940 0.000 0.000

Summary of Forces For SBC		
P	13660.941	kN
ML	-1834.774	kNm
MT	-6.272	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 = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Forces from Case :DL+SIDL		13660.941		-7945.479		-6.272
CWLL-Max. Reaction case	1	809.524	0.850	688.096	1.155	935.000
Vertical Components of LL Surcharge	1	167.095	-3.950	-660.025	0.000	0.000
Total		14637.560		-7917.409		928.728

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	219.599	1769.518	1596.485
due to Earth pressure	1	1563.661		5411.147
due to Live load surcharge	1	403.403		1765.896

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		293.481	1769.518	2133.610
due to Earth pressure	1.5	2345.491		8116.721
due to Live load surcharge	1.2	484.083		2119.075
		3123.056		12369.407

Summary of Forces For SBC		
P	14637.560	kN
ML	856.120	kNm
MT	928.728	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 = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Superstructure				1947.066		1655.006		-6.272
Substructure & Foundation -Portion 1				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	34.851	871.287	-1.800	-1568.317	0.000	0.000
Shaft above HFL	1	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1	15	43.499	652.481	0.850	554.609	0.000	0.000
Back filling above HFL over heel slab	1	20	95.103	1902.062	-1.800	-3423.712	0.000	0.000
Back filling below HFL over heel slab	1	10	201.135	2011.347	-1.846	-3712.103	0.000	0.000
Front Filling over toe slab	1	10	77.416	774.163	1.688	1306.520	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	23.758	356.363	-1.414	-503.933	0.000	0.000
Toe slab	1	15	14.365	215.475	2.417	520.731	0.000	0.000
Portion between Heel & Toe	1	15	8.500	127.500	0.850	108.375	0.000	0.000
Vertical Components of active earth pressure	1			647.689	-3.950	-2558.373	0.000	0.000
				7558.367		-9276.204		0.000
Total				10008.028		-7476.928		-6.272

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = Px eL2 (kNm)
	12315.43087		-55779.39
0.000		-3.100	0.00
0.900		-7.900	-1188.05
	12315.431		-56967.43

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
219.60	1596.49	0.00	0.00
1563.66	5411.15	0.00	0.00
403.403	1765.896	0.000	0.000
2186.66	8773.53	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
293.48	2133.61	0.00	0.00
2345.49	8116.72	0.00	0.00
484.08	2119.08	0.00	0.00
3123.056	12369.407	0.000	0.000

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = Px eL2 (kNm)
	1772.943		-5496.122
	452.336		-1656.883
0.900	784.159	-5.750	-4508.913
0.900	0.000	-3.100	0.000
0.900	587.233	-3.100	-1820.423
0.900	1711.856	-5.750	-9843.171
0.900	1810.212	-5.796	-10491.230
0.900	696.747	-2.262	-1576.282
0.900	0.000	0.000	0.000
0.900	320.726	-5.364	-1720.409
0.900	193.928	-1.533	-297.356
0.900	114.750	-3.100	-355.725
0.900	582.920	-7.900	-4605.072
	6802.530		-35218.579
	9027.809		-42371.584

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	96.225	1769.518	699.558
due to Earth pressure	1	1563.661		4563.283

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		108.421	1769.518	788.219
due to Earth pressure	1.5	2345.491		6844.924
		2453.912		7633.143

Summary of Forces For SBC

P	10008.028	KN
ML	-2214.087	kNm
MT	-6.272	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		10008.028		-7476.928		-6.272
CWLL-Min. Reaction case	1	190.476	0.850	161.904	1.155	220.000
Vertical Components of LL Surcharge	1	182.345	-3.950	-720.261	0.000	0.000
Total		10380.848		-8035.284		213.728

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	250.551	1769.518	1821.509
due to Earth pressure	1	1563.661		4563.283
due to Live load surcharge	1	440.219		1927.057

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		339.910	1769.518	2471.146
due to Earth pressure	1.5	2345.491		6844.924
due to live load surcharge	1.2	528.262		2312.469
		3213.663		11628.539

Summary of Forces For SBC

P	10380.848	KN
ML	276.565	kNm
MT	213.728	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
96.23	699.56	0.00	0.00
1563.66	4563.28	0.00	0.00
1659.89	5262.84	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
108.42	788.22	0.00	0.00
2345.49	6844.92	0.00	0.00
2453.91	7633.14	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)
0.000	9027.809		-42371.584
0.900	0.000	-3.100	0
0.900	164.110	-7.900	-1296.470
	9191.919		-43668.054

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
250.55	1821.51	0.00	0.00
1563.66	4563.28	0.00	0.00
440.219	1927.057	0.000	0.000
2254.43	8311.85	0.00	0.00

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
339.91	2471.15	0.00	0.00
2345.49	6844.92	0.00	0.00
528.262	2312.469	0.000	0.000
3213.66	11628.54	0.00	0.00

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

Seismic Effect Factor =	1	ah=	0.243	In Longitudinal direction	Weight of shaft below Ground level	=	670.013 KN
		ah=	0.303	In Transverse direction	Weight of back fill below Ground level	=	2304.843 KN
		av=	0.202	In Vertical direction			

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															
Dead Load	1			1473.500		134.000	89.333	0.850	1252.475	75.933	1770.465		0.000	0.000	1101.030
SIDL except Wearing Course	1			245.168		22.295	14.864	0.850	208.393	12.634	1771.452		0.000	0.000	205.206
Wearing Course	1			205.838		18.719	12.479	0.850	174.962	10.607	1771.003		0.000	0.000	163.883
Bearing Pedestal	1	25	0.902	22.560		2.052	1.368	0.850	19.176	1.163			-0.278	-6.272	
				1947.066		177.065	118.044		1655.006	100.337				-6.272	1470.119
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	4.807	120.169	29.201	10.928	7.285	0.050	6.008	0.364	1770.061	228.133	0.000	0.000	85.376
Dirt Wall-Tapered portion	1	25	0.271	6.773	1.646	0.616	0.411	0.050	0.339	0.021	1769.065	11.220	0.000	0.000	4.199
Bracket - Uniform portion	1	25	0.765	19.125				-0.250	-4.781						
Bracket - Tapered portion	1	25	0.383	9.563				-0.200	-1.913						
Cap - (uniform portion)	1	25	4.718	117.938	28.659	10.725	7.150	0.825	97.298	5.899	1768.816	188.217	0.000	0.000	70.438
Cap - (corbel portion)	1	25	3.579	89.466	21.740	8.136	5.424	0.825	73.809	4.475	1768.516	136.256	0.000	0.000	50.992
Cantilever Return Wall-Rectangle portion	1	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1771.003	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	0.350	0.000	0.000	1771.003	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.050	1.400				0.000	0.000	
Approach Slab	1	25	4.463	111.563				-0.250	-27.891				0.000	0.000	
				502.595	81.246	30.405	20.270		144.270	10.758		563.826		0.000	211.005
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	34.851	871.287	211.723	79.235	52.823	-1.800	-1568.317	-95.081	1766.974	1000.523	0.000	0.000	374.433
Abutment Shaft	1	25	43.499	1087.469	101.442	37.963	25.309	0.850	924.348	21.513	1767.383	520.929	0.000	0.000	194.951
Back filling over heel slab	1	20	294.547	5890.946	0.000	0.000	0.000	-1.831	-10787.062	0.000	1766.974	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	77.416	1548.326				2.607	4036.027				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	23.758	593.938				-1.414	-839.889				0.000	0.000	
Toe slab	1	25	14.365	359.125				2.417	867.885				0.000	0.000	
portion between heel & toe	1	25	8.500	212.500				0.850	180.625				0.000	0.000	
Vertical component of active earth pressure	1			609.548				-3.950	-2407.716				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			325.982				-3.950	-1287.628				0.000	0.000	
				11499.121	313.165	117.198	78.132		-10881.726	-73.569		1521.452		0.000	569.384
Total =				13948.782	394.411	324.669	216.446		-9082.450	37.527		2085.278		-6.272	2250.507
							-216.446			-37.527					

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
Superstructure								
Dead Load	0.9			1326.150	80.400	-3.100	-4111.065	-249.239
SIDL except Wearing Course	0.9			220.651	13.377	-3.100	-684.019	-41.470
Wearing Course	1.00			205.838	12.479	-3.100	-638.096	-38.685
Bearing Pedestal	0.9	25	0.9024	20.304		-3.100	-62.942	
				1772.943	106.256		-5496.122	-329.394
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	0.9	25	4.807	108.152	6.557	-3.900	-421.792	-25.572
Dirt Wall-Tapered portion	0.9	25	0.271	6.096	0.370	-3.900	-23.775	-1.441
Bracket - Uniform portion	0.9	25	0.765	17.213				
Bracket - Tapered portion	0.9	25	0.383	8.606				
Cap - (uniform portion)	0.9	25	4.718	106.144	6.435	-3.125	-331.699	-20.110
Cap - (corbel portion)	0.9	25	3.579	80.519	4.882	-3.125	-251.622	-15.255
Cantilever Return Wall-Rectangle portion	0.9	25	0.000	0.000	0.000	-3.600	0.000	0.000
Cantilever Return Wall-Triangle portion	0.9	25	0.000	0.000	0.000	-3.600	0.000	0.000
RCC Railing or Crash Barrier	0.9			25.200		-3.900	-98.280	
Approach Slab	0.9	25	4.463	100.406		-4.200	-421.706	
				452.336	18.243		-1548.875	-62.378

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
1.0	29.201	1770.061	228.133
1.0	1.646	1769.065	11.220
1.0	28.659	1768.816	188.217
1.0	21.740	1768.516	136.256
1.0	0.000	1771.003	0.000
1.0	0.000	1771.003	0.000
	81.246		563.826

Substructure & Foundation -Portion 2												
Abutment Shaft	0.9	25	43.499	978.722	18.716	-3.100	-3034.038	-58.019	1.0	83.352	1767.383	428.031
Solid Return wall	0.9	25	34.851	784.159	47.541	-5.750	-4508.913	-273.359	1.0	211.723	1766.974	1000.523
Back filling over heel slab	0.9	20	294.547	5301.852	0.000	-5.781	-30650.7	0.000	1.0	0.000	1766.974	0.000
Front Filling over toe	0.9	20	77.416	1393.493		-1.343	-1871.874					
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000					
Heel slab	0.9	25	23.758	534.544		-5.364	-2867.348					
Toe slab	0.9	25	14.365	323.213		-1.533	-495.593					
portion between heel & toe	0.9	25	8.500	191.250		-3.100	-592.875					
Vertical component of active earth pressure	0.9			582.920		-7.900	-4605.072					
Vertical component of dynamic increment of earth pressure	0.9			293.384		-7.900	-2317.730					
				10383.536	66.257		-50944.11	-331.379		295.074		1428.554
Total =				12608.814	190.756		-57989.11	-723.151		376.320		1992.380
					-190.756	723.151						

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				1772.943	106.256	0.000	-5496.12	-329.394
Forces from Substructure				10835.872	84.500	0.000	-52492.99	-393.757
CWLL-Max. Reaction case	0.00			0.00	0.000	-3.100	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-7.900	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.900	0.00	
Total =				12608.814	190.756		-57989.11	-723.151
					-190.756			723.151

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	980.850	191.789	1769.518	7130.777	1616.692
due to Substructure	1	394.411	147.603		2085.278	780.389
due to Earth pressure	1	2258.570			9249.688	
due to Live load surcharge	0.20	129.392			654.986	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1402.965	1769.518	10199.552
due to Substructure	1.5	564.480		2988.570
due to Active Earth pressure	1	1471.580		5411.147
due to dynamic Earth pressure	1.5	1180.484		5757.8112
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		4619.509		24357.081

Summary of Forces For SBC

	Downward	Upward	
P	14390.544	13938.021	KN
ML	10010.065	9918.325	kNm
MT	2577.809	2577.809	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
980.85	7130.78	0.00	0.00	0.00	0.00	191.79	1616.69
394.41	2085.28	0.00	0.00	0.00	0.00	147.60	780.39
2258.57	9249.69						
129.39	654.99						
3763.22	19120.73	0.00	0.00	0.00	0.00	339.39	2397.08

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1402.96	10199.55	0.00	0.00
564.48	2988.57	0.00	0.00
1471.580	5411.147		
1180.484	5757.811		
0.000	0.000		
0.000	0.000		
4619.51	24357.08	0.00	0.00

Summary of Restoring Forces

Vertical Load	12418.058	kN
Moment	-58712.259	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 = αh x P (kN)	FT = 0.3 x αh 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				1947.066		177.065	118.044		1655.006	100.337				-6.272	1470.119
Substructure & Foundation -Portion 1				502.595	81.246	30.405	20.270		144.270	10.758		563.826		0.000	211.005
Substructure & Foundation -Portion 2															
Solid Return wall	1		34.85	871.29	211.723	79.235	52.823	-1.80	-1568.317	-95.081	1766.97	1000.523	0.00	0.000	374.433
Shaft above HFL	1	25	0.000	0.000	0.000	0.000	0.000	0.850	0.000	0.000	1768.383	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	43.499	652.481	101.442	37.963	25.309	0.850	554.609	21.513	1767.401	522.730	0.000	0.000	195.625
Back filling above HFL over heel slab	1	20	95.103	1902.062	0.000	0.000	0.000	-1.800	-3423.712	0.000	1769.702	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	201.135	2011.347	0.000	0.000	0.000	-1.846	-3712.103	0.000	1765.575	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	77.416	774.163				1.688	1306.520				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	23.758	356.363				-1.414	-503.933				0.000	0.000	
Toe slab	1	15	14.365	215.475				2.417	520.731				0.000	0.000	
portion between heel & toe	1	15	8.500	127.500				0.850	108.375				0.000	0.000	
Vertical component of active earth pressure	1			500.904				-3.950	-1978.569				0.000	0.000	

Vertical component of dynamic increment of earth pressure	1			324.624				-3.950	-1282.264				0.000	0.000	
				7736.205	313.165	117.198	78.132		-9978.663	-73.569		1523.253		0.000	570.058
Total =				10185.866	394.411	324.669	216.446		-8179.387	37.527		2087.079		-6.272	2251.181
								-216.446		-37.527					

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				1772.943	106.256		-5496.122	-329.394
Substructure & Foundation -Portion 1				452.336	18.243		-1548.875	-62.378
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	34.8515	784.159	47.541	-5.75	-4508.913	-273.359
Shaft above HFL	0.9	25	0.000	0.000	0.000	-3.100	0.000	0.000
Shaft below HFL	0.9	15	43.499	587.233	18.716	-3.100	-1820.423	-58.019
Back filling above HFL over heel slab	0.9	20	95.103	1711.856	0.000	-5.750	-9843.171	0.000
Back filling below HFL over heel slab	0.9	10	201.135	1810.212	0.000	-5.796	-10491.230	0.000
Front Filling over toe	0.9	10	77.416	696.747		-2.262	-1576.282	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	23.758	320.726		-5.364	-1720.409	
Toe slab	0.9	15	14.365	193.928		-1.533	-297.356	
portion between heel & toe	0.9	15	8.500	114.750		-3.100	-355.725	
Vertical component of active earth pressure	0.9			450.813		-7.900	-3561.424	
Vertical component of dynamic increment of earth pressure	0.9			292.161		-7.900	-2308.075	
				6962.585	66.257		-36483.01	-331.379
Total =				9187.863	190.756		-43528.00	-723.151
					-190.756	723.151		

For Overturning or Sliding Effect

Load Factor	FL = ah x P (kN)	C.g. of Force (m)	MLs due to FL
	81.246		563.826
1.0	211.723	1766.974	1000.523
1.0	0.000	1768.383	0.000
1.0	83.352	1767.401	429.510
1.0	0.000	1769.702	0.000
1.0	0.000	1765.575	0.000
	295.074		1430.033
	376.320		1993.859

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	935.310	177.065	1769.518	6799.701	1470.119
due to Substructure	1	394.411	147.603		2087.079	781.063
due to Earth pressure	1	1992.999			7730.191	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1402.965	1769.518	10199.552
due to Substructure	1.5	564.480		2990.789
due to Active Earth pressure	1	1209.288		3905.071
due to dynamic Earth pressure	1.5	1175.567		5737.680
		4352.300		22833.092

Summary of Forces For SBC

	Downward	Upward	
P	10402.312	9969.420	kN
ML	8475.111	8400.057	kNm
MT	2244.909	2244.909	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
935.31	6799.70	0.00	0.00	0.00	0.00	177.07	1470.12
394.41	2087.08	0.00	0.00	0.00	0.00	147.60	781.06
1993.00	7730.19						
3322.72	16616.97	0.00	0.00	0.00	0.00	324.67	2251.18

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1402.96	10199.55	0.00	0.00
564.48	2990.79	0.00	0.00
1209.288	3905.071		
1175.567	5737.680		
4352.30	22833.09	0.00	0.00

Summary of Restoring Forces

Vertical Load	8997.107	kN
Moment	-44251.154	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				1947.066		177.065	118.044		1655.006	100.337				-6.272	1470.119
Forces from Substructure				8238.800	394.411	147.603	98.402		-9834.393	-62.810		2087.079		0.000	781.063
CWLL-Max. Reaction case	0.20			38.10		3.464	2.310	0.850	32.381	1.963	1772.203		1.155	44.000	34.488
Vertical component of LL Surcharge	0.20			36.47				-3.950	-144.052				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			33.43				-3.950	-132.058				0.000	0.000	
Total =				10293.862	394.411	328.133	218.755		-8423.117	39.490		2087.079		37.728	2285.669
						-218.755									-39.490

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 = PxeL2	MLS due to Fv
Forces from Superstructure				1772.943	106.256		-5496.122	-329.394
Forces from Substructure				7414.920	84.500		-38031.88	-393.757
CWLL-Max. Reaction case	0.00			0.00	0.00		-3.10	0.00
Vertical component of LL Surcharge	0.00			0.00		-3.100	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.900	0.000	0.000
Total =				9187.863	190.756		-43531.10	-723.151
				-190.756		723.151		

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	980.850	180.530	1769.518	7130.777	1504.606
due to Substructure	1	394.411	147.603		2087.079	781.063
due to Earth pressure	1	1992.999			7730.191	
due to Live load surcharge	0.20	168.757			858.667	

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
980.85	7130.78	0.00	0.00	0.00	0.00	180.53	1504.61
394.41	2087.08	0.00	0.00	0.00	0.00	147.60	781.06
1993.00	7730.19						
168.76	858.67						
3537.02	17806.71	0.00	0.00	0.00	0.00	328.13	2285.67

Horizontal Forces For Overturning or Sliding Effect

Horizontal Forces For Overturning of Sliding Effect				
	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1402.965	1769.518	10199.552
due to Substructure	1.5	564.480		2990.78922
due to Active Earth pressure	1	1209.288		3905.071
due to dynamic Earth pressure	1.5	1175.567		5737.67959
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		4352.300		22833.092

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
1402.96	10199.55	0.00	0.00
564.48	2990.79	0.00	0.00
1209.288	3905.071		
1175.567	5737.680		
0.000	0.000		
0.000	0.000		
4352.30	22833.09	0.00	0.00

Summary of Forces For SBC

	Downward	Upward	
P	10512.618	10075.107	kN
ML	9423.087	9344.108	kNm
MT	2323.397	2323.397	kNm

Summary of Restoring Forces

Vertical Load	8997.107	kN
Moment	-44254.254	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 = PxL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxL1	MTs due to FT
Superstructure				1947.066		590.218	118.044		1655.006	100.337				-6.272	4900.395
Substructure & Foundation -Portion 1				502.595	24.374	101.351	20.270		144.270	10.758		169.148		0.000	703.349
Substructure & Foundation -Portion 2				11499.121	93.949	390.660	78.132		-10881.726	-73.569		456.436		0.000	1897.947
Total =				13948.782	118.323	1082.229	216.446		-9082.450	37.527		625.584		-6.272	7501.691
						-216.446				-37.527					

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	326.133	639.297	1769.518	2370.986	5388.972
due to Substructure	1	118.323	492.010		625.584	2601.296
due to Earth pressure	1	2258.570			9249.688	
due to Live load surcharge	0.2	129.392			654.986	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		420.89	1769.518	3059.866
due to Substructure	1.5	169.34		896.571
due to Active Earth pressure	1	1471.58		5411.147
due to dynamic Earth pressure	1.5	354.15		1727.343
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		2415.96		11094.927

Summary of Forces For SBC

	Downward	Upward
P	14390.544	13938.021
ML	3790.579	3698.839
MT	8170.996	8170.996

Case 11 : DL+SIDL-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
Superstructure	1947.066	118.044	1655.006	100.337	-6.272
Substructure & Foundation -Portion 1	502.595	20.270	144.270	10.758	0.000
Substructure & Foundation -Portion 2	7736.205	78.132	-9978.663	-73.569	0.000
Total =	10185.866	216.446	-8179.387	37.527	-6.272
		-216.446		-37.527	

Horizontal Forces For SBC Calculation

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	280.59	590.22	1769.52	2039.91	4900.40
due to Substructure	1	118.32	492.01		626.12	2603.54
due to Earth pressure	1	1993.00			7730.19	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		420.889	1769.518	3059.87
due to Substructure	1.5	169.344		897.24
due to Active Earth pressure	1	1209.288		3905.07
due to dynamic Earth pressure	1.5	352.670		1721.30
		2152.192		9583.48

Summary of Forces For SBC

	Downward	Upward
P	10402.312	9969.420
ML	2254.364	2179.311
MT	7497.665	7497.665

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
326.13	2370.99	0.00	0.00	0.00	0.00	639.30	5388.97
118.32	625.58	0.00	0.00	0.00	0.00	492.01	2601.30
2258.57	9249.69						
129.39	654.99						
2832.42	12901.24	0.00	0.00	0.00	0.00	1131.31	7990.27

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
420.89	3059.87	0.00	0.00
169.34	896.57	0.00	0.00
1471.580	5411.147		
354.145	1727.343		
0.000	0.000		
0.000	0.000		
2415.96	11094.93	0.00	0.00

Summary of Restoring Forces

Vertical Load	12418.058	kN
Moment	-58712.259	kNm

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
Superstructure	1772.943	106.256		-5496.122	-329.394
Substructure & Foundation	452.336	18.243		-1548.875	-62.378
Substructure & Foundation	6962.585	66.257		-36483.006	-331.379
Total =	9187.863	190.756		-43528.003	-723.151
		-190.756			723.151

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
280.59	2039.91	0.00	0.00	0.00	0.00	590.22	4900.40
118.32	626.12	0.00	0.00	0.00	0.00	492.01	2603.54
1993.00	7730.19						
2391.92	10396.23	0.00	0.00	0.00	0.00	1082.23	7503.94

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
420.89	3059.87	0.00	0.00
169.34	897.24	0.00	0.00
1209.288	3905.071		
352.670	1721.304		
2152.19	9583.48	0.00	0.00

Summary of Restoring Forces

Vertical Load	8997.107	kN
Moment	-44251.154	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 =	10293.862	218.755	-8423.12	39.490	37.728
		-218.755		-39.490	

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	326.133	601.76602	1769.518	2370.986	5015.3544
due to Substructure	1	118.323	492.01047		626.124	2603.5421
due to Earth pressure	1	1992.999			7730.191	
due to Live load surcharge	0.2	168.757			858.667	

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		420.89	1769.518	3059.87
due to Substructure	1.5	169.34		897.24
due to Active Earth pressure	1	1209.29		3905.07
due to dynamic Earth pressure	1.5	352.67		1721.30
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		2152.19		9583.48

Summary of Forces For SBC

	Downward	Upward	
P	10512.618	10075.107	kN
ML	3202.341	3123.362	kNm
MT	7656.624	7656.624	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 (	1772.203 -		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 (	1772.203 -		0.000 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	=	$P/A + ML/ZL - MT/ZT$

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 =	9187.863	190.756		-43531.10	-723.151
		-190.756			723.151

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
326.13	2370.99	0.00	0.00	0.00	0.00	601.77	5015.35
118.32	626.12	0.00	0.00	0.00	0.00	492.01	2603.54
1993.00	7730.19						
168.76	858.67						
2606.21	11585.97	0.00	0.00	0.00	0.00	1093.78	7618.90

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
420.89	3059.87	0.00	0.00
169.34	897.24	0.00	0.00
1209.288	3905.071		
352.670	1721.304		
0.000	0.000		
0.000	0.000		
2152.19	9583.48	0.00	0.00

Summary of Restoring Forces

Vertical Load	8997.107	kN
Moment	-44254.254	kNm

Forces along Long. Axis		Forces along Trans. Axis	
FT Cos θ	MT Cos θ	FT Sin θ	MT Sin θ
0.00	0.00	0.00	0.00

0.00	0.00	0.00	0.00

	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= μP + c.A + F _p	FOS	Overturning moment	Restoring Moment = ΣP.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	13660.941	-1834.774	-6.272	224.125	182.621	224.257	182.753	224.257	182.621	2453.912	8620.802	3.51	8904.940	55779.39	6.26
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	14637.560	856.120	928.728	218.063	237.429	198.537	217.903	237.429	198.537	3123.056	8620.802	2.76	12369.407	56967.43	4.61
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	10008.028	-2214.087	-6.272	174.016	123.932	174.148	124.064	174.148	123.932	2453.912	6319.466	2.58	7633.143	42371.58	5.55
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	10380.848	276.565	213.728	153.711	159.967	149.217	155.473	159.967	149.217	3213.663	6434.343	2.00	11628.539	43668.05	3.76
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	14165.228	9089.744	2244.235	131.732	337.349	84.549	290.166	337.349	84.549	4619.509	8692.641	1.88	24357.081	58712.26	2.41
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	14390.544	10010.065	2577.809	128.185	354.620	73.989	300.424	354.620	73.989	4619.509	8692.641	1.88	24357.081	58712.26	2.41
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	10402.312	8475.111	2244.909	82.653	274.367	35.456	227.170	274.367	35.456	4352.300	6297.975	1.45	22833.092	44251.15	1.94
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	10512.618	9423.087	2323.397	74.399	287.557	25.552	238.710	287.557	25.552	4352.300	6297.975	1.45	22833.092	44254.25	1.94
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	14165.228	2870.258	7495.419	257.277	322.205	99.693	164.621	322.205	99.693	2811.095	8692.641	3.09	13186.926	58712.26	4.45
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	14390.544	3790.579	8170.996	257.325	343.071	85.538	171.284	343.071	85.538	2415.959	8692.641	3.60	11094.927	58712.26	5.29
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	10402.312	2254.364	7497.665	208.229	259.225	50.598	101.594	259.225	50.598	2152.192	6297.975	2.93	9583.478	44251.15	4.62
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	10512.618	3202.341	7656.624	200.821	273.261	39.848	112.287	273.261	39.848	2152.192	6297.975	2.93	9583.478	44254.25	4.62
								SAFE	SAFE			SAFE			SAFE

DESIGN OF FOUNDATION

Foundation Lvl = 1762.248 m

Properties of Footing Base:

A	=	67.150	m ²
ZL	=	88.414	m ³
ZT	=	95.129	m ³

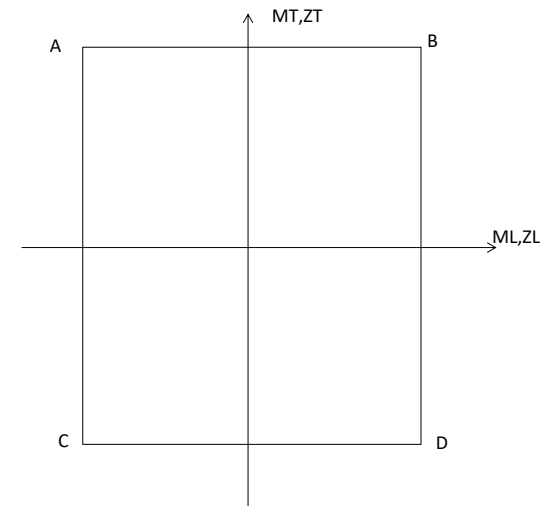
Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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	1.35			1989.225	0.850	1690.841	0.000	0.000
SIDL except Wearing Course	1.35			330.977	0.850	281.330	0.000	0.000
Wearing Course	1.75			360.216	0.850	306.183	0.000	0.000
Bearing Pedestal	1.35	25	0.902	30.456	0.85	25.888	-0.278	-8.467
				2710.873		2304.242		-8.467
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	0.050	8.111	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	0.050	0.457	0.000	0.000
Bracket - Uniform portion	1.35	25	0.765	25.819	-0.250	-6.455	0.000	0.000
Bracket - Tapered portion	1.35	25	0.383	12.909	-0.200	-2.582	0.000	0.000
Cap - (uniform portion)	1.35	25	4.718	159.216	0.825	131.353	0.000	0.000
Cap - (corbel portion)	1.35	25	3.579	120.779	0.825	99.642	0.000	0.000
Cantilever Return Wall-Rectangle p	1.35	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle port	1.35	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier or	1.35			37.800	0.050	1.890	0.000	0.000
Crash Barrier	1.35	25	4.463	150.609	-0.250	-37.652	0.000	0.000
Approach Slab	1.35	25	4.463	150.609	-0.250	-37.652	0.000	0.000
				678.504		194.765		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	34.851	1176.238	-1.800	-2117.229	0.000	0.000
Abutment Shaft	1.35	25	43.499	1468.083	0.850	1247.870	0.000	0.000
Back filling over heel slab	1.35	20	294.547	7952.777	-1.831	-14562.534	0.000	0.000
Front Filling over toe slab	1.35	20	77.416	2090.240	2.607	5448.637	0.000	0.000
Side filling between heel and toe	1.35	20	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.35	25	23.758	801.816	-1.414	-1133.850	0.000	0.000
Toe slab	1.35	25	14.365	484.819	2.417	1171.645	0.000	0.000
portion between heel & toe	1.35	25	8.500	286.875	0.850	243.844	0.000	0.000
Vertical Components of active earth pressure	1.5			971.534	-3.950	-3837.560	0.000	0.000
				15232.382		-13539.176		0.000
Total				18621.759		-11040.168		-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		134.021	1769.518	974.332
due to Earth pressure	1.5	2345.491		8116.721

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
134.02	974.33	0.00	0.00
2345.49	8116.72	0.00	0.00
2479.512	9091.053	0.000	0.000

**Summary of Forces About C.G. OF Footing**

P	18621.759	kN
ML	-1949.116	kNm
MT	-8.467	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical load**

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		18621.759		-11040.168		-8.467
CWLL-Max. Reaction case	1.5	1214.286	0.850	1032.143	1.155	1402.501
Vertical Components of LL Surcharge	1.2	200.514	-3.950	-792.030	0.000	0.000
Total		20036.559		-10800.055		1394.033

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		319.082	1769.518	2319.723
due to Earth pressure	1.5	2345.491		8116.721
due to Live load surcharge	1.2	484.083		2119.075
		3148.656		12555.519

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
319.08	2319.72	0.00	0.00
2345.49	8116.72		
484.08	2119.08		
3148.656	12555.519	0.000	0.000

Summary of Forces About C.G. OF Footing

P	20036.559	KN
ML	1755.464	kNm
MT	1394.033	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				2710.873		2304.242		-8.467
Substructure & Foundation -Portion 1				678.504		194.765		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	34.851	1176.238	-1.800	-2117.229	0.000	0.000
Shaft above HFL	1.35	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1.35	15	43.499	880.850	0.850	748.722	0.000	0.000
Back filling above HFL over heel slab	1.35	20	95.103	2567.784	-1.800	-4622.011	0.000	0.000
Back filling below HFL over heel slab	1.35	10	201.135	2715.318	-1.846	-5011.339	0.000	0.000
Front Filling over toe slab	1.35	10	77.416	1045.120	1.688	1763.801	0.000	0.000
Side filling between heel and toe	1.35	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.35	15	23.758	481.089	-1.414	-680.310	0.000	0.000
Toe slab	1.35	15	14.365	290.891	2.417	702.987	0.000	0.000
Portion between Heel & Toe	1.35	15	8.500	172.125	0.850	146.306	0.000	0.000
Vertical Components of active earth pressure	1.5			971.534	-3.950	-3837.560	0.000	0.000
				10300.949		-12906.631		0.000
Total				13690.326		-10407.624		-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		134.021	1769.518	974.332
due to Earth pressure	1.5	2345.491		6844.924

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
134.02	974.33	0.00	0.00
2345.49	6844.92		
2479.512	7819.256	0.000	0.000

Summary of Forces About C.G. OF Footing

P	13690.326	kN
ML	-2588.368	kNm
MT	-8.467	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical load**

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		13690.326		-10407.624		-8.467
CWLL-Min. Reaction case	1.5	285.714	0.850	242.857	1.155	329.999
Vertical Components of LL Surcharge	1.2	218.813	-3.950	-864.313	0.000	0.000
Total		14194.853		-11029.080		321.532

Forces due to Horizontal Load

	load factor	FL (kN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		365.510	1769.518	2657.259
due to Earth pressure	1.5	2345.491		6844.924
due to Live load surcharge	1.2	528.262		2312.469

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
365.51	2657.26	0.00	0.00
2345.49	6844.92		
528.26	2312.47		
3239.263	11814.652	0.000	0.000

Summary of Forces About C.G. OF Footing

P	14194.853	kN
ML	785.571	kNm
MT	321.532	kNm

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

Seismic Effect Factor =	1.50	α_h =	0.243 In Longitudinal direction	Weight of shaft below Ground level	=	670.013 KN
		α_h =	0.303 In Transverse direction	Weight of back fill below Ground level	=	2304.843 KN
		α_v =	0.202 In Vertical direction			

Forces due to Vertical Load

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	1.35			1989.225		200.999	134.000	0.850	1690.841	113.900	1770.465		0.000	0.000	1651.545
SIDL except Wearing Course	1.35			330.977		33.443	22.295	0.850	281.330	18.951	1771.452		0.000	0.000	307.809
Wearing Course	1.75			360.216		28.078	18.719	0.850	306.183	15.911	1771.003		0.000	0.000	245.824
Bearing Pedestal	1.35	25	0.9024	30.456		3.077	2.052	0.850	25.888	1.744			-0.278	-8.467	
				2710.873		265.598	177.065		2304.242	150.506				-8.467	2205.178

Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	43.802	16.392	10.928	0.050	8.111	0.546	1770.061	342.199	0.000	0.000	128.064
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	2.469	0.924	0.616	0.050	0.457	0.031	1769.065	16.830	0.000	0.000	6.299
Bracket - Uniform portion	1.35	25	0.765	25.819				-0.250	-6.455						
Bracket - Tapered portion	1.35	25	0.383	12.909				-0.200	-2.582						
Cap - (uniform portion)	1.35	25	4.718	159.216	42.988	16.088	10.725	0.825	131.353	8.848	1768.816	282.325	0.000	0.000	105.657
Cap - (corbel portion)	1.35	25	3.579	120.779	32.610	12.204	8.136	0.825	99.642	6.712	1768.516	204.385	0.000	0.000	76.488
Cantilever Return Wall-Rectangle p	1.35	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1771.003	0.000	0.000	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	0.000	0.000	0.000	0.000	0.000	0.350	0.000	0.000	1771.003	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.050	1.890				0.000	0.000	
Approach Slab	1.35	25	4.463	150.609				-0.250	-37.652				0.000	0.000	
				678.504	121.869	45.608	30.405		194.765	16.138		845.739		0.000	316.507
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	34.851	1176.238	317.584	118.852	79.235	-1.800	-2117.229	-142.622	1766.974	1500.784	0.000	0.000	561.649
Abutment Shaft	1.35	25	43.499	1468.083	215.479	80.640	53.760	0.850	1247.870	45.696	1767.383	1106.538	0.000	0.000	414.108
Back filling over heel slab	1.35	20	294.547	7952.777	0.000	0.000	0.000	-1.831	-14562.534	0.000	1766.974	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	77.416	2090.240				2.607	5448.637				0.000	0.000	
Side filling between heel and toe	1.35	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1.35	25	23.758	801.816				-1.414	-1133.850				0.000	0.000	
Toe slab	1.35	25	14.365	484.819				2.417	1171.645				0.000	0.000	
portion between heel & toe	1.35	25	8.500	286.875				0.850	243.844				0.000	0.000	
Vertical component of active earth pressure	1.00			609.548				-3.950	-2407.716						
Vertical component of dynamic increment of earth pressure	1.50			488.973				-3.950	-1931.441						
				15359.369	533.063	199.492	132.995		-14040.774	-96.926		2607.323		0.000	975.757
Total =				18748.746	654.932	510.698	340.465		-11541.767	69.717		3453.062		-8.467	3497.442
							-340.465			-69.717					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1402.965	265.598	1769.518	10199.552	2205.178
due to Substructure		654.932	245.100		3453.062	1292.264
due to Active Earth pressure	1.00	1471.580			5411.147	
due to dynamic increment of EP	1.50	1180.484			5757.811	

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
1402.96	10199.55	0.00	0.00	0.00	0.00	265.60	2205.18
654.93	3453.06	0.00	0.00	0.00	0.00	245.10	1292.26
1471.58	5411.15						
1180.48	5757.81						
4709.96	24821.57	0.00	0.00	0.00	0.00	510.70	3497.44

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	19089.211	18408.281	KN
ML	13349.523	13210.089	kNm
MT	3488.975	3488.975	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Forces due to Vertical Load**

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				2710.873		265.598	177.065		2304.242	150.506				-8.467	2205.178
Forces from Substructure				16037.873	654.932	245.100	163.400		-13846.009	-80.788		3453.062		0.000	1292.264
CWLL-Max. Reaction case	0.75			607.14		82.820	55.213	0.850	516.072	46.931	1772.203		1.155	701.250	824.474
Vertical component of LL Surcharge	0.20			33.419				-3.950	-132.005						
Vertical component of dynamic increment LL Surcharge	1.50			151.326				-3.950	-597.738						
Total =				19540.634	654.932	593.518	395.679		-11755.438	116.649		3453.062		692.783	4321.916
								-395.679							

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1448.505	348.418	1769.518	10530.628	3029.652
due to Substructure		654.932	245.100		3453.062	1292.264
due to Active Earth pressure	1	1471.580			5411.147	
due to dynamic increment of EP	1.50	1180.484			5757.811	
due to Live load surcharge	0.20	80.681			353.179	
due to dynamic increment of Surcharge	1.50	365.334			2263.552	

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
1448.50	10530.63	0.00	0.00	0.00	0.00	348.42	3029.65
654.93	3453.06	0.00	0.00	0.00	0.00	245.10	1292.26
1471.58	5411.15						
1180.48	5757.81						
80.68	353.18						
365.33	2263.55						
5201.52	27769.38	0.00	0.00	0.00	0.00	593.52	4321.92

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	19936.313	19144.955	kN
ML	16130.590	15897.293	kNm
MT	5014.699	5014.699	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Forces due to Vertical Load**

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				2710.873		265.598	177.065		2304.242	150.506				-8.467	2205.178
Substructure & Foundation -Portion 1				678.504	121.869	45.608	30.405		194.765	16.138		845.739		0.000	316.507
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	34.851	1176.238	317.584	118.852	79.235	-1.800	-2117.229	-142.622	1766.974	1500.784	0.000	0.000	561.649
Shaft above HFL	1.35	25	0.000	0.000	0.000	0.000	0.000	0.850	0.000	0.000	1768.383	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	15	43.49875	880.850	56.926	21.304	14.203	0.850	748.722	12.072	1767.401	293.340	0.000	0.000	109.779
Back filling above HFL over heel slab	1.35	20	95.1031	2567.784	0.000	0.000	0.000	-1.800	-4622.011	0.000	1769.702	0.000	0.000	0.000	0.000
Back filling below HFL over heel slab	1.35	10	201.13465	2715.318	0.000	0.000	0.000	-1.846	-5011.339	0.000	1765.575	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	10	77.416	1045.120				1.688	1763.801				0.000	0.000	
Side filling between heel and toe	1.35	10	0.000	0.000				0.000	0.000				0.000	0.000	

Heel slab	1.35	15	23.758	481.089				-1.414	-680.310			0.000	0.000	
Toe slab	1.35	15	14.365	290.891				2.417	702.987			0.000	0.000	
portion between heel & toe	1.35	15	8.500	172.125				0.850	146.306			0.000	0.000	
Vertical component of active earth pressure	1.00			500.904				-3.950	-1978.569					
Vertical component of dynamic increment of earth pressure	1.50			486.936				-3.950	-1923.396					
				10317.254	374.510	140.156	93.437		-12971.036	-130.550		1794.124	0.000	671.428
Total =				13706.631	496.379	451.362	300.908		-10472.029	36.093		2639.863	-8.467	3193.113
							-300.908							

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1402.965	265.598	1769.518	10199.552	2205.178
due to Substructure		496.379	185.764		2639.863	987.935
due to Active Earth pressure	1.00	1209.288			3905.071	
due to dynamic increment of EP	1.50	1175.567			5737.680	

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
1402.96	10199.55	0.00	0.00	0.00	0.00	265.60	2205.18
496.38	2639.86	0.00	0.00	0.00	0.00	185.76	987.94
1209.29	3905.07						
1175.57	5737.68						
4284.20	22482.17	0.00	0.00	0.00	0.00	451.36	3193.11

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	14007.54	13405.72	KN
ML	12046.23	11974.04	kNm
MT	3184.65	3184.65	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 = 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				2710.873		265.598	177.065		2304.242	150.506				-8.467	2205.178
Forces from Substructure				10995.758	496.379	185.764	123.842		-12776.271	-114.412		2639.863		0.000	987.935
CWLL-Min. Reaction case	0.75			142.86		19.487	12.991	0.850	121.428	11.043	1772.203		1.155	165.000	193.993
Vertical component of LL Surcharge	0.20			36.469				-3.950	-144.052						
Vertical component of dynamic increment LL Surcharge	1.50			250.743				-3.950	-990.435						
Total =				14136.700	496.379	470.849	313.899		-11485.088	47.136		2639.863		156.532	3387.106
							-313.899								

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1448.505	285.085	1769.518	10530.628	2399.171
due to Substructure		496.379	185.764		2639.863	987.935
due to Active Earth pressure	1	1209.288			3905.071	
due to dynamic increment of EP	1.50	1175.567			5737.680	
due to Live load surcharge	0.20	88.044			385.411	

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
1448.50	10530.63	0.00	0.00	0.00	0.00	285.09	2399.17
496.38	2639.86	0.00	0.00	0.00	0.00	185.76	987.94
1209.29	3905.07						
1175.57	5737.68						
88.04	385.41						

due to dynamic increment of Surchage	1.50	605.347			3549.418	

605.35	3549.42						
5023.13	26748.07	0.00	0.00	0.00	0.00	470.85	3387.11

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	14450.599	13822.801	kN
ML	15310.120	15215.848	kNm
MT	3543.639	3543.639	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Forces due to Vertical Load**

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				2710.873		885.327	177.065		2304.242	150.506				-8.467	7350.593
Substructure & Foundation -Portion 1				678.504	36.561	152.026	30.405		194.765	16.138		253.722		0.000	1055.024
Substructure & Foundation -Portion 2				15359.369	159.919	664.974	132.995		-14040.774	-96.926		782.197		0.000	3252.524
Total =				18748.746	196.480	1702.327	340.465		-11541.767	69.717		1035.919		-8.467	11658.141
							-340.465			-69.717					

Forces due to Horizontal Load

	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	420.889	885.327	1769.518	3059.866	7350.593
due to Substructure	196.480	817.000		1035.919	4307.548
due to Active Earth pressure	1471.580			5411.147	
due to dynamic increment of EP	354.145			1727.343	

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
420.89	3059.87	0.00	0.00	0.00	0.00	885.33	7350.59
196.48	1035.92	0.00	0.00	0.00	0.00	817.00	4307.55
1471.58	5411.15						
354.15	1727.34						
2443.09	11234.28	0.00	0.00	0.00	0.00	1702.33	11658.14

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	19089.211	18408.281	kN
ML	-237.774	-377.209	kNm
MT	11649.674	11649.674	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = Px eL1	MLs due to Fv	MT = Px eT
Total =				19540.634	395.679	-11755.438	116.649	692.783
					-395.679		-116.649	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		466.429	1161.394	1769.518	3390.941	10098.8392
due to Substructure		196.480	817.000		1035.919	4307.54771
due to Earth pressure		1471.580			5411.147	
due to dynamic increment of EP		354.145			1727.343	
due to Live load surcharge		80.681			353.179	
due to dynamic increment of Surcharge		109.600			679.066	

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	19936.313	19144.955	KN
ML	958.806	725.509	kNm
MT	15099.170	15099.170	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	Vertical Load(P) kN.	Fv = 0.3 x av x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure				2710.873	177.065	2304.242	150.506	-8.467
Substructure & Foundation -Portion 1				678.504	30.405	194.765	16.138	0.000
Substructure & Foundation -Portion 2				10317.254	93.437	-12971.036	-130.550	0.000
Total =				13706.631	300.908	-10472.029	36.093	-8.467

-300.908

-36.093

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		420.889	885.327	1769.518	3059.866	7350.59308
due to Substructure		148.914	619.212		791.959	3293.117
due to Active Earth pressure	1.00	1209.288			3905.071	
due to dynamic increment of EP	1.50	352.670			1721.304	

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	14007.539	13405.723	KN
ML	-957.736	-1029.922	kNm
MT	10635.243	10635.243	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
466.43	3390.94	0.00	0.00	0.00	0.00	1161.39	10098.84
196.48	1035.92	0.00	0.00	0.00	0.00	817.00	4307.55
1471.58	5411.15						
354.15	1727.34						
80.68	353.18						
109.60	679.07						
2678.92	12597.60	0.00	0.00	0.00	0.00	1978.39	14406.39

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
420.89	3059.87	0.00	0.00	0.00	0.00	885.33	7350.59
148.91	791.96	0.00	0.00	0.00	0.00	619.21	3293.12
1209.29	3905.07						
352.67	1721.30						
2131.76	9478.20	0.00	0.00	0.00	0.00	1504.54	10643.71

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Forces due to Vertical Load**

Lloads	Vertical Load(P) kN.	$F_v = 0.3 \times \alpha_v \times P$ (kN)	$ML = P \times e_{L1}$	MLs due to F_v	$MT = P \times e_T$
Total =	14136.700	313.899	-11485.088	47.136	156.532
		-313.899		-47.136	

Forces due to Horizontal Load

	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	466.429	950.284	1769.518	3390.941	7997.23754
due to Substructure	148.914	619.212		791.959	3293.11715
due to Earth pressure	1209.288			3905.071	
due to dynamic increment of EP	352.670			1721.304	
due to Live load surcharge	88.044			385.411	
due to dynamic increment of Surcharge	181.604			1064.825	

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
466.43	3390.94	0.00	0.00	0.00	0.00	950.28	7997.24
148.91	791.96	0.00	0.00	0.00	0.00	619.21	3293.12
1209.29	3905.07						
352.67	1721.30						
88.04	385.41						
181.60	1064.83						
2446.95	11259.51	0.00	0.00	0.00	0.00	1569.50	11290.35

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	14450.599	13822.801	KN
ML	-178.439	-272.711	kNm
MT	11446.887	11446.887	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.50	x	0.00	=	0.000 KN	
Transverse Moment due to C.F.	=	0.000	x (	1772.203 -	1762.248)	=	0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.75	x	0.00	=	0.00 KN	
Transverse Moment due to C.F.	=	0.000	x (	1772.203 -	1762.248)	=	0.000 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	=	$P/A + ML/ZL - MT/ZT$

Forces along Long. Axis		Forces along Trans. Axis	
FT Cos θ	MT Cos θ	FT Sin θ	MT Sin θ
0.00	0.00	0.00	0.00

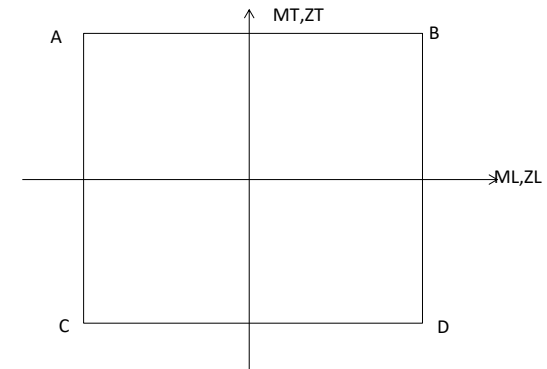
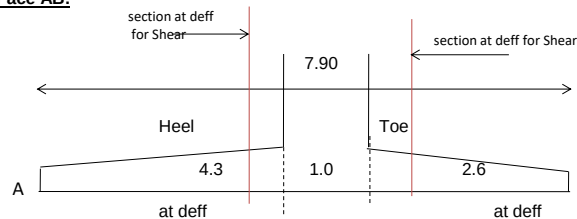
Normal

Seismic

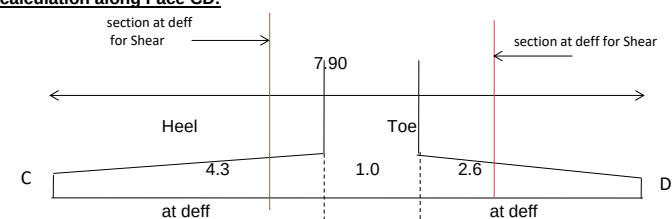
0.00	0.00	0.00	0.00

Summary of Design Base Pressure

LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D
	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²
Normal Dry Case							
Case 1 : DL+SIDL-Normal Dry Case	18621.759	-1949.116	-8.467	299.272	255.182	299.450	255.360
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	20036.559	1755.464	1394.033	293.184	332.894	263.876	303.586
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	13690.326	-2588.368	-8.467	233.063	174.512	233.241	174.690
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	14194.853	785.571	321.532	205.885	223.655	199.125	216.895
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	19089.211	13349.523	3488.975	169.965	471.942	96.612	398.589
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	19936.313	16130.590	5014.699	167.163	532.050	61.734	426.621
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	14007.539	12046.231	3184.646	105.830	378.326	38.876	311.371
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	14450.599	15310.120	3543.639	79.286	425.613	4.784	351.112
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	19089.211	-237.774	11649.674	409.428	404.049	164.505	159.126
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	19936.313	958.806	15099.170	444.771	466.460	127.325	149.014
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	14007.539	-957.736	10635.243	331.231	309.566	107.635	85.970
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	14450.599	-178.439	11446.887	337.547	333.511	96.887	92.851

**Pressure calculation along Face AB:**

Case : 1	299.272	280.380	275.27	269.692	264.586	255.182
Case : 2	293.184	310.199	314.80	319.825	324.424	332.894
Case : 3	233.063	207.975	201.19	193.782	187.001	174.512
Case : 4	205.885	213.499	215.56	217.807	219.865	223.655
Case : 5	169.965	299.356	334.33	372.557	407.533	471.942
Case : 6	167.163	323.511	365.77	411.961	454.223	532.050
Case : 7	105.830	222.589	254.15	288.643	320.205	378.326
Case : 8	79.286	227.681	267.79	311.632	351.745	425.613

Pressure calculation along Face CD:

Case : 1	299.450	280.558	275.45	269.870	264.764	255.360
Case : 2	263.876	280.891	285.49	290.517	295.116	303.586
Case : 3	233.241	208.153	201.37	193.960	187.179	174.690
Case : 4	199.125	206.739	208.80	211.047	213.105	216.895
Case : 5	96.612	226.004	260.98	299.205	334.180	398.589
Case : 6	61.734	218.081	260.34	306.532	348.794	426.621
Case : 7	38.876	155.635	187.20	221.689	253.251	311.371
Case : 8	4.784	153.179	193.29	237.130	277.243	351.112

Case : 9	409.428	407.123	406.50	405.820	405.197	404.049
Case : 10	444.771	454.064	456.58	459.321	461.833	466.460
Case : 11	331.231	321.948	319.44	316.696	314.187	309.566
Case : 12	337.547	335.817	335.35	334.839	334.371	333.511

Average MAX Base Pressure for Design of Heel Slab-along Face AB	=	450.673 kN/m ²
Average MIN Base Pressure for Design of Heel Slab-along Face AB	=	173.540 kN/m ²
Average MAX Base Pressure for Design of Toe Slab-along Face AB	=	472.006 kN/m ²
Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	461.833 kN/m ²
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	454.064 kN/m ²

Case : 9	164.505	162.200	161.58	160.896	160.273	159.126
Case : 10	127.325	136.618	139.13	141.876	144.388	149.014
Case : 11	107.635	98.352	95.84	93.101	90.591	85.970
Case : 12	96.887	95.158	94.69	94.179	93.712	92.851

Average MAX Base Pressure for Design of Heel Slab-along Face CD	=	287.451 kN/m ²
Average MIN Base Pressure for Design of Heel Slab-along Face CD	=	95.789 kN/m ²
Average Base Pressure for Design of Toe Slab-along Face CD	=	366.576 kN/m ²
Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	348.794 kN/m ²
Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	280.891 kN/m ²

Calculation of Moment and Shear Force Along Traffic Direction:

Case 1 : Maximum Base Pressure Case (Dry Case)

Heel Slab - Maximum Moment Calculation

Max Average Base Pressure for Design of Heel Slab	=	450.673 kN/m ²							
Upward moment due to Base pressure	=	4166.474 kNm/m							
		factor							
Downward moment due to backfill	=	1.35 x	294.547	/	8.500	x	20	x	2.150
	=	2011.585 kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	23.758	/	8.500	x	25	x	1.764
	=	166.410 kNm/m							

Net Moment at face of shaft	=	4166.474	-2011.585	-166.410	=	1988.479 kNm/m	Tension at Bottom of Heel Slab

Case 2 : Minimum Base Pressure Case (HFL Case)**Heel Slab - Maximum Moment Calculation**

Min Average Base Pressure for Design of Heel Slab	=	95.789	kN/m ²							
Upward moment due to Base pressure	=	885.565	kNm/m							
Downward moment due to backfill	=	factor								
	=	1.35 x	294.547	/	8.500	x	10	x	2.150	
Downward moment due to self weight of Heel slab	=	1005.792	kNm/m							
	=	1.35 x	23.758	/	8.500	x	15	x	1.764	
	=	99.846	kNm/m							

Net Moment at face of shaft	=	885.565	-1005.792	-99.846	=	-220.073	kNm/m	Tension at Top of Heel Slab	

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	0.851	m							
effective depth at critical section	=	0.766	m							
Base pressure at deff from face of wall	=	454.064	kN/m ²							
Shear Force due to upward pressure at deff from face of wall	=	449.417 x	3.385 x	8.500	=	12930.857	KN			
		factor								
Downward Force due to backfill	=	1.35 x	294.547 x	20	=	7952.777	KN			
Downward Force due to self weight of Heel slab	=	1.35 x	23.758 x	25	=	801.816	KN			
Net Shear Force	=	12930.857	-7952.777	-801.816	=	4176.264	KN			
Net Shear Force / unit meter	=	4176.264	/	8.500	=	491.325	KN/m			

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	472.006	kN/m ²							
Upward moment due to Base pressure	=	1595.379	kNm/m							
Downward moment due to self weight of Toe slab	=	1.35 x	14.365	/	8.500	x	25	x	1.067	
	=	60.8400	kNm/m							

Net Moment at face of shaft	=	1595.379	-60.840	=	1534.539	kNm/m	Tension at Bottom of Toe Slab	

Toe Slab - Shear Calculation at deff from Face of Wall

For shear, critical section is assumed to be located at a distance equal to effective depth from face of wall

Depth of slab at critical section	=	0.754	m							
effective depth at critical section	=	0.666	m							
Base pressure at deff from face of wall	=	461.833	kN/m ²							
upward shear force due to base pressure	=	464.147 x	1.685 x	8.500	=	6647.738	KN			
C.g. Of base pressure	=	0.722	m							
moment due to upward pressure at critical section	=	4796.917	kNm							
tanβ	=	0.269								
reduction in shear force (V _{cod})	=	M tanβ	=	1713.622	KN					
		d								
Downward force due to self weight of toe slab	=	1.35 x	0.527 x	1.685 x	8.500 x	25				
	=	254.660	KN							
Net Shear Force at deff	=	6647.738	-	254.660	-	1713.622	=	4679.457	KN	
Net Shear Force / unit meter	=	4679.457	/	8.500	=	550.524	KN/m			

Design Input :

Design length	=	1000	mm
Clear Cover For Foundation	=	75	mm
Grade of Concrete for Footing	=	M 35	
fck	=	35.00	N/mm ²

	f_{ctm}	=	2.77 N/mm ²	
Grade of Reinforcement Steel		=	Fe 500D	(HYSD Bars)
	f_y or f_{yk}	=	500.00 N/mm ²	
	f_{yd}	=	434.78 N/mm ²	($f_y/1.15$)
	E_s	=	200000.00 N/mm ²	

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	Toe Slab
Ultimate bending moment, M_u (kNm/m)	=	220.07	1534.54
Effective depth required (dreq) (mm)	=	194.04	512.39
Effective depth provided (dpro) (mm)	=	915.00	915.00
Check for provided depth	=	SAFE	SAFE
$R = M_u/(b d^2)$	=	0.26	1.83
Total depth provided (mm)	=	1000.00	1000.00
Limiting depth of neutral axis (mm)	=	564.43	564.43
Actual depth of neutral axis (mm)	=	88.89	169.38
Check for Neutral axis depth	=	OK	OK
Lever arm (z), mm	=	878.02	844.54
Moment of Resistance w.r.to steel	=	983.42	1802.44
Check for Moment Capacity	=	SAFE	SAFE
Ast reqd (mm ² / m)	=	609.63	4503.55
cl. 16.6.1 (2) of IRC :112-2011			
$A_{s,min} = 0.26 f_{ctm} b_t d / f_{yk} \geq 0.0013 b_t d$	=	1318.57	1318.57
Governing Ast (mm ² / m)	=	1318.57	4503.55
Tension Reinforcement			
Dia (mm)	=	20.00	25.00
Spacing (mm)	=	200.00	200.00
+ Dia (mm)	=	16.00	25.00
Spacing (mm)	=	200.00	200.00
Ast provided (mm ² / m)	=	2576.11	4908.74
Check for Ast provided	=	OK	OK
As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement			
Secondary Reinforcement (mm ² /m)	=	515.22	981.75
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	100.00
Ast provided (mm ² /m)	=	565.49	1130.97
Check for Ast provided	=	OK	OK

Shear Reinforcement Calculation:

		Along Traffic Direction		
		Heel Slab	Toe Slab	
Ultimate Shear Force (V_{Ed})	=	491.325	550.524	kN/m
Ast provided	=	2576.106	4908.74	mm ² /m
Depth of slab at critical section	=	851.047	753.654	mm
Effective depth at critical section	=	766.047	666.154	mm
Percentage of steel provided (ρ_1)	=	0.0030	0.0065	
cl. 10.3.1 of IRC :112-2011				
$\rho_1 = A_s/(b_w d) \leq 0.02$	=	OK	OK	
Actual shear stress= $v_{ED} = (V_{Ed}/b*0.9d)$	=	0.713	0.918	N/mm ²
Max shear capacity, $0.135 f_{ck}(1-f_{ck}/310)$	=	4.192	4.192	N/mm ²
Depth Check for Shear Resistance	=	SAFE	SAFE	

cl. 10.3.2(2) Eq. 10.2 of IRC :112-2010					
$K = 1 + \sqrt{200/d} \leq 2.0$	=	1.511	1.548		
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010					
$V_{min} = 0.031 K^{3/2} f_{ck}^{1/2}$	=	0.341	0.353		N/mm ²
$0.12 K (80 \rho_1 f_{ck})^{0.33}$	=	0.367	0.484		N/mm ²
$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 f_{cd}$	=	0.000	0.000		N/mm ²
cl. 10.3.2(2) Eq. 10.1 of IRC :112-2011					
$V_{Rd,c} = [0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}]b_w d$ subjected to minimum ($V_{min} + 0.15 \sigma_{cp}$) $b_w d$	=	335.854	443.068		kN
Check for Shear Reinforcement		Provide Shear Reinf.	Provide Shear Reinf.		
Balance Shear Force= $V_{Rd,s} = V_{Ed} - V_{Rd,c}$	=	155.471	107.457		kN/m
b	=	8.500	8.500		m
Total Shear Force	=	1321.501	913.381		kN
$\theta = 0.5 \times \sin^{-1} [V_{Ed} / (0.18 f_{ck} (1 - f_{ck}/250))]$	=	3.779	4.879		
$\cot \theta = (< 1 \cot \theta < 2.5)$	=	2.500	2.500		
$f_{ywd} = 0.8 \times f_y / 1.15$	=	347.826	347.826		N/mm ²
Provide Shear Reinforcement					
Legged	=	16	16		
Dia	=	12	12		mm
Area of Shear Reinf, A_{sw}	=	1809.557	1809.557		mm ²
$z = 0.9 \times d$	=	689.442	599.538		mm
Spacing of shear Reinforcement required					
$S = A_{sw} z^* f_{ywd} \cot \theta / V_{Rd}$	=	820.927	1032.855		mm
As per Clause 10.3.3.5 of IRC:112-2011					
$A_{sw} / (b S) = \rho_{w,min} = (0.072 f_{ck}^{0.5}) / f_{yk}$	=	0.001	0.001		
Spacing of shear Reinforcement required	=	249.895	249.895		mm
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011					
$S_{max} = 0.75 d$	=	574.535	499.615		mm
Governing Spacing of Shear Reinf.	=	249.895	249.895		mm
Provided Spacing of Shear Reinf.	=	200	200		mm

SLS CHECK OF FOUNDATION

Foundation Lvl = 1762.248 m

Properties of Footing Base:

A	=	67.150	m ²
ZL	=	88.414	m ³
ZT	=	95.129	m ³

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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			1473.500	0.850	1252.475	0.000	0.000
SIDL except Wearing Course	1.00			245.168	0.850	208.393	0.000	0.000
Wearing Course	1.20			247.005	0.850	209.954	0.000	0.000
Bearing Pedestal	1.00	25	0.902	22.560	0.850	19.176	-0.278	-6.272
				1988.233		1689.998		-6.272
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	4.807	120.169	0.050	6.008	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.271	6.773	0.050	0.339	0.000	0.000
Bracket - Uniform portion	1.00	25	0.765	19.125	-0.250	-4.781	0.000	0.000
Bracket - Tapered portion	1.00	25	0.383	9.563	-0.200	-1.913	0.000	0.000
Cap - (uniform portion)	1.00	25	4.718	117.938	0.825	97.298	0.000	0.000
Cap - (corbel portion)	1.00	25	3.579	89.466	0.825	73.809	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	0.000	0.000	0.350	0.000	0.000	0.000
Cantilever Return Wall-Triangle portion	1.00	25	0.000	0.000	0.350	0.000	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.050	1.400	0.000	0.000
Approach Slab	1.00	25	4.463	111.563	-0.250	-27.891	0.000	0.000
				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	34.851	871.287	-1.800	-1568.317	0.000	0.000
Abutment Shaft	1.00	25	43.499	1087.469	0.850	924.348	0.000	0.000
Back filling over heel slab	1.00	20	294.547	5890.946	-1.831	-10787.062	0.000	0.000
Front Filling over toe slab	1.00	20	77.416	1548.326	2.607	4036.027	0.000	0.000
Side filling between heel and toe	1.00	20	0.000	0.000	0.000	0.000	0.000	0.000

Heel slab	1.00	25	23.758	593.938	-1.414	-839.889	0.000	0.000
Toe slab	1.00	25	14.365	359.125	2.417	867.885	0.000	0.000
portion between heel & toe	1.00	25	8.500	212.500	0.850	180.625	0.000	0.000
Vertical Components of active earth pressure	1.00			647.689	-3.950	-2558.373	0.000	0.000
				11211.280		-9744.755		0.000
Total				13702.109		-7910.487		-6.272

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		98.284	1769.518	714.522
due to Earth pressure	1.00	1563.661		5411.147

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
98.28	714.52	0.00	0.00
1563.66	5411.15		
1661.944	6125.670	0.000	0.000

Summary of Forces

P	13702.109	KN
ML	-1784.818	kNm
MT	6.272	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical load**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = Px eL1 (kNm)	Trans. Ecc (eT) (m)	MT = Px eT (kNm)
Forces from Case :DL+SIDL		13702.109		-7910.487		-6.272
CWLL-Max. Reaction case	1.00	809.524	0.850	688.096	1.155	935.000
Vertical Components of LL Surcharge	0.80	133.676	-3.950	-528.020	0.000	0.000
Total		14645.309		-7750.412		928.728

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		221.657	1769.518	1611.450
due to Earth pressure	1.00	1563.661		5411.147
due to Live load surcharge	0.80	322.722		1412.717

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
221.66	1611.45	0.00	0.00
1563.66	5411.15		
322.72	1412.72		
2108.040	8435.314	0.000	0.000

Summary of Forces

P	14645.309	kN
ML	684.902	kNm
MT	928.728	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1988.233		1689.998		-6.272
Substructure & Foundation -Portion 1				502.595		144.270		0.000
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	34.851	871.287	-1.800	-1568.317	0.000	0.000
Shaft above HFL	1.00	25	0.000	0.000	0.850	0.000	0.000	0.000
Shaft below HFL	1.00	15	43.499	652.481	0.850	554.609	0.000	0.000
Back filling above HFL over heel slab	1.00	20	95.103	1902.062	-1.800	-3423.712	0.000	0.000
Back filling below HFL over heel slab	1.00	10	201.135	2011.347	-1.846	-3712.103	0.000	0.000
Front Filling over toe slab	1.00	10	77.416	774.163	1.688	1306.520	0.000	0.000
Side filling between heel and toe	1.00	10	0.000	0.000	0.000	0.000	0.000	0.000
Heel slab	1.00	15	23.758	356.363	-1.414	-503.933	0.000	0.000
Toe slab	1.00	15	14.365	215.475	2.417	520.731	0.000	0.000
Portion between Heel & Toe	1.00	15	8.500	127.500	0.850	108.375	0.000	0.000
Vertical Components of active earth pressure	1.00			647.689	-3.950	-2558.373	0.000	0.000
				7558.367		-9276.204		0.000
Total				10049.195		-7441.935		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (kN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		98.284	1769.518	714.522
due to Earth pressure	1.00	1563.661		4563.283

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
98.28	714.52	0.00	0.00
1563.66	4563.28		
1661.944	5277.805	0.000	0.000

Summary of Forces

P	10049.195	kN
ML	-2164.131	kNm
MT	6.272	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical load**

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		10049.195		-7441.935		-6.272
CWLL-Min. Reaction case	1.00	190.476	0.850	161.904	1.155	220.000
Vertical Components of LL Surcharge	0.80	145.876	-3.950	-576.209	0.000	0.000
Total		10385.547		-7856.240		213.728

Forces due to Horizontal Load

	load factor	Horizontal Force (kN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		252.610	1769.518	1836.474
due to Earth pressure	1.00	1563.661		4563.283
due to Live load surcharge	0.80	352.175		1541.646

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
252.61	1836.47	0.00	0.00
1563.66	4563.28		
352.17	1541.65		
2168.445	7941.402	0.000	0.000

Summary of Forces

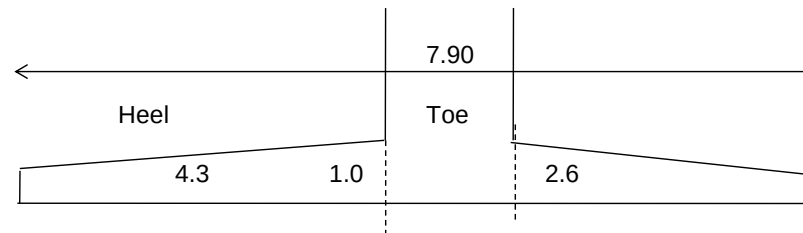
P	10385.547	kN
ML	85.162	kNm
MT	213.728	kNm

Centrifugal Force : Normal Case

$$\begin{aligned}
 \text{Centrifugal Force (C.F.)} &= 1.00 \times 0.00 = 0.000 \text{ KN} \\
 \text{Transverse Moment due to C.F.} &= 0.000 \times (1772.203 - 1762.248) = 0.000 \text{ kNm}
 \end{aligned}$$

$$\begin{aligned}
 \text{Base pressure on corner A} &= \sigma_A = P/A - ML/ZL + MT/ZT \\
 \text{Base pressure on corner B} &= \sigma_B = P/A + ML/ZL + MT/ZT \\
 \text{Base pressure on corner C} &= \sigma_C = P/A - ML/ZL - MT/ZT \\
 \text{Base pressure on corner D} &= \sigma_D = P/A + ML/ZL - MT/ZT
 \end{aligned}$$

LOAD CASES	Design Base Pressure						
	P	ML	MT	σ_A	σ_B	σ_C	σ_D
Normal Dry Case	kN	kNm	kNm	kN/m ²	kN/m ²	kN/m ²	kN/m ²
Case 1 : DL+SIDL-Normal Dry Case	13702.109	-1784.818	6.272	224.305	183.931	224.173	183.799
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	14645.309	684.902	928.728	220.115	235.608	200.589	216.082
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	10049.195	-2164.131	6.272	174.196	125.242	174.064	125.110
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	10385.547	85.162	213.728	155.945	157.872	151.452	153.378

Pressure calculation along Face AB:

Case 1 :	224.305	202.33	197.219	183.931
Case 2:	220.115	228.55	230.509	235.608
Case 4:	174.196	147.55	141.353	125.242
Case 5:	155.945	156.99	157.238	157.872

For Rare Combination

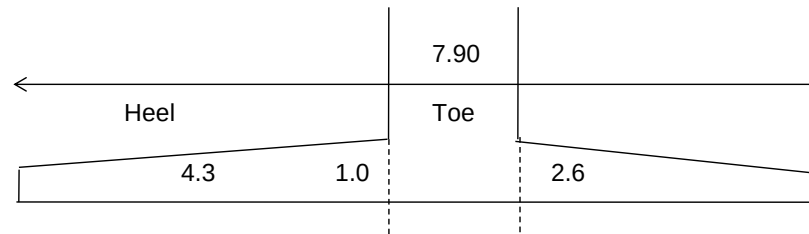
Average Base Pressure for Design of Heel Slab-along Face AB = 224.331 kN/m²

Average Base Pressure for Design of Toe Slab-along Face AB = 233.058 kN/m²

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 213.317 kN/m²

Average Base Pressure for Design of Toe Slab-along Face AB = 190.575 kN/m²

Pressure calculation along Face CD:

Case 1 :	224.173	202.20	197.087	183.799
Case 2:	200.589	209.02	210.983	216.082
Case 4:	174.064	147.42	141.221	125.110
Case 5:	151.452	152.50	152.744	153.378

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 213.185 kN/m²

Average Base Pressure for Design of Toe Slab-along Face CD = 213.533 kN/m²

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 213.185 kN/m²

Average Base Pressure for Design of Toe Slab-along Face CD = 190.443 kN/m²

Moment Calculation

	Rare Combination		Quasi-Permanent		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	224.33	233.06	213.32	190.58	kN/m ²
Upward moment due to Base pressure	2073.94	787.74	1972.12	644.14	kNm/m
Downward moment due to backfill	1490.06	0.00	1490.06	0.00	kNm/m
Downward moment due to self weight of slab	123.27	45.07	123.27	45.07	kNm/m
Net Moment	460.61	742.67	358.79	599.08	kNm/m
	Tension at Bottom of Heel Slab	Tension at Bottom of Toe Slab	Tension at Bottom of Toe Slab	Tension at Bottom of Toe Slab	

Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff	=	2	
Ecm	=	32308.25 N/mm ²	
Es	=	200000.00 N/mm ²	
Eceff	=	$\frac{E_{cm}}{(1 + \phi)}$	1.08E+04
Modular Ratio (m)	=	Es/ Eceff	18.57

		Rare Combination		Quasi Permanent Comb.		
		Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Working bending moment, M	=	460.61	742.67	358.79	599.08	kNm/m
Dx	=	1.00	1.00			m
Dy	=	1.00	1.00			m
Section Modulus (ZL) of uncracked sec	=	0.17	0.17			m ³
Bending Stress (M/ZL)	=	2.764	4.456			N/mm ²
Tensile stress of concrete , fctm	=	2.771	2.771			N/mm ²
Cracked or Uncracked Section	=	Uncracked	Cracked			
Section properties of Cracked section:						
Note: Stresses under Service load are usually within Linear Elastic Range hence such analysis involved use of Modulus ratio.						
Clear Cover, c	=	75.000	75.000			mm
Maximum dia used, ϕ	=	20.000	25.000			mm
Effective Depth deff (dy)	=	915.000	915.000			mm
Ast provided	=	2576.106	4908.739			mm ² /m
Percentage of steel , pt	=	0.0028	0.0054			
$k = \sqrt{2 \text{ pt} * m + (\text{pt} * m)^2} - \text{pt} * m$	=	0.275	0.358			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	251.889	327.330			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	663.111	587.670			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	578.111	500.170			mm
Cracked moment of Inertia (Icr)	=	$Dx * (k * dy)^3/3 + m \text{ Ast} * (dy - k * dy)^2$				
Icr	=	2.636E+10	4.317E+10			mm ⁴
Maximum compressive stress in concrete	=	4.401	5.631	3.428	4.542	< 16.8, SAFE

Maximum Tensile stress in steel	=	187.576	159.784	146.110	128.891	< 300, SAFE

Check For Crack Width in Quasi-Permanent Load Combination

Crack width , W_k = $S_r \max (\epsilon_{sm} - \epsilon_{cm})$

Above Formula For Calculation of $S_r \max$ is applicable if the spacing between the reinf. is less or equal to $5*(c+\phi/2)$

$5*(c+\phi/2)$	=	425.000	437.500	mm
Provided Spacing	=	100.000	100.000	mm
Check for Applicability of Formula	=	OK	OK	
Maximum crack spacing , $S_{r \max}$	=	$3.4 c +$	$0.425 k_1 k_2 \phi$	
		$\rho_{p \text{ eff}}$		
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , y_c	=	251.889	327.330	mm
$\rho_{p \text{ eff}} = A_s/A_{c \text{ eff}}$	=	, where $A_{c \text{ eff}}$ = effective area of concrete in tension surrounding the reinf.		
$h_{c \text{ eff}} = \text{Min of } 2.5 (D_y - d_y) , D_y - y_c/3 , D_y/2$	=	212.500	212.500	mm
$A_{c \text{ eff}} = D_x * h_{c \text{ eff}}$	=	212500.000	212500.000	mm
$\rho_{p \text{ eff}} = A_s/A_{c \text{ eff}}$	=	0.012	0.023	
Maximum crack spacing , $S_{r \max}$	=	535.462	438.983	mm
$(\epsilon_{sm} - \epsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct \text{ eff}} (1 + \alpha_e \rho_{p \text{ eff}})}{\rho_{p \text{ eff}}}$	/ E_s	
tensile stress in steel , σ_{sc}	=	146.110	128.891	N/mm ²
Kt	=	0.500	0.500	
Tensile strength of concrete = $f_{ct \text{ eff}} = f_{ctm}$	=	2.771	2.771	N/mm ²
$\alpha_e = E_s/E_{cm}$	=	6.190	6.190	
$(\epsilon_{sm} - \epsilon_{cm})$	=	0.00044	0.0004	
Crack width , $W_k = S_r \max (\epsilon_{sm} - \epsilon_{cm})$	=	0.235	0.170	mm
Check	=	SAFE	SAFE	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom lvl = 1763.248 m

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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	1.35			1989.225	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1.35			330.977	0.000	0.000	0.000	0.000
Wearing Course	1.75			360.216	0.000	0.000	0.000	0.000
Bearing Pedestal	1.35	25	0.902	30.456	0.000	0.000	-0.278	-8.467
				2710.873		0.000		-8.467
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	-0.800	-129.782	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	-0.800	-7.315	0.000	0.000
Bracket - Uniform portion	1.35	25	0.765	25.819	-1.100	-28.401	0.000	0.000
Bracket - Tapered portion	1.35	25	0.383	12.909	-1.050	-13.555	0.000	0.000
Cap - (uniform portion)	1.35	25	4.718	159.216	-0.025	-3.980	0.000	0.000
Cap - (corbel portion)	1.35	25	3.579	120.779	-0.025	-3.019	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.800	-30.240	0.000	0.000
Approach Slab	1.35	25	4.463	150.609	-1.100	-165.670	0.000	0.000
				678.504		-381.963		0.000
Substructure-Portion 2								
Abutment Shaft	1.35	25	43.499	1468.083	0.000	0.000	0.000	0.000
Total				4857.460		-381.963		-8.467

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		134.021	1769.518	840.311
due to Earth pressure	1.5	1840.285		5641.019

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
134.02	840.31	0.00	0.00
1840.28	5641.02	0.00	0.00
1974.306	6481.330	0.000	0.000

Summary of Forces

P	4857.460	kN
ML	6099.367	kNm
MT	8.467	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical Load**

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		4857.460		-381.963		-8.467
CWLL-Max. Reaction case	1.5	1214.286	0.000	0.000	1.155	1402.501
Total		6071.746		-381.963		1394.033

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		319.082	1769.518	2000.641
due to Earth pressure	1.5	1840.285		5641.019
due to Live load surcharge	1.2	428.791		1662.638

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
319.08	2000.64	0.00	0.00
1840.28	5641.02		
428.79	1662.64		
2588.158	9304.299	0.000	0.000

Summary of Forces

P	6071.746	KN
ML	8922.335	kNm
MT	1394.033	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				2710.873		0.000		-8.467
Substructure-Portion 1				678.504		-381.963		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	0.000	0.000	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	23.500	43.499	1379.998	0.000	0.000	0.000	0.000
Total				4769.375		-381.963		-8.467

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		134.021	1769.518	840.311
due to Earth pressure	1.5	1840.285		4757.136

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
134.02	840.31	0.00	0.00
1840.28	4757.14	0.00	0.00
1974.306	5597.447	0.000	0.000

Summary of Forces

P	4769.375	KN
ML	5215.484	kNm
MT	8.467	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical Load**

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		4769.375		-381.963		-8.467
CWLL-Max. Reaction case	1.5	285.714	0.000	0.000	1.155	329.999
Total		5055.089		-381.963		321.532

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		365.510	1769.518	2291.749
due to Earth pressure	1.5	1840.285		4757.136
due to Live load surcharge	1.2	467.924		1814.375

Forces along Long. Axis		Forces along Trans. Axis	
FL Cos θ	ML Cos θ	FL Sin θ	ML Sin θ
365.51	2291.75	0.00	0.00
1840.28	4757.14		
467.92	1814.38		
2673.719	8863.260	0.000	0.000

Summary of Forces

P	5055.089	KN
ML	8481.297	kNm
MT	321.532	kNm

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

Seismic Effect Factor = **1.50** $\alpha_h =$ 0.180 In Longitudinal direction
 $\alpha_h =$ 0.303 In Transverse direction
 $\alpha_v =$ 0.202 In Vertical direction

Weight of shaft below Ground level = 670.0 KN

Forces due to Vertical Load

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	1.35			1989.225		200.999	134.000	0.000	0.000	0.000	1770.465		0.000	0.000	1450.545
SIDL except Wearing Course	1.35			330.977		33.443	22.295	0.000	0.000	0.000	1771.452		0.000	0.000	274.366
Wearing Course	1.75			360.216		28.078	18.719	0.000	0.000	0.000	1771.003		0.000	0.000	217.746
Bearing Pedestal	1.35	25	0.9024	30.456		3.077	2.052	0.000	0.000	0.000			-0.278	-8.467	
				2710.873		265.598	177.065		0.000	0.000				-8.467	1942.657
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	4.807	162.228	32.446	16.392	10.928	-0.800	-129.782	-8.742	1770.061	221.035	0.000	0.000	111.672
Dirt Wall-Tapered portion	1.35	25	0.271	9.144	1.829	0.924	0.616	-0.800	-7.315	-0.493	1769.065	10.638	0.000	0.000	5.375
Bracket - Uniform portion	1.35	25	0.765	25.819				-1.100	-28.401						
Bracket - Tapered portion	1.35	25	0.383	12.909				-1.050	-13.555						
Cap - (uniform portion)	1.35	25	4.718	159.216	31.843	16.088	10.725	-0.025	-3.980	-0.268	1768.816	177.287	0.000	0.000	89.569
Cap - (corbel portion)	1.35	25	3.579	120.779	24.156	12.204	8.136	-0.025	-3.019	-0.203	1768.516	127.240	0.000	0.000	64.284
RCC Railing or Crash Barrier	1.35	25		37.800				-0.800	-30.240				0.000		
Approach Slab	1.35	25	4.463	150.609				-1.100	-165.670				0.000		
				678.504	90.273	45.608	30.405		-381.963	-9.707		536.200		0.000	270.899
Substructure-Portion 2															
Abutment Shaft	1.35	25	43.499	1468.083	159.614	80.640	53.760	0.000	0.000	0.000	1767.383	660.044	0.000	0.000	333.468
Total =				4857.460	249.887	391.846	261.231		-381.963	-9.707		1196.244		-8.467	2547.024
							-261.231			9.707					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1039.233	265.598	1769.518	6515.991	1942.657
due to Substructure		249.887	126.248		1196.244	604.367
due to Active Earth pressure	1.00	1226.857			3760.679	
due to dynamic increment of EP	1.50	1180.484			4577.327	

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
1039.23	6515.99	0.00	0.00	0.00	0.00	265.60	1942.66
249.89	1196.24	0.00	0.00	0.00	0.00	126.25	604.37
1226.86	3760.68						
1180.48	4577.33						
3696.46	16050.24	0.00	0.00	0.00	0.00	391.85	2547.02

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5118.691	4596.229	KN
ML	15658.572	15677.985	kNm
MT	2538.557	2538.557	kNm

Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case**Forces due to Vertical Load**

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				2710.873		265.598	177.065		0.000	0.000				-8.467	1942.657
Forces from Substructure				2146.586	249.887	126.248	84.165		-381.963	-9.707		1196.244		0.000	604.367
CWLL-Max. Reaction case	0.20			161.90		22.085	14.724	0.000	0.000	0.000	1772.203		1.155	187.000	197.774
Total =				5019.365	249.887	413.932	275.954		-381.963	-9.707		1196.244		178.533	2744.798
								-275.954					9.707		

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1084.773	287.684	1769.518	6801.527	2140.431
due to Substructure		249.887	126.248		1196.244	604.367
due to Active Earth pressure	1.00	1226.857			3760.679	
due to dynamic increment of EP	1.50	1180.484			4577.327	
due to Live load surcharge	0.20	71.465			277.106	
due to dynamic increment of Surcharge	1.50	365.334			1898.218	

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
1084.77	6801.53	0.00	0.00	0.00	0.00	287.68	2140.43
249.89	1196.24	0.00	0.00	0.00	0.00	126.25	604.37
1226.86	3760.68						
1180.48	4577.33						
71.47	277.11						
365.33	1898.22						
4178.80	18511.10	0.00	0.00	0.00	0.00	413.93	2744.80

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5295.319	4743.410	kN
ML	18119.432	18138.846	kNm
MT	2923.331	2923.331	kNm

Case 7 : DL+SIDL-Long. Seismic HFL Case**Forces due to Vertical Load**

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				2710.873		265.598	177.065		0.000	0.000				-8.467	1942.657
Substructure-Portion 1				678.504	90.273	45.608	30.405		-381.963	-9.707		536.200		0.000	270.899
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1768.383	0.000	0.000	0.000	0.000
Shaft below HFL	1.350	23.500	43.499	1379.998	141.997	71.740	47.827	0.000	0.000	0.000	1767.401	589.714	0.000	0.000	297.935
				1379.998	141.997	71.740	47.827		0.000	0.000		589.714		0.000	297.935
Total =				4769.375			255.297		-381.963	-9.707				-8.467	
								-255.297					9.707		

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1039.233	265.598	1769.518	6515.991	1942.657
due to Substructure		232.270	117.348		1125.914	568.835
due to Active Earth pressure	1.00	981.825			2811.261	
due to dynamic increment of EP	1.50	1175.567			4562.113	

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
1039.23	6515.99	0.00	0.00	0.00	0.00	265.60	1942.66
232.27	1125.91	0.00	0.00	0.00	0.00	117.35	568.83
981.83	2811.26						
1175.57	4562.11						
3428.89	15015.28	0.00	0.00	0.00	0.00	382.95	2511.49

Summary of Forces

	Downward	Upward	
P	5024.672	4514.078	KN
ML	14623.609	14643.022	kNm
MT	2519.959	2519.959	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case**Forces due to Vertical Load**

Lloads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 = 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				2710.873		265.598	177.065		0.000	0.000				-8.467	1942.657
Forces from Substructure				2058.502	232.270	117.348	78.232		-381.963	-9.707		1125.914		0.000	568.835
CWLL-Min. Reaction case	0.20			38.10		5.197	3.464	0.000	0.000	0.000	1772.203		1.155	44.000	46.535
Total =				4807.470	232.270	388.142	258.762		-381.963	-9.707		1125.914		35.533	2558.027

-258.762

9.707

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1084.773	270.795	1769.518	6801.527	1989.192
due to Substructure		232.270	117.348		1125.914	568.835
due to Active Earth pressure	1.00	981.825			2811.261	
due to dynamic increment of EP	1.50	1175.567			4562.113	
due to Live load surcharge	0.20	77.987			302.396	
due to dynamic increment of Surcharge	1.50	605.347			2944.071	

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
1084.77	6801.53	0.00	0.00	0.00	0.00	270.79	1989.19
232.27	1125.91	0.00	0.00	0.00	0.00	117.35	568.83
981.83	2811.26						
1175.57	4562.11						
77.99	302.40						
605.35	2944.07						
4157.77	18547.28	0.00	0.00	0.00	0.00	388.14	2558.03

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5066.232	4548.709	KN
ML	18155.611	18175.025	kNm
MT	2593.559	2593.559	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				4857.460	261.231 -261.231	-381.963	-9.707 9.707	-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		311.770	885.327231	1769.518	1954.797	6475.52384
due to Substructure		74.966	420.827068		358.873	2014.55621
due to Active Earth pressure	1.00	1226.857			3760.679	
due to dynamic increment of EP	1.50	354.145			1373.198	

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
311.77	1954.80	0.00	0.00	0.00	0.00	885.33	6475.52
74.97	358.87	0.00	0.00	0.00	0.00	420.83	2014.56
1226.86	3760.68						
354.15	1373.20						
1967.74	7447.55	0.00	0.00	0.00	0.00	1306.15	8490.08

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5118.691	4596.229	kN
ML	7055.878	7075.292	kNm
MT	8498.547	8498.547	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				5019.365	275.954 -275.954	-381.963	-9.707 9.707	178.533

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		357.310	958.945074	1769.518	2240.333	7134.77162
due to Substructure		74.966	420.827068		358.873	2014.556
due to Earth pressure	1.00	1226.857			3760.679	
due to dynamic increment of EP	1.50	354.145			1373.198	
due to Live load surcharge	0.20	71.465			277.106	
Surcharge	1.50	109.600			569.466	

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
357.31	2240.33	0.00	0.00	0.00	0.00	958.95	7134.77
74.97	358.87	0.00	0.00	0.00	0.00	420.83	2014.56
1226.86	3760.68						
354.15	1373.20						
71.47	277.11						
109.60	569.47						
2194.34	8579.66	0.00	0.00	0.00	0.00	1379.77	9149.33

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5295.319	4743.410	kN
ML	8187.986	8207.399	kNm
MT	9327.861	9327.861	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				4769.375	255.297 -255.297	-381.963	-9.707 9.707	-8.467

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		311.770	885.327231	1769.518	1954.797	6475.524
due to Substructure		69.681	391.158861		337.774	1896.115
due to Active Earth pressure	1.00	981.825			2811.261	
due to dynamic increment of EP	1.50	352.670			1368.634	

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
311.77	1954.80	0.00	0.00	0.00	0.00	885.33	6475.52
69.68	337.77	0.00	0.00	0.00	0.00	391.16	1896.12
981.83	2811.26						
352.67	1368.63						
1715.95	6472.47	0.00	0.00	0.00	0.00	1276.49	8371.64

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5024.672	4514.078	kN
ML	6080.796	6100.210	kNm
MT	8380.106	8380.106	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case**Forces due to Vertical Load**

Loads	Load Factor	Unit Weights (kN/m ³)	Volume (m ³)	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 =				4807.470	258.762 -258.762	-381.963	-9.707 9.707	35.533

Forces due to Horizontal Load

	load factor	FL (kN)	FT (kN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		357.310	902.649031	1769.518	2240.333	6630.64056
due to Substructure		69.681	391.158861		337.774	1896.115
due to Earth pressure	1.00	981.825			2811.261	
due to dynamic increment of EP	1.50	352.670			1368.634	
due to Live load surcharge	0.20	77.987			302.396	
Surcharge	1.50	181.604			883.221	

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
357.31	2240.33	0.00	0.00	0.00	0.00	902.65	6630.64
69.68	337.77	0.00	0.00	0.00	0.00	391.16	1896.12
981.83	2811.26						
352.67	1368.63						
77.99	302.40						
181.60	883.22						
2021.08	7943.62	0.00	0.00	0.00	0.00	1293.81	8526.76

Summary of Forces

	Seismic Downward	Seismic Upward	
P	5066.232	4548.709	kN
ML	7551.949	7571.363	kNm
MT	8562.289	8562.289	kNm

Centrifugal Force : Normal Case

$$\begin{aligned} \text{Centrifugal Force (C.F.)} &= 1.50 \times 0.00 \\ \text{Transverse Moment due to C.F.} &= 0.000 \times (1772.203 - 1763.248) = 0.000 \text{ KN} \\ &= 0.000 \text{ kNm} \end{aligned}$$

Centrifugal Force : Seismic Case

$$\begin{aligned} \text{Centrifugal Force (C.F.)} &= 0.20 \times 0.00 \\ \text{Transverse Moment due to C.F.} &= 0.000 \times (1772.203 - 1763.248) = 0.000 \text{ KN} \\ &= 0.000 \text{ kNm} \end{aligned}$$

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

Seismic	0.00	0.00	0.00	0.00

Summary of ULS Forces for Design of Abutment Shaft

		Total forces at bottom of abutment shaft		
LOAD CASES		P	ML	MT
Normal Dry Case		kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case		4857.460	6099.367	8.467
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		6071.746	8922.335	1394.033
Normal HFL Case				
Case 3 : DL+SIDL-Normal HFL Case		4769.375	5215.484	8.467
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		5055.089	8481.297	321.532
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	5118.691	15658.572	2538.557
	UP	4596.229	15677.985	2538.557
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	5295.319	18119.432	2923.331
	UP	4743.410	18138.846	2923.331
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	5024.672	14623.609	2519.959
	UP	4514.078	14643.022	2519.959
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	5066.232	18155.611	2593.559
	UP	4548.709	18175.025	2593.559

Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	5118.691	7055.878	8498.547
	UP	4596.229	7075.292	8498.547
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	5295.319	8187.986	9327.861
	UP	4743.410	8207.399	9327.861
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	5024.672	6080.796	8380.106
	UP	4514.078	6100.210	8380.106
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	5066.232	7551.949	8562.289
	UP	4548.709	7571.363	8562.289
MAX =		6071.75	18175.02	9327.86

Design of Wall:**Material Property:**

Grade of Concrete	=	M 35
fck	=	35 N/mm ²
fcd	=	15.633 N/mm ²
Grade of steel	=	Fe 500
fy	=	500 N/mm ²
fyd	=	434.783 N/mm ²
Es	=	200000.00 N/mm ²

Cross section of Wall:

Thickness of Wall (B)	=	1.000 m
Depth of Wall (D)	=	8.500 m
Area of Concrete (Ac)	=	8.500 m ²
Clear Cover to earth faces	=	75 mm
Clear Cover to non earth faces	=	50 mm
Maximum Dia of Vertical Reinf.	=	32 mm
Dia of Horizontal Reinf.	=	16 mm

As per Clause 7.6.4.1 of IRC:112-2011

Ultimate axial force (Pu) = 6071.75 kN

$$0.1 f_{cd} A_c = 0.1 \times 15.63 \times 8500000 = 13288333.3 \text{ N} = 13288.33 \text{ kN}$$

Since Axial Force is less than axial capacity of section , Section will design as bending element . Neglecting axial force

PART 1: LONGITUDINAL MOMENT : VERTICAL REINFORCEMENT ON EARTH FACE

Ultimate Design bending moment (ML)	=	18175.02 kNm	=	2138.238 kNm/m

Check For Depth of Wall :

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 2138.24 \text{ kNm/m}$$

$$b = 1000.00 \text{ mm}$$

$$\text{Effective Depth Required (dreq)} = \text{SQRT} \left(\frac{2138.24 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$(dreq) = 608.488 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 715.49 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 1000.00 \text{ mm}$$

$$\text{Effective depth provided (deff)} = 893 \text{ mm}$$

OK

$$R = \frac{M_u}{(b \times d^2)} = 2.68$$

Minimum Longitudinal Reinforcement in wall on each face

$$A_{st \text{ min}} = 0.0012 \times b \times D = 1200.00 \text{ mm}^2/\text{m}$$

Area of Steel Required:

$$\frac{p_t}{100} = \frac{A_{st \text{ req}}}{b \times D} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$$

$$A_{st \text{ req}} = 6831.033 \text{ mm}^2/\text{m}$$

$$A_{st \text{ required}} = \max(A_{st \text{ min}}, A_{st \text{ req}}) = 6831.03 \text{ mm}^2/\text{m}$$

$$\text{Total area of steel required in full length} = 58063.78 \text{ mm}^2$$

Provide	32 mm dia	@	200.00 mm c/c	=	8042.48	mm ² /m	OK
Provide	32 mm dia	@	200.00 mm c/c	=			

Effective length of shaft = 8254 mm

Calculation of reinforcement in numbers

Provide	32	mm dia	-	42.00	nos	=	68361.06	mm ²	OK
Provide	32	mm dia	-	43.00	nos	=			

Percentage of steel = 0.804 %

Check for Moment of Resistance of Section due to Steel

$$\begin{aligned} \text{Limiting Depth of Neutral Axis, } X_m &= \frac{0.0035 \cdot d}{(0.0035 + f_{yd}/E_s)} \\ &= \frac{0.0035}{0.0035 + \frac{893.00}{200000}} \times 893.00 \\ &= 550.85 \text{ mm} \end{aligned}$$

$$\begin{aligned} \text{Depth of Neutral Axis, } X &= \frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b} \\ &= \frac{434.78}{0.36} \times \frac{8042.48}{35.00 \times 1000.00} \\ &= 277.52 \text{ mm} \quad \text{OK} \end{aligned}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned} z &= d - 0.416 \cdot X \\ &= 893.00 - 115.45 \\ &= 777.55 \text{ mm} \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned} MR &= f_{yd} \cdot A_{st} \cdot z \\ &= 434.78 \times 8042.48 \times 777.55 \\ &= 2.72E+09 \text{ Nmm/m} \\ &= 2718.89 \text{ kNm/m} > 2138.24 \text{ kNm/m} \end{aligned}$$

Moment of Resistance of Wall is More than Design Bending Moment, HENCE Wall IS SAFE IN BENDING

LONGITUDINAL REINFORCEMENT ON NON EARTH FACE

Minimum Longitudinal Reinforcement in wall on each face

$$\begin{aligned} A_{st \text{ min}} &= 0.0012 \times b \times D \\ &= 1200.00 \text{ mm}^2/\text{m} \\ &= 10200.00 \text{ mm}^2 \end{aligned}$$

Provide	20	mm dia	@	200.00	mm c/c	=	1570.80 mm ² /m	OK

Calculation of reinforcement in numbers

Provide	20	mm dia	@	42.00	nos	=	13194.69 mm ²	OK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

$$\begin{aligned} \text{maximum of following} &= 0.2500 \times 9613.27 = 2403.318 \text{ As per IRC: 112-2011, Clause 16.3.2} \\ &= 0.001 \times 1.00E+06 = 1000.000 \\ \text{Maximum Horizontal Reinf.} &= 2403 \text{ mm}^2 \text{ per meter} \end{aligned}$$

$$\begin{aligned} \text{Min dia of bar} &= 0.250 \times 32 = 8 \text{ mm} \\ \text{or} &= 8 \text{ mm} \end{aligned}$$

$$\text{Maximum Spacing between} \leq 300 \text{ mm c/c}$$

2 Legged	16	dia	@	150	c/c	=	2680.826 mm ²	OK

Confinement Reinforcement

As per Clause 17.2.1.3 (Note 2) of IRC:112-2011

$$\text{Distance between links or ties (ST)} = \frac{1}{3} \times 843 = 281$$

$$\text{or } 200.00 \text{ mm}$$

$$\text{Governing Spacing} = 200.00 \text{ mm}$$

As per Clause 17.2.1.3 (Note 1) of IRC:112-2011

The Spacing of hoops and ties in the longitudinal direction (SL)

$$\text{SL} = 5 \times 32 = 160 \text{ mm}$$

$$\text{or } \frac{1}{5} \times 843 = 168.6 \text{ mm}$$

$$\text{Min} = 100 \text{ mm}$$

2 Legged	16 dia	@	100 c/c	=	4021.239 mm ²	OK
50 Legged	8 dia	@	100 c/c	=	25132.741 mm ²	
75 links	8 dia	@	100 c/c	=	37699.112 mm ²	
66853.092 mm ²						

Minimum Confinement Reinforcement:

$$n_k = \frac{\text{NED}}{A_k f_{ck}} = \frac{6071746.3}{297500000} = 0.0204$$

$$A_C = 8.500 \text{ mm}^2$$

$$A_{CC} = 0.875 \times 8.400 = 7.350 \text{ mm}^2$$

$$\rho_L = 0.00806 \text{ per meter}$$

$$\rho_L = 0.06849$$

$$f_{yd} = 434.783$$

$$f_{cd} = 15.633$$

$$\omega_{wd,req} = 0.37 \frac{A_C}{A_{CC}} n_k + 0.13 \frac{f_{yd}}{f_{cd}} (\rho_L - 0.01)$$

$$\omega_{wd,req} = 0.2202$$

$$\omega_{wd} = \max(\omega_{wd,req}, 0.12) = 0.2202$$

As per Clause 17.2.1.1 (4) of IRC:112-2011

$$\text{Confined Reinforcement} = \omega_{wd} = \rho_w f_{yd} / f_{cd} \quad \text{where, } \rho_w = \frac{A_{sw}}{S_L \cdot b}$$

Volumetric ratio,

$$A_{sw} = 66853.092 \text{ mm}^2$$

$$S_L = 100.000 \text{ mm}$$

$$b = 843.000 \text{ mm}$$

$$\rho_w = 0.793$$

$$\omega_{wd,c} = 22.055$$

$$\omega_{wd,c} \geq \omega_{wd} \quad \text{as per equation 17.7 of IRC:112-2011}$$

$$\omega_{wd,c} = 22.06 \text{ OK}$$

Length of Potential Plastic Hinges

Refer clause 17.2.1.4 of IRC:112-2011

$$n_k = \frac{\text{NED}}{A_k f_{ck}} = 0.0204 < 0.30$$

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1763.248 m

Case 1 : DL+SIDL-Normal Dry Case**Forces due to Vertical Load**

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			1473.500	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1			245.168	0.000	0.000	0.000	0.000
Wearing Course	1			205.838	0.000	0.000	0.000	0.000
Bearing Pedestal	1	25	0.902	22.560	0.000	0.000	-0.278	-6.272
				1947.066		0.000		-6.272
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	4.807	120.169	-0.800	-96.135	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.271	6.773	-0.800	-5.419	0.000	0.000
Bracket - Uniform portion	1	25	0.765	19.125	-1.100	-21.038	0.000	0.000
Bracket - Tapered portion	1	25	0.383	9.563	-1.050	-10.041	0.000	0.000
Cap - (uniform portion)	1	25	4.718	117.938	-0.025	-2.948	0.000	0.000
Cap - (corbel portion)	1	25	3.579	89.466	-0.025	-2.237	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.800	-22.400	0.000	0.000
Approach Slab	1	25	4.463	111.563	-1.100	-122.719	0.000	0.000
				502.595		-282.936		0.000
Substructure-Portion 2								
Abutment Shaft	1	25	43.499	1087.469	0.000	0.000	0.000	0.000
Total				3537.130		-282.936		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		98.284	1769.518	616.238
due to Earth pressure	1	1226.857		3760.679
				4376.918

Summary of Forces at Bottom of abutment shaft

P	3537.130	KN
ML	4093.982	kNm
MT	-6.272	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case**Forces due to Vertical Load**

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		3537.130		-282.936		-6.272
CWLL-Max. Reaction case	1	809.524	0.000	0.000	1.155	935.000
Total		4346.654		-282.936		928.728

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		221.657	1769.518	1389.792
due to Earth pressure	1	1226.857		3760.679
due to Live load surcharge	0.8	285.861		1108.425
				6258.897

Summary of Forces at Bottom of abutment shaft

P	4346.654	KN
ML	5975.961	kNm
MT	928.728	kNm

Case 3 : DL+SIDL-Normal HFL Case**Forces due to Vertical Load**

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				1947.066		0.000		-6.272
Substructure-Portion 1				502.595		-282.936		0.000

Substructure-Portion 2								
Shaft above HFL	1.000	25.000	0.000	0.000	0.00	0.00	0.00	0.00
Shaft below HFL	1.000	23.500	43.499	1022.221	0.00	0.00	0.00	0.00
				3471.881		-282.936		-6.272

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		98.284	1769.518	616.238
due to Earth pressure	1	1226.857		3171.424
				3787.663

Summary of Forces at Bottom of abutment shaft

P	3471.881	KN
ML	3504.727	kNm
MT	-6.272	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case**Forces due to Vertical Load**

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		3471.881		-282.936		-6.272
CWLL-Max. Reaction case	1	190.476	0.000	0.000	1.155	220.000
Total		3662.357		-282.936		213.728

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		252.610	1769.518	1583.864
due to Earth pressure	1	1226.857		3171.424
due to Live load surcharge	0.8	311.949		1209.584
				5964.872

Summary of Forces at Bottom of abutment shaft

P	3662.357	KN
ML	5681.936	kNm
MT	213.728	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.00	x	0.00	=	0.000 KN
Transverse Moment due to C.F.	=	0.000	x (	1772.203 - 1763.248)	=	0.000 kNm

Summary of SLS Forces for Design of Abutment Shaft

LOAD CASES	Total forces at bottom of abutment shaft		
Normal Dry Case	P	ML	MT
	kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case	3537.130	4093.982	-6.272
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	4346.654	5975.961	928.728
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	3471.881	3504.727	-6.272
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	3662.357	5681.936	213.728

IN RARE COMBINATION

Max SLS Moment	=	5975.961	kNm
Max Moment per meter	=	703.054	kNm/m

IN QUASI-PERMANENT

Max SLS Moment	=	4093.982	kNm
Max Moment per meter	=	481.645	kNm/m

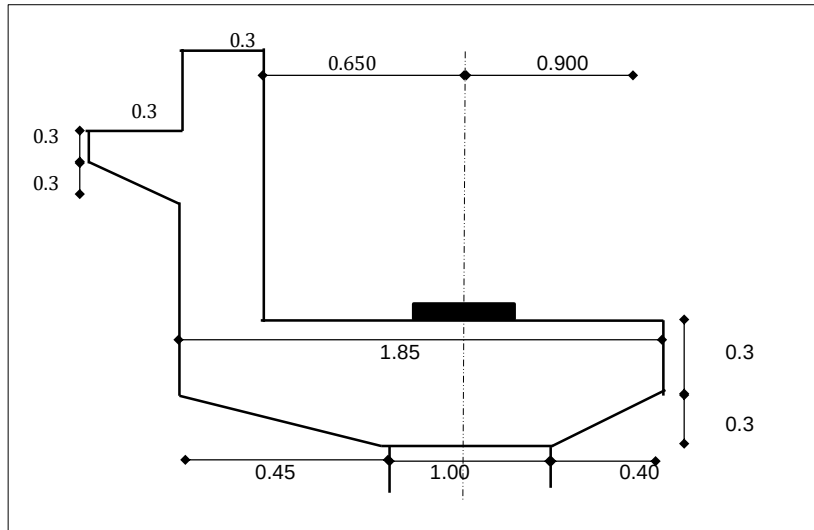
Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff = 2

		Rare Combination		Quasi permanent	
		Short term	Long Term		
Working bending moment, M	=	703.05	703.05	481.64	kNm/m
Dx (unit width of shaft)	=	1.00	1.00	1.00	m
Dy (Thickness of shaft)	=	1.00	1.00	1.00	m
Section Modulus (ZL) of uncracked	=	0.17	0.17	0.17	m ³
Bending Stress (M/ZL)	=	4.218	4.218	2.890	N/mm ²
Tensile stress of concrete , fctm	=	2.771	2.771	2.771	N/mm ²
Cracked or Uncracked Section	=	Cracked	Cracked	Cracked	
Section properties of Cracked section:					
Note: Stresses under Service load are usually within Linear Elastic Range hence such analysis involved use of Modulus ratio.					
Es	=	200000.00	200000.00	200000.00	N/mm ²
Ecm	=	32308.25	32308.25	32308.25	N/mm ²
Eceff	=	32308.25	10769.42	10769.42	N/mm ²
Modular Ratio (m)	=	6.19	18.57	18.57	
Clear Cover, c	=	75.000	75.000	75.00	mm
Maximum dia used, ϕ	=	32.000	32.000	32.00	mm
Effective Depth deff (dy)	=	893.000	893.000	893.00	mm
Ast provided	=	8042.477	8042.477	8042.48	mm ² /m
Percentage of steel , pt	=	0.0080	0.0080	0.0080	
$k = \sqrt{2 p_t * m + (p_t * m)^2} - p_t * m$	=	0.270	0.417	0.417	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	240.813	372.588	372.588	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	652.187	520.412	520.412	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	561.187	429.412	429.412	mm
Cracked moment of Inertia (Icr)	=	$Dx * (k * dy)^3 / 3 + m A_{st} * (dy - k * dy)^2$			
Icr	=	2.583E+10	5.769E+10	5.769E+10	mm ⁴
Maximum compressive stress in concrete	=	6.554	4.541	3.111	< 16.8, SAFE
Maximum tensile stress in concrete	=	17.751	6.342	4.345	
Maximum Tensile stress in steel	=	94.551	97.183	66.578	< 300, SAFE

Check For Crack Width in Quasi-Permanent Case

Crack width , Wk	=	Sr max (ε _{sm} - ε _{cm})	
Above Formula For Calculation of Sr max is applicable if the spacing between the reinf. is less or equal to 5*(c+φ/2)			
5*(c+φ/2)	=	455.000	mm
Provided Spacing	=	200.000	mm
Check for Applicability of Formula	=	OK	
Maximum crack spacing , S _{r,max}	=	3.4 c + $\frac{0.425 k_1 k_2 \phi}{\rho_{p,eff}}$	
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	372.588	mm
$\rho_{p,eff} = A_s / A_{c,eff}$	=	, where A _{c,eff} = effective area of concrete in tension surrounding the reinf.	
hc eff = Min of 2.5 (Dy - dy) , Dy - yc/3 , Dy/2	=	267.500	mm
A _{c, eff} = Dx * hc,eff	=	267500.000	mm
$\rho_{p,eff} = A_s / A_{c,eff}$	=	0.030	
Maximum crack spacing , S _{r,max}	=	435.939	mm
(ε _{sm} - ε _{cm})	=	$\frac{\sigma_{sc} - k_t f_{ct,eff} (1 + \alpha_e \rho_{p,eff})}{E_s}$	
tensile stress in steel , σ _{sc}	=	66.578	N/mm ²
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm ²
α _e = Es/Ecm	=	18.571	
(ε _{sm} - ε _{cm})	=	0.00020	
Crack width , Wk=Sr max (ε _{sm} - ε _{cm})	=	0.087	mm
Check	=	< 0.3 ,SAFE	

DESIGN OF ABUTMENT CAP

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl. 710.8.7 of IRC : 78-2014 is 225 mm.

$$\begin{aligned} \text{Assuming a cap thickness of} &= 300 \text{ mm} \\ \text{Volume of abutment cap} &= 300 \times 1850 \times 8500 \\ &= 4.72\text{E}+09 \text{ mm}^3 \end{aligned}$$

as per cl. 710.8.7 of IRC : 78- 2014

$$\begin{aligned} \text{Quantity of steel} &= 1 \% \text{ of volume} \\ &= \frac{1}{100} \times 4.72\text{E}+09 = 4.72\text{E}+07 \text{ mm}^3 \end{aligned}$$

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction

$$= 2.36\text{E}+07 \text{ mm}^3$$

Clear cover

$$= 50 \text{ mm}$$

Length of bar

$$= 8500 - 100 = 8400 \text{ mm}$$

Area of steel required in longitudinal direc

$$= \frac{2.36\text{E}+07}{8400} = 2808.036 \text{ mm}^2 \quad (\text{top +Bottom})$$

Provide	10	Nos. of	16	mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

$$\text{Provided steel} = 2011 \text{ mm}^2$$

(b) Transverse steel

$$\text{Volume of steel to be provided in transverse direction} = 2.36\text{E}+07 \text{ mm}^3$$

$$\text{Volume of steel required per meter} = \frac{2.36\text{E}+07}{8.50} = 2.78\text{E}+06 \text{ mm}^3/\text{m}$$

Provide	2 L	16 mm dia bar @	200 mm c/c stirrups

$$\text{Length of each stirrups} = 1850 - 100 - 16 = 1734 \text{ mm}$$

$$\text{Volume of steel provided per meter} = 3.49\text{E}+06 \text{ mm}^3/\text{m} \quad \text{OK}$$

$$\text{Thickness of Abutment Cap (uniform) } s = 0.3 \text{ m}$$

$$\text{Thickness of Abutment Cap (tapered)} = 0.3 \text{ m}$$

$$\text{C.G. of dirt wall from face of abutment shaft (a)} = 0.300 \text{ m}$$

$$\text{Overall depth of Abutment cap at face of abutment shaft} = 0.600 \text{ m}$$

$$\text{Clear cover} = 50 \text{ mm}$$

$$\text{Diameter of the main bar} = 20 \text{ mm}$$

$$\text{Effective cover } d' = 60 \text{ mm}$$

$$\text{Effective depth of cap } d = 0.540 \text{ m}$$

For the section to be designed as corbel " a / d " shall be less than 1.

$$\begin{aligned} \text{Hence } a/d &= 0.3 / 0.54 \\ &= 0.556 < 1.0 \end{aligned}$$

For the section to be designed as corbel "s / d" shall be greater than 0.5.

$$\begin{aligned} \text{Hence } s/d &= 0.3 / 0.54 \\ &= 0.56 > 0.5 \quad \text{Proceed with the design} \end{aligned}$$

Note: THE ABUTMENT CAP HAS BEEN DESIGNED AS CORBEL FOR DIRT WALL AND LIVE LOAD ON DIRT WALL

1. Dead Load

Self Weight of Dirt Wall	=	14.9344 kN		
Self Weight of Bracket	=	0.135 m ³ /m	x	25
	=	3.375 kN		
Total Dead Load	=	18.3094 kN		
Load Factor	=	1.35		
Ultimate Dead Load	=	24.7177 kN		

2. Live Load

Assuming Class 70R Boggie load, One Axle is Directly over Dirt Wall

Vertical Load on Dirt Wall	=	200 kN
Load Factor	=	1.5
Factored Live load	=	300 kN

Actual horizontal force in normal case	=	40.00 kN
Effective width for this load is considered as (b)	=	1000 mm

Vertical Load (V_u)

Total maximum Vertical Load " V _u "	324.72 kN

Horizontal Load (H_u)

$$\begin{aligned} H_u &= 1.7 \times \text{actual horizontal force in working load condition} \\ &\quad \text{but not less than} \\ &= 0.2 \times V_u \end{aligned}$$

Total maximum Horizontal Load " H _u "	68.00 kN

Now the design for the corbel is carried out as per the following steps.

STEP I

Check for Nominal Shear Strength

Ensure $V_u / b d \leq 0.15 f_c'$

$$\begin{aligned} \text{where; } V_u &= 324.72 \text{ kN} \\ &= 324717.66 \text{ N} \\ bd &= 1000 \times 540 \text{ sqmm} \\ &= 540000.00 \text{ mm}^2 \\ V_u / b d &= 324717.66 / 540000 \\ V_u / b d &= 0.60 \text{ N / mm}^2 \\ f_c' &= 28 - \text{day standard cylinder strength of concrete used.} \\ &= 0.80 \text{ times the standard cube strength} \\ \text{Grade of Concrete} &= \text{M 35} \\ 0.15 f_c' &= 0.15 \times 0.8 \times 35 \\ &= 4.20 \text{ N / mm}^2 \quad \text{Ensured} \end{aligned}$$

Hence $V_u / b d \leq 0.15 f_c'$ is Ensured

STEP II

Calculation of Shear Friction Reinforcement " A_{vf} "

$$A_{vf} = \frac{V_u}{0.87 f_{sy} m}$$

$$\begin{aligned} \text{where; } f_{sy} &= \text{yield stress value of the reinforcement used.} \\ &= 500.00 \text{ N / mm}^2 \end{aligned}$$

Type of Surface		m
1	Concrete placed monolithically across interface.	1.40
2	Concrete placed against hardened concrete but with roughened surface	1.00
3	Concrete anchored to structural steel	0.70
4	Concrete placed against hardened concrete but surface not roughened	0.60

$$\text{Type of Surface (1 / 2 / 3 / 4) ?} = \mathbf{1.00}$$

$$m = 1.40$$

(Note: Only monolithic construction is recommended)

$$A_{vf} = \frac{324.72 \times 1000}{0.87 \times 500 \times 1.4} \text{ mm}^2$$

$$A_{vf} = \mathbf{545.74} \text{ mm}^2$$

STEP III

Calculation for Direct Tension Reinforcement " A_t ".

$$A_t = \frac{H_u}{0.87 f_{sy}}$$

$$H_u = 68.00 \text{ kn}$$

$$A_t = \frac{68 \times 1000}{0.87 \times 500}$$

$$= \mathbf{156.32} \text{ mm}^2$$

STEP IV

Calculation for Flexural Tension Reinforcement " A_f "

$$A_f = \frac{[V_u a + H_u (h - d')]}{0.87 f_{sy} d}$$

$$= \frac{324.72 \times 1000 \times 300 + 68 \times 1000 (600 - 60)}{0.87 \times 500 \times 540}$$

$$= \mathbf{571.03} \text{ mm}^2$$

STEP V

Total Primary Tensile Reinforcement " A_s "

$$A_s \geq (A_f + A_t)$$

$$\geq (2 / 3 A_{vf} + A_t)$$

$$\geq (0.04 f'_c / f_{sy}) b d$$

Provide the largest of these three magnitudes as A_s .

$$(A_f + A_t) = 571.04 + 156.33 \text{ mm}^2$$

$$= 727.35 \text{ mm}^2$$

$$(2 / 3 A_{vf} + A_t) = 2 / 3 \times 545.75 + 156.32 \text{ mm}^2$$

$$= 520.15 \text{ mm}^2$$

$$(0.04 f'_c / f_{sy}) b d = 0.04 \times 0.8 \times 35 / 500 \times 1000 \times 540$$

$$= 1209.60 \text{ mm}^2$$

$$\text{Hence } A_s = \mathbf{1209.60} \text{ mm}^2$$

STEP VI

The total sectional area of the stirrups is " A_h " (closed ties) to be provided horizontally, one below the other, and next to " A_s "

$$A_h \geq 0.25 * A_s$$

$$\geq 0.333 * A_{vf}$$

Provide the largest of these two magnitudes as A_h .

$$0.25 * A_s = 0.25 \times 1209.6$$

$$= 285.52 \text{ mm}^2$$

$$0.333 * A_{vf} = 0.333 \times 545.75$$

$$= 181.73 \text{ mm}^2$$

$$\text{Hence } A_h = 285.52 \text{ mm}^2$$

STEP VII

The total steel in vertical stirrups is " A_v "

$$\begin{aligned} V_c &= 10 \text{ bd in kgs} \quad \text{where } b \text{ \& } d \text{ are in cms} \\ &= 10 \times 100 \times 54 \\ &= 54000.00 \text{ kg} \\ &= 540.00 \text{ kN} \\ \text{Pitch} &= 400 \text{ mm} \\ A_v &= \frac{0.50 * (V_u - V_c) * p}{f_{sy} d} \\ &= \frac{0.5 \times (324.72 - 540) \times 400 \times 1000}{500 \times 540} \\ A_v &= 0.00 \text{ mm}^2 \end{aligned}$$

Reinforcement Details**Total Primary Tensile Reinforcement " A_s "**

$$\begin{aligned} A_s &= 1209.60 \text{ mm}^2 \\ \text{Diameter of Primary Steel} &= 20 \text{ mm} \\ \text{Area of one bar} &= 314.16 \text{ mm}^2 \\ \text{Spacing of Bar} &= 200 \text{ mm c/c} \\ A_{s \text{ provided}} &= 1570.796 \text{ mm}^2 > A_s \quad \text{R/F is adequate} \end{aligned}$$

provide	200 mm c/c 20 mm diameter bars as main reinforcement

Horizontal Steel (Closed Stirrups) " A_h "

$$\begin{aligned} A_h &= 285.52 \text{ mm}^2 \\ A_h / m &= 285.52 \text{ mm}^2 / m \end{aligned}$$

The stirrups shall be provided below A_s and within a depth of " $2 / 3 d$ " below A_s .

$$\begin{aligned} 2 / 3 d &= (2 / 3) \times 540 \\ &= 360.00 \text{ mm} \\ \text{Diameter of Stirrup Bar} &= 12.00 \text{ mm} \\ \text{Area of one bar} &= 113.10 \text{ mm}^2 \\ \text{No. of layers of Stirrups} &= 2.00 \text{ nos.} \\ \text{Spacing of Stirrups} &= 180.000 \text{ mm} \\ \text{No. of legs of stirrups} &= 2.00 \\ A_{h \text{ provided}} &= 113.1 \times 2 \times 2 \\ &= 452.389 \text{ mm}^2 > A_h \quad \text{R/F is adequate} \end{aligned}$$

provide	2 legged. 12 mm diameter bars as horizontal stirrups in 2 layers per meter width

Vertical Steel (Closed Stirrups) " A_v "

A _v	=	0.00	mm ²	
A _v	=	0.25 x A _s	=	302.4 mm ²
A _v max	=	302.40		
A _v /m	=	302.40	mm ² /m	
Diameter of Stirrup Bar	=	10.00	mm	
Area of one bar	=	78.54	mm ²	
Pitch	=	400	mm	
No. of legs of stirrups	=	2.00		
A _h provided	=			
	=	393	mm ²	> A _v R/F is adequate

provide 2 legged. 10 mm diameter bars as vertical stirrups at 400 mm spacing per meter width	

DESIGN OF DIRT WALL

Dirt wall will be designed as a vertical cantilever.

1.) NORMAL CASE

1a. Dead Load

$$\text{Self Weight of Dirt Wall} = 5.078 \text{ m}^3 \times 25.00 = 126.942 \text{ kN}$$

$$\text{Self Weight of Dirt Wall/ m} = 126.942 / 8.50 = 14.934 \text{ kN}$$

1b. Live Load

Assuming Class 70R Boggie load, One Axle is Directly over Dirt Wall

$$\text{Vertical Load on Dirt Wall} = 200 \text{ kN}$$

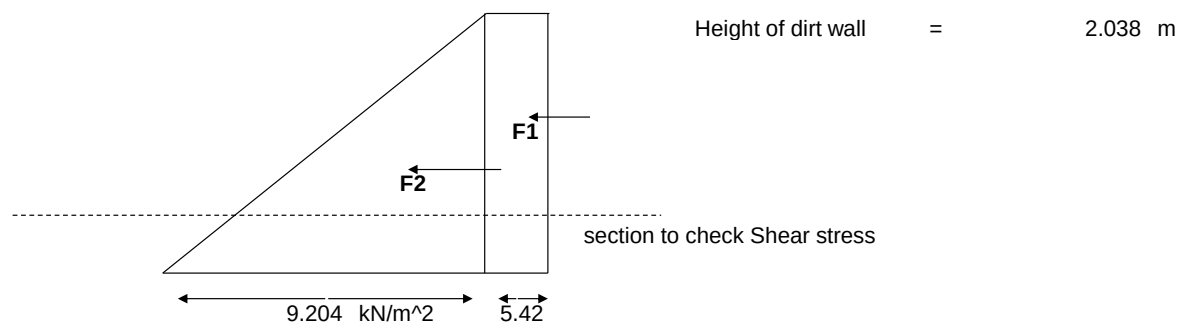
Braking Load

$$\text{Assuming 20\% braking Force i.e. } 0.2 \times 200 = 40.000 \text{ kN acting at 1.2 m above deck}$$

$$\text{Effective Width} = 2.79 \text{ m}$$

$$\text{Moment (due to Braking)} = \frac{40.000 \times 3.238}{2.79} = 46.416 \text{ kNm/m}$$

1c. EARTH PRESSURE



Normal Earth Pressure

Earth Pressure Diagram

$$\text{Intensity for rectangular portion} = 0.226 \times 20 \times 1.2 = 5.421 \text{ kN/m}^2$$

$$F1 = 5.421 \times 2.04 \times 1.00 = 11.045 \text{ kN/m}$$

$$\text{Intensity for triangular portion} = 0.2259 \times 20 \times 2.038 = 9.204 \text{ kN/m}^2$$

$$F2 = 0.50 \times 9.20 \times 2.038 \times 1.00 = 9.377 \text{ kN/m}$$

$$\text{Moment @ RL} = 1768.97 \text{ m (at dirt wall base)}$$

$$M1 = 11.045 \times 1.019 = 11.252 \text{ kN.m/m}$$

(Centre of pressure considered at an elevation of 0.42 x the height of the wall as per cl. 217.1 of IRC:6-2014)

$$M2 = 9.377 \times 0.856 = 8.024 \text{ kN.m/m}$$

Design Horizontal Forces (Normal Case):

$$\text{Load Factor For Live Load Surcharge} = 1.2$$

$$\text{Ultimate Moment due to Live Load Surcharge} = 13.502 \text{ kN.m/m}$$

$$\text{Load Factor For Earth Pressure} = 1.5$$

$$\text{Ultimate Moment due to Earth Pressure} = 12.036 \text{ kN.m/m}$$

$$\text{Load Factor For Braking Force} = 1.5$$

$$\text{Ultimate Moment due to Braking Force} = 69.624 \text{ kN.m/m}$$

Total Ultimate Moment	= 95.162 kN.m/m

Material Property:

Grade of Concrete	=	M 35
Characteristic Strength of Concrete, f_{ck}	=	35.00 Mpa at 28 days
Grade of Reinforcement	=	Fe 500
Yield Strength of Reinforcement, f_y or f_{yk}	=	500 N/mm ²
Design Yield Strength of Reinforcement, f_{yd}	=	434.783 N/mm ²
Modulus of Elasticity of Steel (E_s)	=	200000 N/mm ²

(a) Vertical steel on earth face

As per Clause 16.3.1 of IRC:112-2011

Adopting clear cover on either face	=	50 mm
Minimum Dia of Reinforcement	=	16 mm
Maximum Spacing of Steel	=	200 mm
Thickness of dirtwall	=	0.300 m
Available effective depth	=	300 - 50 - 8 = 242 mm

Check for Depth:

Mult	=	$0.165 \times f_{ck} \times b \times d^2$	=	95.16 kNm/m
Effective Depth of Cap Required (dreq)	=	$\text{SQRT}\left(\frac{95.16 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$	=	128.368 mm
Total Depth Required (Dreq)	=	186.37 mm		
Total Depth Provided (Dprov)	=	300.00 mm		OK
$R = M_u / (b \times d^2)$	=	1.625		

Area of Steel Required:

$\frac{p_t}{100} = \frac{A_{st_{req}}}{b \times d}$	=	$\frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$	=	0.004
$A_{st_{req}}$	=	958.246 mm ² /m		
As per Clause 16.3.1 of IRC:112-2011				
Minimum Reinforcement	=	$0.12/100 \times b \times D$	=	360 mm ² /m
Maximum ($A_{st_{req}}, A_{st_{min}}$)	=	958.246 mm ² /m		

Provide 16 mm dia bar @ 200 mm c/c as vertical steel at earth face.**Provide A_{st} = 1005 mm²/m) OK**

Percentage of Steel Provided	=	0.415 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis, X_m	=	$\frac{0.0035 \times d}{(0.0035 + f_{yd} / E_s)}$	=	$\frac{0.0035 \times 242}{0.0035 + 0.00217}$
	=	149.2797 mm		

Depth of Neutral Axis, =	$\frac{f_{yd} \times A_{st}}{0.36 \times f_{ck} \times b}$			
=	$\frac{435 \times 1005}{0.36 \times 35.00 \times 1000}$	=	34.707 mm	OK

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

z	=	$d - 0.416 \times X$	=	242 - 14.438
	=	227.562 mm		

Moment of Resistance of Section w.r.t. Steel (M_R)

M_R	=	$f_{yd} \times A_{st} \times z$	=	434.78 x 1005 x 227.56
	=	9.9E+07 Nmm	=	99.465 kNm/m > 95.16 kNm/m
				SAFE

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

(b) Horizontal steel

Refer Clause 16.3.2 of IRC:112-2011

Adopting distribution steel bars Dia.	=	10 mm
Minimum Area of Steel	=	0.001x 0.5 x b x D OR 25% of Ast on Vertical Face
0.001x0.5xbxD	=	150 mm ² /m OR 239.561 mm ² /m
Governing Ast	=	239.561 mm ² /m
Maximum Spacing of Bars	=	300 mm

Provide	10 mm dia bar @	200 mm c/c horizontal steel at non earth face.

Provided Ast	=	393 mm ² /m)	OK

(c) Vertical steel on other face

As per Clause 16.3.1 of IRC:112-2011

Minimum Reinforcement	=	0.12/100 b x D	=	360 mm ² /m

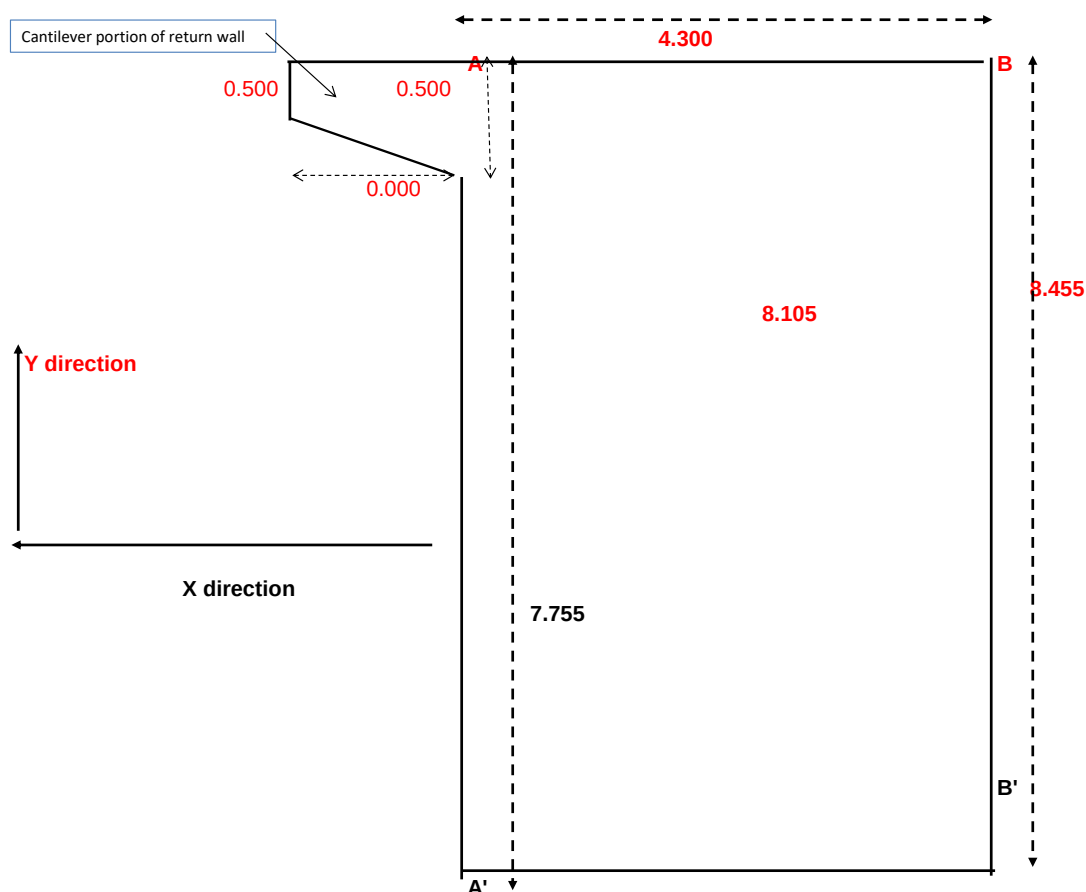
Provide	10 mm dia bar @	200 mm c/c as vertical steel at earth face.

Provided Ast=	393 mm ² /m)	OK

Design of Solid Return wall

THICKNESS OF SOLID RETURN WALL = 0.500 m

THICKNESS OF CANTILEVER RETURN WALL = 0.500 m

Width of Solid Return $a = 4.30$ mAvg. Height of Solid Return $b = 8.105$ m**a) Design of Solid Return wall***For design of return wall Load case 11.a & 11.d and their formulae given by Roark have been used.*

Here,

a/b	=	0.531			
a/b	=	0.5	β_1	=	0.631
a/b	=	0.75	β_1	=	1.246
			β_2	=	0.632
			β_2	=	1.186

For uniformly distributed load over entire plate

For, $a/b = 0.531$ $\beta_1 = 0.706$ $\beta_2 = 0.700$

Live Load Surcharge Intensity: $q = 0.2259 \times 20.00 \times 1.200 = 5.421 \text{ kN/m}^2$

$$\begin{aligned} \text{Max. } \sigma_b &= \frac{\beta_1 \times q \times b^2}{(t_1)^2} \\ \sigma_a &= \frac{\beta_2 \times q \times b^2}{(t_2)^2} \\ \sigma_b &= \frac{0.706 \times 5.421 \times 65.691}{0.250} \end{aligned}$$

At bottom edge = 1005.793 kN/m² = 1.006 MPa

$$\begin{aligned} \text{For } 1000 \text{ mm of width, } Z &= \frac{1000 \times 250000}{6} \\ &= 4.17\text{E}+07 \text{ mm}^3 \end{aligned}$$

Hence Moment /m width along Y direction -

$$\begin{aligned}
 \text{My /m width} &= 1.006 \times 4.167\text{E}+07 \\
 &= 41908047 \text{ Nmm/m} = 41.908 \text{ kN.m/m} \\
 \sigma a &= \frac{0.700 \times 5.421 \times 65.691}{0.250} \\
 &= 997 = 0.9966 \text{ MPa} \\
 \text{For } 1000 \text{ mm of height, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence, Moment /m height along X direction -

$$\begin{aligned}
 \text{Mx /m height} &= 0.9966 \times 4.167\text{E}+07 = 4.153\text{E}+07 \text{ Nmm/m} \\
 &= 41.525 \text{ kN.m/m}
 \end{aligned}$$

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.500	β_1 =	0.328	β_2 =	0.200
a/b =	0.75	β_1 =	0.537	β_2 =	0.276

$$\begin{aligned}
 \text{For, } a/b &= 0.531 \quad \beta_1 = 0.354 \\
 &\quad \beta_2 = 0.209 \\
 q &= 0.226 \times 20.00 \times 8.10 \\
 &= 36.613 \text{ kN/m}^2 \\
 \text{Max. } \sigma b &= \frac{\beta_1 \times q \times b^2}{(t_1)^2} \\
 \sigma a &= \frac{\beta_2 \times q \times b^2}{(t_2)^2} \\
 \sigma b &= \frac{0.354 \times 36.613 \times 65.691}{0.25} \\
 &= 3401.15 \text{ kN/m}^2 \\
 &= 3.401 \text{ MPa}
 \end{aligned}$$

$$\begin{aligned}
 \text{For } 1000 \text{ mm of width, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence Moment /m width along Y direction -

$$\begin{aligned}
 \text{My /m width} &= 3.401 \times 4.167\text{E}+07 \\
 &= 141714784 \text{ Nmm/m} = 141.715 \text{ kN.m/m} \\
 \sigma a &= \frac{0.209 \times 36.613 \times 65.691}{0.25} \\
 &= 2013.4 \text{ kN/m}^2 = 2.013 \text{ MPa}
 \end{aligned}$$

$$\begin{aligned}
 \text{For } 1000 \text{ mm of height, Z} &= \frac{1000 \times 250000}{6} \\
 &= 4.167\text{E}+07 \text{ mm}^3
 \end{aligned}$$

Hence Moment /m height along X direction -

$$\begin{aligned}
 \text{Mx /m height} &= 2.013 \times 4.167\text{E}+07 = 8.389\text{E}+07 \text{ Nmm/m} \\
 &= 83.893 \text{ kN.m/m}
 \end{aligned}$$

Total Moment in Solid Return Wall / m height = 125.418 kN.m/m

Total Moment in Solid Return Wall / m width = 183.623 kN.m/m

Final Design Moments:

Load Factor for Earth pressure	=	1.50
Load Factor for live load surcharge	=	1.20
Total Moment(Mx) in Solid Return Wall / m height	=	176 kN.m/m
Total Moment(My) in Solid Return Wall / m width	=	263 kN.m/m

Material Property:

- Refer Table No 6.5 of IRC : 112-2011

Grade of Concrete	=	M 35
Characteristic Strength of Concrete, fck	=	35.00 Mpa at 28 days
Grade of Reinforcement	=	Fe 500
Yield Strength of Reinforcement, fy or fyk	=	500.00 Mpa
Design Yield Strength of Reinforcement, fyd	=	434.78 Mpa (1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00 Mpa

1. Design of Face BB'**Moment in Solid Return /m height (including cantilever moment) =**

$$= 175.669 + 0.00$$

$$= 175.67 \text{ kN.m / m}$$

Adopting clear cover on either face	=	75 mm
Minimum Dia of Reinforcement	=	16 mm
Maximum Spacing of Steel	=	200 mm
Thickness of wall	=	0.500 m
Available effective depth	=	500 -75 -8
	=	417 mm

Check for Depth:

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 175.67 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \text{SQRT} \left(\frac{175.67 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$\text{Effective Depth of Cap Required (dreq)} = 174.410 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 257.41 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 500.00 \text{ mm}$$

OK

$$R = M_u / (b \times d^2) = 1.01$$

Area of Steel Required:

$$\frac{p_t}{100} = \frac{A_{st_{req}}}{b \times d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y}$$

$$A_{st_{req}} = 1002.960 \text{ mm}^2/\text{m}$$

$$\text{Minimum Reinforcement} = 0.12/100 \times b \times D = 600 \text{ mm}^2/\text{m}$$

As per Clause 16.3.1 of IRC:112-2011

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 1002.960 \text{ mm}^2/\text{m}$$

Provide 16 mm dia bar @ 200 mm c/c as Horizontal steel at earth face.**Provide Ast= 1005 mm²/m) OK**

$$\text{Percentage of Steel Provided} = 0.241 \%$$

Check for Moment of Resistance of section due to steel

$$\text{Limiting Depth of Neutral Axis , } X_m = \frac{0.0035 \times d}{(0.0035 + f_{yd} / E_s)}$$

$$= \frac{0.0035 \times 417}{0.0035 + 0.00217}$$

$$= 257.230 \text{ mm}$$

$$\text{Depth of Neutral Axis , } X = \frac{f_{yd} \times A_{st}}{0.36 \times f_{ck} \times b}$$

$$= \frac{434.78 \times 1005}{0.36 \times 35.00 \times 1000}$$

$$= 34.690 \text{ mm} \quad \boxed{\text{OK}}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned} z &= d - 0.416 \cdot X \\ &= 417 - 14.431 \\ &= 402.569 \text{ mm} \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned} \text{MR} &= f_{yd} \cdot A_{st} \cdot z \\ &= 434.78 \times 1005 \times 402.569 \\ &= 1.76\text{E}+08 \text{ Nmm} \\ &= 175.959 \text{ kNm/m} > 175.67 \text{ kNm/m} \end{aligned}$$

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

Provide 12 mm dia bar @ 200 mm c/c as Horizontal steel at non earth face.

$$\text{Provided } A_{st} = 565 \text{ mm}^2/\text{m}$$

2. Design for Face A'B'

$$\text{Moment in Solid Return /m width} = 262.86 \text{ kN.m / m}$$

$$\begin{aligned} \text{Adorting clear cover on either face} &= 75 \text{ mm} \\ \text{Minimum Dia of Reinforcement} &= 20 \text{ mm} \\ \text{Maximum Spacing of Steel} &= 150 \text{ mm} \\ \text{Thickness of wall} &= 0.500 \text{ m} \\ \text{Available effective depth} &= 500 \text{ mm} \quad -75 \quad -16 \quad -10 \\ &= 399 \text{ mm} \end{aligned}$$

Check for Depth:

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 262.86 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \text{SQRT} \left(\frac{262.86 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$\text{Effective Depth of Cap Required (dreq)} = 213.348 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 298.35 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 500.00 \text{ mm} \quad \boxed{\text{OK}}$$

$$R = M_u / (b \cdot d^2) = 1.65$$

Area of Steel Required:

$$\begin{aligned} \frac{p_t}{100} &= \frac{A_{st_{req}}}{b \cdot d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y} \\ &= 0.004 \\ A_{st_{req}} &= 1607.052 \text{ mm}^2/\text{m} \end{aligned}$$

$$\begin{aligned} \text{Minimum Reinforcement} &= 0.12/100 \cdot b \times D \\ &= 600 \text{ mm}^2/\text{m} \end{aligned} \quad \text{As per Clause 16.3.1 of IRC:112-2011}$$

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 1607.052 \text{ mm}^2/\text{m}$$

Provide 20 mm dia bar @ 150 mm c/c as vertical steel at earth face.

Provide A_{st} = 2094 mm²/m) OK

$$\text{Percentage of Steel Provided} = 0.5249 \%$$

Provide 12 mm dia bar @ 300 mm c/c as Vertical steel at non earth face.

Check for Moment of Resistance of section due to steel

$$\begin{aligned} \text{Limiting Depth of Neutral Axis , } X_m &= \frac{0.0035 \cdot d}{(0.0035 + f_{yd} / E_s)} \\ &= \frac{0.0035 \times 399}{0.0035 + 0.00217} \\ &= 246.13 \text{ mm} \end{aligned}$$

$$\begin{aligned}
 \text{Depth of Neutral Axis, } X &= \frac{f_{yd} \cdot A_{st}}{0.36 \cdot f_{ck} \cdot b} \\
 &= \frac{434.78}{0.36} \times \frac{2094}{35.00 \times 1000} \\
 &= 72.270 \text{ mm} \quad \boxed{\text{OK}}
 \end{aligned}$$

Lever Arm (z) between Compressive Force (C) and Tensile Force (T)

$$\begin{aligned}
 z &= d - 0.416 \cdot X \\
 &= 399 - 30.064 \\
 &= 368.94 \text{ mm}
 \end{aligned}$$

Moment of Resistance of Section w.r.t. Steel (MR)

$$\begin{aligned}
 MR &= f_{yd} \cdot A_{st} \cdot z \\
 &= 434.78 \times 2094 \times 368.936 \\
 &= 3.36E+08 \text{ Nmm} \\
 &= 335.955 \text{ kNm/m} > 262.86 \text{ kNm/m}
 \end{aligned}$$

Moment of Resistance of Shaft is More than Design Bending Moment , HENCE SHAFT IS SAFE IN BENDING

b) Cantilever Portion of Return Wall

$$\begin{aligned}
 \text{Self-weight of cantilever portion of return wall} &= 6 \text{ kN/m} \\
 \text{Crash Barrier weight} &= 10.0 \text{ kN/m} \\
 \text{Total Load} &= 16 \text{ kN/m} \\
 \text{Moment at Cantilever Face} &= 0 \text{ kNm} \\
 \text{Load Factor} &= 1.35 \\
 \text{Design Moment} &= 0 \text{ kNm} \\
 \text{Effective Depth} &= 444.000 \text{ mm}
 \end{aligned}$$

$$R = M_u / (b \cdot d^2) = 0.00$$

Area of Steel Required:

$$\begin{aligned}
 \frac{p_t}{100} &= \frac{A_{st_{req}}}{b \cdot d} = \frac{f_{ck} \{ 1 - \sqrt{1 - 4.598 R / f_{ck}} \}}{2 f_y} \\
 &= \frac{0.000}{2} \\
 A_{st_{req}} &= 0.000 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 \text{Minimum Reinforcement} &= 0.12/100 \cdot b \times D \quad \text{As per Clause 16.3.1 of IRC:112-2011} \\
 &= 266.4 \text{ mm}^2
 \end{aligned}$$

$$\text{Maximum (} A_{st_{req}}, A_{st_{min}} \text{)} = 266.4000 \text{ mm}^2$$

Provide	12	4	=	452 mm ²

ANNEXURE-I (LOAD CALCULATION-DL & SIDL)

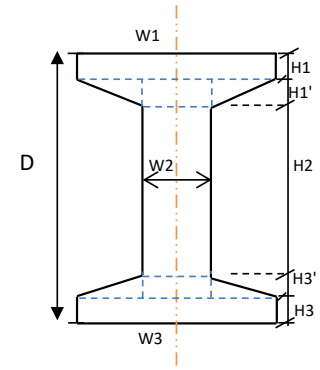
SECTIONAL PROPERTIES OF OUTER GIRDER & INNER GIRDER

1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	100
H2	=	1350
H3'	=	0
H3	=	0
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	1350	405000
3' (Tri.)	500	0	0
3' (Rect.)	300	0	0
3 (Rect.)	800	0	0
			620000



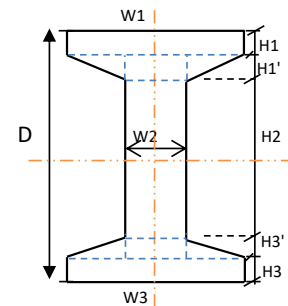
Total Area of Section A = 0.620 m²

2. SECTIONAL PROPERTIES OF GIRDERS -VARRYING PORTION

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	64
H2	=	1386
H3'	=	0
H3	=	0
W1	=	1000
W2	=	550
W3	=	550

Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	450	64	14464
1' (Rect.)	550	64	35357
2 (Rect.)	550	1386	762143
3' (Tri.)	0	0	0
3' (Rect.)	550	0	0
3 (Rect.)	550	0	0
			961964



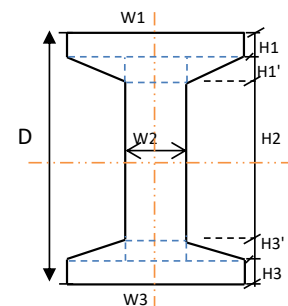
Total Area of Section A = 0.962 m²

3. SECTIONAL PROPERTIES OF GIRDERS (AT SUPPORT)

(Dimension in mm)

D	=	1600
H1	=	150
H1'	=	29
H2	=	1421
H3'	=	0.00
H3	=	0.00
W1	=	1000
W2	=	800
W3	=	800

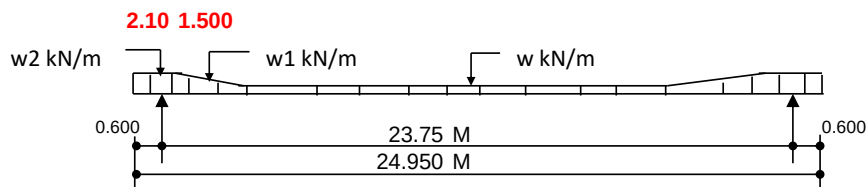
Element	Width (mm)	Height (mm)	A (mm ²)
1 (Rect.)	1000	150	150000
1' (Tri.)	200	29	2857
1' (Rect.)	800	29	22857
2 (Rect.)	800	1421	1137143
3' (Tri.)	0	0	0
3' (Rect.)	800	0	0
3 (Rect.)	800	0	0
			1312857



Total Area of Section A = 1.313 m²

1.0 Self weight of Inner Girder & Outer Girder

Length of the girder = 24.950 m



UDL , w	=	15.5000 kN/m
Avg. , w ₁	=	24.0491 kN/m
UDL , w ₂	=	32.8214 kN/m
Supp. Reaction , R	=	242.56 kN

2.0 Weight of cast-in-situ deck slab

Effective Width of Slab	=	2.850 m	For Outer Girder
Effective Width of Slab	=	2.80 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	=	15.675 kN/m	For Outer Girder
on entire length of girder	=	15.400 kN/m	For Inner Girder

Supp. Reaction , R	=	195.546 kN	For Outer Girder
	=	192.115 kN	For Inner Girder

weight of deck slab has been Increase by 5 %

Supp. Reaction , R	=	205.323 kN	For Outer Girder
	=	201.721 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	=	44.745 kN	on outer bearings
	=	43.960 kN	on Inner bearings

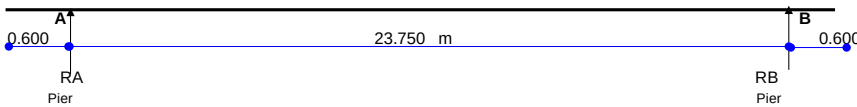
4.0 Weight due to wearing coat

Wearing coat width	=	2.350 m	For Outer Girder
Wearing coat width	=	2.800 m	For Inner Girder
Weight of wearing coat including future	=	2.2 kN/m ²	
UDL due to Wearing coat	=	5.170 kN/m	For Outer Girder
on entire length of girder	=	6.160 kN/m	For Inner Girder
Supp. Reaction , R	=	64.496 kN	For Outer Girder
	=	76.846 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 = **2 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

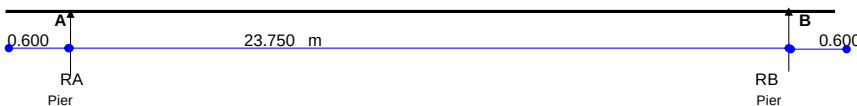
1st Train	Total Load	500.0kN				
					392.345	107.655
114.000	3.200	0.000	-0.600	-68.400		
114.000	1.200	1.200	0.600	68.400		
68.000	4.300	5.500	4.900	333.200		
68.000	3.000	8.500	7.900	537.200		
68.000	3.000	11.500	10.900	741.200		
68.000	3.000	14.500	13.900	945.200		

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

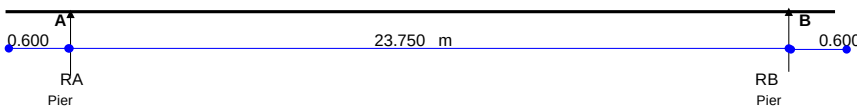
1st Train	Total Load	1000.0kN				
					809.524	190.476
170.000		0.000	-0.600	-102.000		
170.000	1.370	1.370	0.770	130.900		
170.000	3.050	4.420	3.820	649.400		
170.000	1.370	5.790	5.190	882.300		
120.000	2.130	7.920	7.320	878.400		
120.000	1.520	9.440	8.840	1060.800		
80.000	3.960	13.400	12.800	1024.000		

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

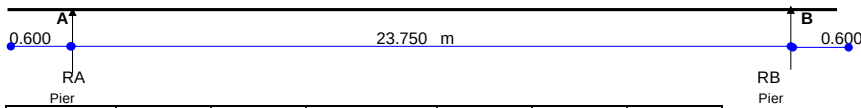
1st Train	Total Load	554.0kN				
					341.524	212.476
68.000		0.000	-0.600	-40.800		
68.000	3.000	3.000	2.400	163.200		
68.000	3.000	6.000	5.400	367.200		
68.000	3.000	9.000	8.400	571.200		
114.000	4.300	13.300	12.700	1447.800		
114.000	1.200	14.500	13.900	1584.600		
27.000	3.200	17.700	17.100	461.700		
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 = 8.500 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.	=	4.25	-0.5	-0.4	-0.9	0	0	=	2.45 m
Maximum Reaction (RA)	=	392.345	KN	Corr. MT	=	961.246	kNm		
Minimum Reaction (RB)	=	107.655	KN	Corr. MT	=	263.754	kNm		

2-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-0.85	0	=	0.70 m
Maximum Reaction (RA)	=	784.691	KN	Corr. MT	=	549.283	kNm		
Minimum Reaction (RB)	=	215.309	KN	Corr. MT	=	150.717	kNm		

3-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-1.7	-0.9	=	-1.05 m
Maximum Reaction (RA)	=	1177.036 KN	Corr. MT	=	-1235.888 kNm				
Minimum Reaction (RB)	=	322.964 KN	Corr. MT	=	-339.112 kNm				

4-CLASS A

transverse ecc.	=	4.25	-0.5	-0.4	-1.8	-1.7	-1.8	-0.85	=	-2.8 m
Maximum Reaction (RA)	=	1569.381 KN	Corr. MT	=	-4394.267 kNm					
Minimum Reaction (RB)	=	430.619 KN	Corr. MT	=	-1205.733 kNm					

1- 70RW

Transverse Ecc.	for 70RW	=	4.25	-0.5	-1.2	-1.395	=	1.155 m

Maximum Reaction (RA)	=	809.524 KN	Corr. MT	=
Minimum Reaction (RB)	=	190.476 KN	Corr. MT	=

1-CLASS A + 1-70RW

Transverse Ecc.	for Class A	=	4.25	-0.5	-0.4	-0.9	=	2.45 m			
	for 70RW	=	4.25	-0.5	-0.4	-1.8	-0.25	-1.2	-1.395	=	-1.295
Maximum Reaction (RA)	=	1151.048 KN	Corr. MT	=	341.52 x	2.45 +	809.52 x	-1.295	=	-212	
Minimum Reaction (RB)	=	402.952 KN	Corr. MT	=	212.476	2.45 +	190.476	-1.295	=	274	

2-CLASS A + 1-70RW

Transverse Ecc.	for 1st Class A	=	2.45 m								
	for 2nd Class A	=	2.45	-0.90	-0.85	0	=	0.7 m			
	for 70RW	=	0.7	-0.85		-1.8	-0.25	-1.2	-1.395	=	-4.795
Maximum Reaction (RA)	=	1492.573 KN	Corr. MT	=	341.52 x	2.45 +	341.52 x	0.700	=	-2806	
						+	809.52 x	-4.795			
Minimum Reaction (RB)	=	615.427 KN	Corr. MT	=	212.476	2.45 +	212.48 x	0.700	=	-244	
						+	190.48 x	-4.795			

1- 70RW + 1-CLASS A

Transverse Ecc.		for 70RW	=	4.25	-0.5	-1.2	-1.395	=	1.155 m	
		for Class A	=	4.25	-0.5	-7.25	-0.25	-0.9 =		-4.650 m
Maximum Reaction (RA)	=	1151.048 KN	Corr. MT	=	809.52 x	1.155 +	341.52 x	-4.650	=	-653
Minimum Reaction (RB)	=	402.952 KN	Corr. MT	=	190.476	1.155 +	212.476	-4.650	=	-768

1- 70RW + 2-CLASS A

Transverse Ecc.	for 70RW	=	1.155 m							
	for 1st Class	=	-4.650 m							
	for 2nd Class	=	-4.650	-0.9	-0.85	=		-6.400 m		
Maximum Reaction (RA)	=	1492.573 KN	Corr. MT	=	809.52 x	1.155 +	341.52 x	-4.650	=	-2839
					+		341.52 x	-6.400		
Minimum Reaction (RB)	=	615.427 KN	Corr. MT	=	190.476	1.155 +	212.476	-4.650	=	-2128
					+		212.48 x	-6.400		

2- 70RW

Transverse Ecc.	for 1-70RW for 2-70RW	= =	1.16 m 1.155	-1.395	-1.2	-1.40	=	-2.835	m
Maximum Reaction (RA)	=	1619.048 KN	Corr. MT	=	809.52 x	1.155 +	809.52 x	-2.835	= -1360
Minimum Reaction (RB)	=	380.952 KN	Corr. MT	=	190.476	1.155 +	190.476	-2.835	= -320

Summary of Reaction and Corresponding moments

Max Reaction Cases	Reaction (RA) , (kN)	Corr. MT (kNm)
1-CLASS A	392.345	961.246
2-CLASS A	784.691	549.283
3-CLASS A	0.000	0.000
4-CLASS A	0.000	0.000
1- 70RW	809.524	935.000
1-CLASS A + 1-70RW	0.000	0.000
1- 70RW + 1-CLASS A	0.000	0.000
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	263.754
215.309	150.717
0.000	0.000
0.000	0.000
190.476	220.000
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000
0.000	0.000

Maximum Reaction & Corresponding Transverse Moment

Reaction RA = 809.524 KN
 Reaction RB = 190.476 KN

& Corresponding moment = 935.000 kNm
 & Corresponding moment = 220.000 kNm

Maximum Transverse Moment & Corresponding Reaction

Reaction RA = 392.345 KN
 Reaction RB = 107.655 KN

& Max moment = 961.246 kNm
 & Max moment = 263.754 kNm

***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 –8.5 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	=	7.50	m	
* Width of Footpath	=	0.00	m	both side
* Overall Deck Width	=	8.50	m	
* Cross Slope of superstructure	=	2.50	%	(bidirectional)
* Camber	=	2.50	%	(bidirectional)
* Nos. of Main Longitudinal Girders	=	3	Nos.	
* c/c Distance of Longitudinal Girder	=	2.800	m	
* Dist. bet. c/c outer girder and edge of deck	=	1.45	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 Ist 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)

- * 56 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)

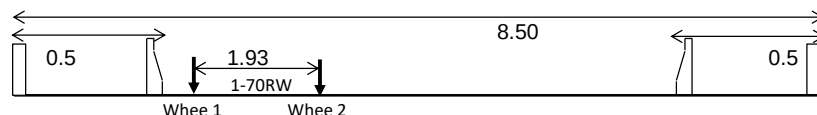
Superstructure meant for 2-lane carriageway has been designed for governing effect of CWLL loading as specified in IRC:6-2017 with appropriate lane reduction factor and suitable allowances for impact so placed on the span to produce maximum stresses.

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)
- * 2-Class A

1) 1-70RW

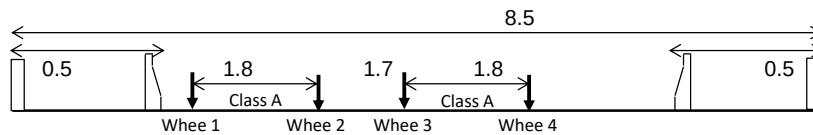
Load position of 1- 70RW



$$\text{Wheel 1} = 0.5 + 1.2 + 0.43 = 2.13 \text{ m}$$

$$\text{Wheel 2} = 2.13 + 1.93 = 4.06 \text{ m}$$

2) 2-Class A

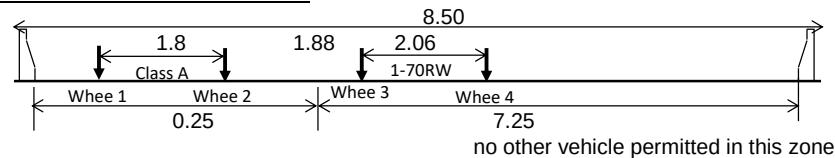


$$\begin{aligned} \text{Wheel 1} &= 0.5 + 0.15 + 0.25 = 0.90 \text{ m} \\ \text{Wheel 2} &= 0.9 + 1.8 = 2.7 \text{ m} \\ \text{Wheel 3} &= 2.7 + 1.7 = 4.4 \text{ m} \\ \text{Wheel 4} &= 4.4 + 1.8 = 6.2 \text{ m} \end{aligned}$$

3) 1- Lane Class A + 1-70RW

Not Applicable for this bridge

Load position of 1-Lane Class A + 1- 70RW



From left Side

$$\begin{aligned} \text{Wheel 1} &= 0.5 + 0.15 + 0.25 = 0.9 \text{ m} \\ \text{Wheel 2} &= 0.9 + 1.8 = 2.7 \text{ m} \\ \text{Wheel 3} &= 2.7 + -1.52 = 1.18 \text{ m} \\ \text{Wheel 4} &= 1.18 + 2.06 = 3.24 \text{ m} \end{aligned}$$

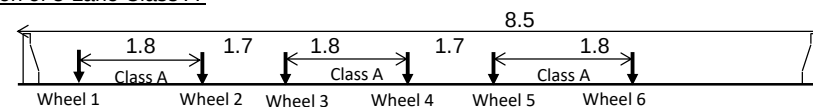
From Right Side

$$\begin{aligned} \text{Wheel 4} &= 0.5 + 1.2 + 0.43 = 2.13 \text{ m} \\ \text{Wheel 3} &= 2.13 + 2.06 = 4.19 \text{ m} \\ \text{Wheel 2} &= 7.75 + 0.25 = 8 \text{ m} \\ \text{Wheel 1} &= 8 + 1.8 = 9.8 \text{ m} \end{aligned}$$

4) 3- Lane Class A

Not Applicable for this bridge

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 critirical 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

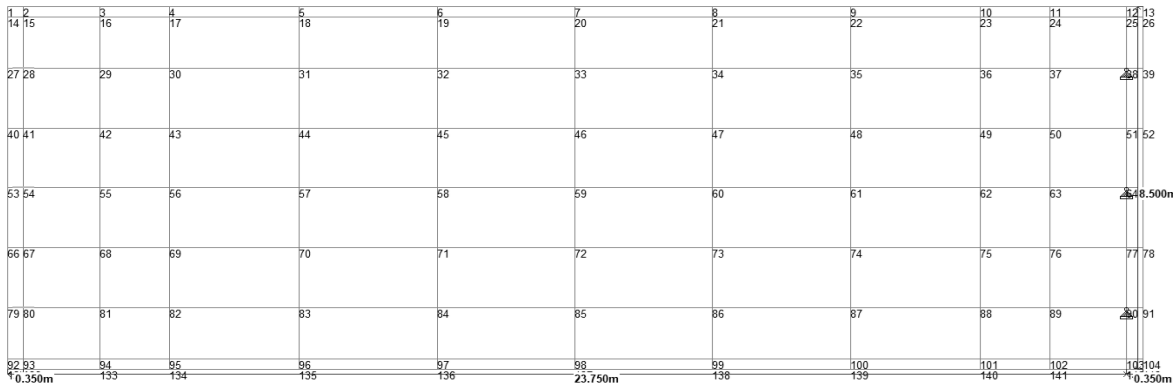
Deck slab has been analysed along transverse direction as a continuous beam with overhangs supported on longitudinal girders and diaphragms. Live load effects shall be taken based on effective width method.

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

1343	2	153	3	163	4	173	5	183	6	193	7	203	8	213	9	223	10	233	11	243	12
1344	14	154	15	164	16	174	17	184	18	194	19	204	20	214	21	224	22	234	23	244	24
25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
1345	155	165	175	185	195	205	215	225	235	245	255	265	275	285	295	305	315	325	335	345	355
37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58
1346	156	166	176	186	196	206	216	226	236	246	256	266	276	286	296	306	316	326	336	346	356
49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70
1347	157	167	177	187	197	207	217	227	237	247	257	267	277	287	297	307	317	327	337	347	357
61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82
1348	158	168	178	188	198	208	218	228	238	248	258	268	278	288	298	308	318	328	338	348	358
73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94
1349	159	169	179	189	199	209	219	229	239	249	259	269	279	289	299	309	319	329	339	349	359
95	96	97	98	99	100	101	102	103	104	105	106	107	108	109	110	111	112	113	114	115	116
1350	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360
117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138
1351	161	171	181	191	201	211	221	231	241	251	261	271	281	291	301	311	321	331	341	351	361
119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140
1352	162	172	182	192	202	212	222	232	242	252	262	272	282	292	302	312	322	332	342	352	362
121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142
1353	163	173	183	193	203	213	223	233	243	253	263	273	283	293	303	313	323	333	343	353	363
123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144
1354	164	174	184	194	204	214	224	234	244	254	264	274	284	294	304	314	324	334	344	354	364
125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146
1355	165	175	185	195	205	215	225	235	245	255	265	275	285	295	305	315	325	335	345	355	365
127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148
1356	166	176	186	196	206	216	226	236	246	256	266	276	286	296	306	316	326	336	346	356	366
129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150
1357	167	177	187	197	207	217	227	237	247	257	267	277	287	297	307	317	327	337	347	357	367
131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152
1358	168	178	188	198	208	218	228	238	248	258	268	278	288	298	308	318	328	338	348	358	368
133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154
1359	169	179	189	199	209	219	229	239	249	259	269	279	289	299	309	319	329	339	349	359	369
135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156
1360	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370
137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158
1361	171	181	191	201	211	221	231	241	251	261	271	281	291	301	311	321	331	341	351	361	371
139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160
1362	172	182	192	202	212	222	232	242	252	262	272	282	292	302	312	322	332	342	352	362	372
141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162
1363	173	183	193	203	213	223	233	243	253	263	273	283	293	303	313	323	333	343	353	363	373
143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164
1364	174	184	194	204	214	224	234	244	254	264	274	284	294	304	314	324	334	344	354	364	374
145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166
1365	175	185	195	205	215	225	235	245	255	265	275	285	295	305	315	325	335	345	355	365	375
147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168
1366	176	186	196	206	216	226	236	246	256	266	276	286	296	306	316	326	336	346	356	366	376
149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170
1367	177	187	197	207	217	227	237	247	257	267	277	287	297	307	317	327	337	347	357	367	377
151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172
1368	178	188	198	208	218	228	238	248	258	268	278	288	298	308	318	328	338	348	358	368	378
153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174
1369	179	189	199	209	219	229	239	249	259	269	279	289	299	309	319	329	339	349	359	369	379
155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176
1370	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380
157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178
1371	181	191	201	211	221	231	241	251	261	271	281	291	301	311	321	331	341	351	361	371	381
159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180
1372	182	192	202	212	222	232	242	252	262	272	282	292	302	312	322	332	342	352	362	372	382
161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182
1373	183	193	203	213	223	233	243	253	263	273	283	293	303	313	323	333	343	353	363	373	383
163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184
1374	184	194	204	214	224	234	244	254	264	274	284	294	304	314	324	334	344	354	364	374	384
165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186
1375	185	195	205	215	225	235	245	255	265	275	285	295	305	315	325	335	345	355	365	375	385
167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188
1376	186	196	206	216	226	236	246	256	266	276	286	296	306	316	326	336	346	356	366	376	386
169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190
1377	187	197	207	217	227	237	247	257	267	277	287	297	307	317	327	337	347	357	367	377	387
171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192

SECTIONAL PROPERTIES OF OUTER GIRDER

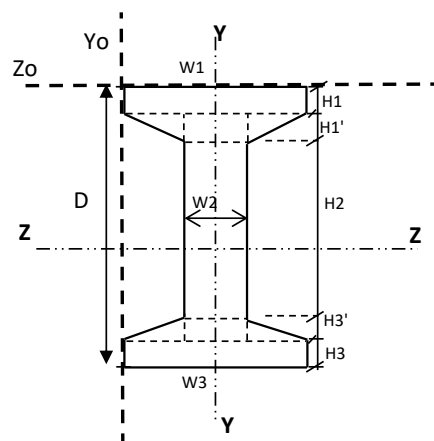
1. SECTIONAL PROPERTIES OF GIRDERS -MID SPAN

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	=	0.7813 m	
c.g. of Section from Bottom of Girder	Ybg	=	0.819 m	
Moment of Inertia of Section	Iz	=	0.221756 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	

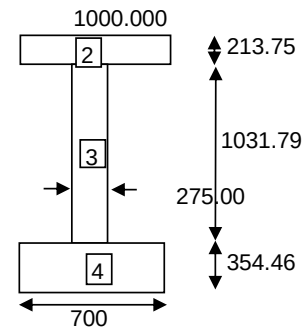
Moment of Inertia of Section $I_{yy} = 0.026 \text{ m}^4$

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	292.365	4.39E+07
1' (Tri.)	725	100	36250	184.031	6.67E+06
1' (Rect.)	275	100	27500	167.365	4.60E+06
2 (Rect.)	275	117.365	32275	58.682	1.89E+06
$\Sigma (Ay)p$					5.70E+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

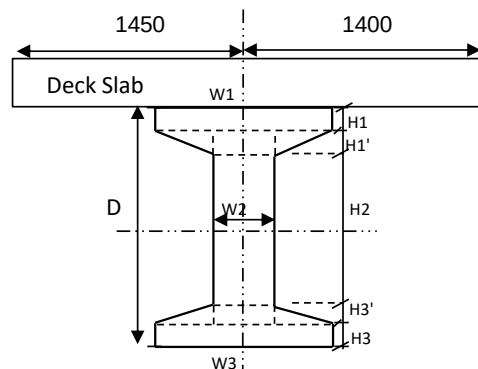
Effective Width of Flange

$$\begin{aligned} bw &= b = 2.850 \text{ m} \\ &= w2 = 275 \text{ mm} \\ &= b1 = 1312.5 \text{ mm} \\ &= b2 = 1262.5 \text{ mm} \\ &= lo = 23.75 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{effective width of cantilever on one end} &= \min (0.2 \times b1 + 0.1 \times lo , 0.2 \times lo , b1) \\ \min (2.6375 , 4.75 , 1.3125) &= 1.3125 \\ \text{beff, 1} &= 1.3125 \end{aligned}$$

$$\begin{aligned} \text{effective width of cantilever on other end} &= \min (0.2 \times b2 + 0.1 \times lo , 0.2 \times lo , b2) \\ \min (2.6275 , 4.75 , 1.2625) &= 1.263 \\ \text{beff, 2} &= 1.263 \end{aligned}$$

$$\begin{aligned} \text{Total effective width of flange beff} &= 1.313 + 1.2625 + 0.275 \\ &= 2.850 \text{ m} \end{aligned}$$



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

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	2850	220	627000	110.00	6.897E+07	7.587E+09	2.529E+09	1.012E+10
Girder			723125	1001.27	7.240E+08	7.250E+11	2.2176E+11	9.467E+11
			1350125		7.930E+08	7.326E+11	2.243E+11	9.568E+11

Total Area of Section A = 1.350 m²

c.g. of Section from Top of Slab	Yts	=	0.587 m	
c.g. of Section from Top of Girder	Ytg	=	0.367 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.233 m	
Moment of Inertia of Section	Iz	=	0.491 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.836 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.337 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.398 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6140.000 mm	
Total perimeter of composite girder		=	11812.297 mm	

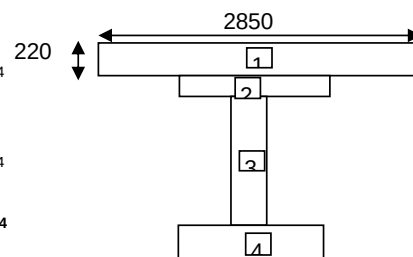
2b. Calculation of Sectional Properties about Minor Axis Y-Y

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	2850	220	627000	25.00	1.568E+07	3.919E+08	4.244E+11	4.248E+11
Girder			723125	0.00	0.000E+00	0.000E+00	2.5873E+10	2.587E+10
			1350125		1.568E+07	3.919E+08	4.503E+11	4.507E+11

Moment of Inertia of Section $I_{yy} = 0.451 \text{ m}^4$

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}	=	$b * d^3/6$
		=	5.06E+09 mm ⁴
Total Ixx for Composite Girder		=	2.13E+10 mm⁴

**2d. First moment of Area Above Centroid of Composite Section (Ay)c****Property of Composite Girder For Mid Span**

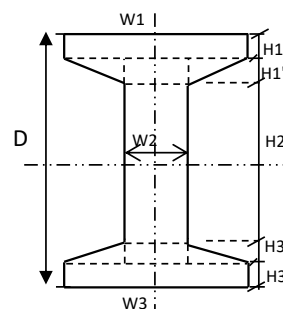
Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid (mm)	(Ay)c (mm ³)
Slab	2850	220	627000	477.365	2.99E+08
1 (Rect.)	1000	150	150000	292.365	4.39E+07
1' (Tri.)	725	100	72500	184.031	1.33E+07
1' (Rect.)	275	100	27500	167.365	4.60E+06
2 (Rect.)	275	117	32275.306	58.682	1.89E+06
			$\Sigma (Ay)c$		3.63E+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	342.673	5.14E+07
1' (Tri.)	513	71	36228.45	244.110	8.84E+06
1' (Rect.)	488	71	34461.21	232.328	8.01E+06
2 (Rect.)	488	197	96029.38	98.492	9.46E+06
				Σ (Ay)p	7.77E+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	2850	220	627000	110	6.897E+07	7.587E+09	2.529E+09	1.012E+10
Girder			936083	991.11	9.278E+08	9.195E+11	2.400E+11	1.159E+12
			1563083		9.967E+08	9.271E+11	2.425E+11	1.170E+12

Total Area of Section	A	=	1.563 m ²	
c.g. of Section from Top of Slab	Yts	=	0.638 m	
c.g. of Section from Top of Girder	Ytg	=	0.418 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.182 m	
Moment of Inertia of Section	Iz	=	0.534 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.837 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.279 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.452 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6140.000 mm	
Total perimeter of composite girder		=	11540.372 mm	

First moment of Area Above Centroid of Composite Section (Ay)c

Property of Composite Girder at Section L/8

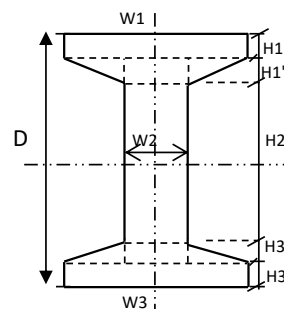
Element	Width (mm)	Height (mm)	A (mm ²)	y from centroid mm	(Ay)c mm ³
Slab	2850	220	627000.00	527.673	3.31E+08
1 (Rect.)	1000	150	150000.00	342.673	5.14E+07
1' (Tri.)	513	71	36228.45	244.110	8.84E+06
1' (Rect.)	488	71	34461.21	232.328	8.01E+06
2 (Rect.)	488	197	96029.38	98.492	9.46E+06
				Σ (Ay)c	4.09E+08

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

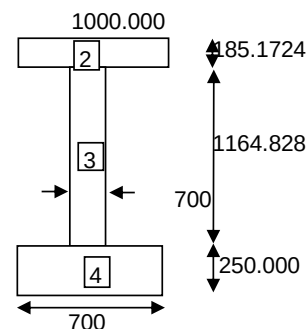
Moment of Inertia of Section Iyy = 0.0549 m⁴

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	387.362	5.81E+07
1' (Tri.)	300	41	12413.79	298.568	3.71E+06
1' (Rect.)	700	41	28965.52	291.672	8.45E+06
2 (Rect.)	700	271	189687.54	135.491	2.57E+07
$\Sigma (Ay)p$					9.60E+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	2850	220	627000	110	6.897E+07	7.587E+09	2.529E+09	1.012E+10
Girder			1171207	988.77	1.158E+09	1.145E+12	2.640E+11	1.409E+12
			1798207		1.227E+09	1.153E+12	2.666E+11	1.419E+12

Total Area of Section A = 1.798 m²
 c.g. of Section from Top of Slab Yts = 0.682 m
 c.g. of Section from Top of Girder Ytg = 0.462 m
 c.g. of Section from Bottom of Girder Ybg = 1.138 m
 Moment of Inertia of Section Iz = 0.582 m⁴ (Ip - A Yt²)
 Section Modulus w.r.to Top of Slab Zts = 0.853 m³
 Section Modulus w.r.to Top of Girder Ztg = 1.259 m³
 Section Modulus w.r.to Bottom of Girder Zbg = 0.512 m³
 Perimeter of Deck slab excluding top flange surface,uslab = 6140.000 mm
 Total perimeter of composite girder = 11268.447 mm

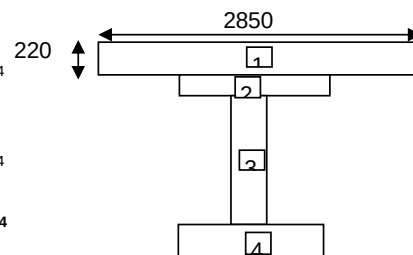
6b. Calculation of Sectional Properties about Minor Axis Y-Y

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	2850	220	627000	25.00	1.568E+07	3.919E+08	4.244E+11	4.248E+11
Girder			1171207	0.00	0.000E+00	0.000E+00	5.4947E+10	5.495E+10
			1798207		1.568E+07	3.919E+08	4.793E+11	4.797E+11

Moment of Inertia of Section Iyy = 0.480 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 = 5.06E+09 mm⁴
Total Ixx for Composite Girder = 9.79E+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	2850	220	627000	572.362	3.59E+08
1 (Rect.)	1000	150	150000	387.362	5.81E+07
1' (Tri.)	300	41	12413.79	298.568	3.71E+06
1' (Rect.)	700	41	28965.52	291.672	8.45E+06
2 (Rect.)	700	271	189687.54	135.491	2.57E+07
$\Sigma (Ay)c$					4.55E+08

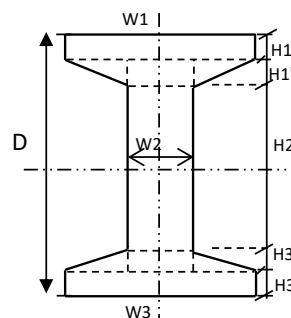
SECTIONAL PROPERTIES OF INNER GIRDER

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

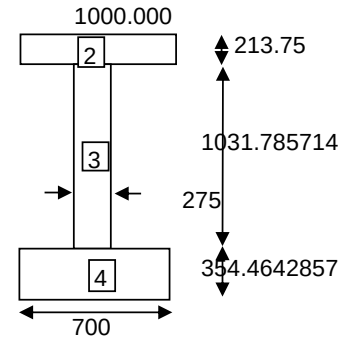
Moment of Inertia of Section	Iyy	=	0.026 m ⁴

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 = 2.80 m

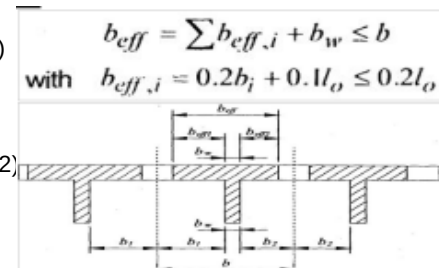
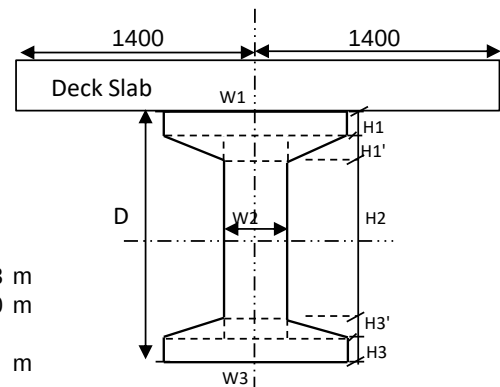
Governing width of flange = m

bw	=	b	=	2.800 m
		w2	=	275 mm
		b1	=	1262.5 mm
		b2	=	1262.5 mm
		lo	=	23.75 m

effective width of cantilever on one end = $\min (0.2 \times b_1 + 0.1 \times l_o, 0.2 \times l_o, b_1)$
 $\min (2.6275, 4.75, 1.2625) = 1.2625$
 $b_{eff, 1} = 1.2625$

effective width of cantilever on other end = $\min (0.2 \times b_2 + 0.1 \times l_o, 0.2 \times l_o, b_2)$
 $\min (2.6275, 4.75, 1.2625) = 1.263$
 $b_{eff, 2} = 1.263$

Total effective width of flange $b_{eff} = 1.263 + 1.2625 + 0.275 = 2.80 \text{ m}$



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	2800	220	616000	110.00	6.776E+07	7.454E+09	2.485E+09	9.938E+09
Girder			723125	1001.27	7.240E+08	7.250E+11	2.2176E+11	9.467E+11
			1339125		7.918E+08	7.324E+11	2.242E+11	9.567E+11

Total Area of Section	A	=	1.339 m ²	
c.g. of Section from Top of Slab	Yts	=	0.591 m	
c.g. of Section from Top of Girder	Ytg	=	0.371 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.229 m	
Moment of Inertia of Section	Iz	=	0.488 m ⁴	(Ip - A Yt ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.826 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.316 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.398 m ³	
Perimeter of Deck slab excluding top flange surface,uslab		=	6040.000 mm	
Total perimeter of composite girder		=	11712.297 mm	

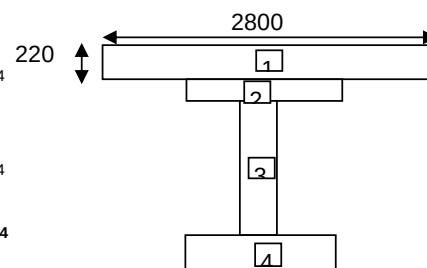
2b. Calculation of Sectional Properties about Minor Axis Y-Y

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	2800	220	616000	0.00	0.000E+00	0.000E+00	4.025E+11	4.025E+11
Girder			723125	0.00	0.000E+00	0.000E+00	2.5873E+10	2.587E+10
			1339125		0.000E+00	0.000E+00	4.283E+11	4.283E+11

Moment of Inertia of Section	Iyy	=	0.428 m⁴

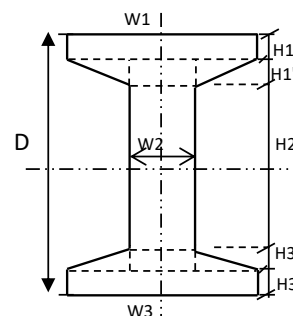
2c. Calculation of Sectional Properties about X-X Axis

Moment of Inertia of Precast Girde Ixx	=	1.63E+10 mm ⁴
Moment of Inertia of Deck Slab Ixxslab	=	b * d ³ /6
	=	4.97E+09 mm ⁴
Total Ixx for Composite Girder	=	2.12E+10 mm⁴

**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 -VARRYING 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	2800	220	616000	110	6.776E+07	7.454E+09	2.485E+09	9.938E+09
Girder			936083	991.11	9.278E+08	9.195E+11	2.400E+11	1.159E+12
			1552083		9.955E+08	9.270E+11	2.424E+11	1.169E+12

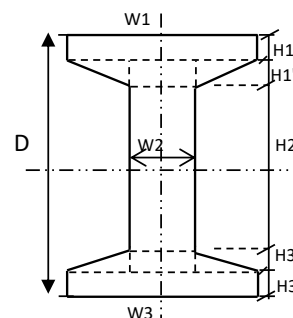
Total Area of Section	A	=	1.552 m ²	
c.g. of Section from Top of Slab	Yts	=	0.641 m	
c.g. of Section from Top of Girder	Ytg	=	0.421 m	
c.g. of Section from Bottom of Girder	Ybg	=	1.179 m	
Moment of Inertia of Section	Iz	=	0.531 m ⁴	(I _p - A Y _t ²)
Section Modulus w.r.to Top of Slab	Zts	=	0.828 m ³	
Section Modulus w.r.to Top of Girder	Ztg	=	1.260 m ³	
Section Modulus w.r.to Bottom of Girder	Zbg	=	0.450 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 $I_{yy} = 0.0549 \text{ m}^4$

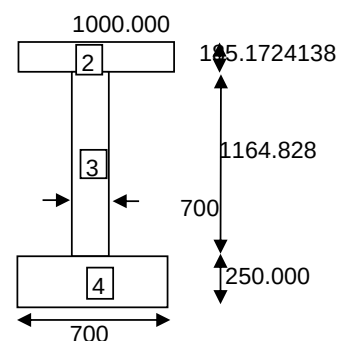
5c. Calculation of Sectional Properties about X-X Axis

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	2800	220	616000	110	6.776E+07	7.454E+09	2.485E+09	9.938E+09
Girder			1171207	988.77	1.158E+09	1.145E+12	2.640E+11	1.409E+12
			1787207		1.226E+09	1.153E+12	2.665E+11	1.419E+12

Total Area of Section A = 1.787 m²
 c.g. of Section from Top of Slab Yts = 0.686 m
 c.g. of Section from Top of Girder Ytg = 0.466 m
 c.g. of Section from Bottom of Girder Ybg = 1.134 m
 Moment of Inertia of Section Iz = 0.578 m⁴ (Ip - A Yt²)
 Section Modulus w.r.to Top of Slab Zts = 0.843 m³
 Section Modulus w.r.to Top of Girder Ztg = 1.241 m³
 Section Modulus w.r.to Bottom of Girder Zbg = 0.510 m³
 Perimeter of Deck slab excluding top flange surface, uslab = 6040.000 mm
 Total perimeter of composite girder = 11168.447 mm

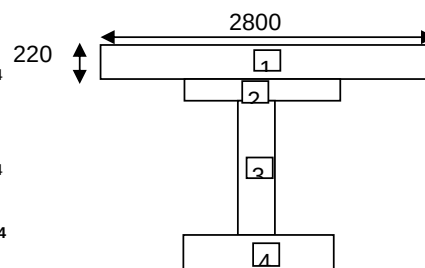
6b. Calculation of Sectional Properties about Minor Axis Y-Y

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	2800	220	616000	0.00	0.000E+00	0.000E+00	4.025E+11	4.025E+11
Girder			1171207	0.00	0.000E+00	0.000E+00	5.4947E+10	5.495E+10
			1787207		0.000E+00	0.000E+00	4.574E+11	4.574E+11

Moment of Inertia of Section Iyy = 0.457 m⁴

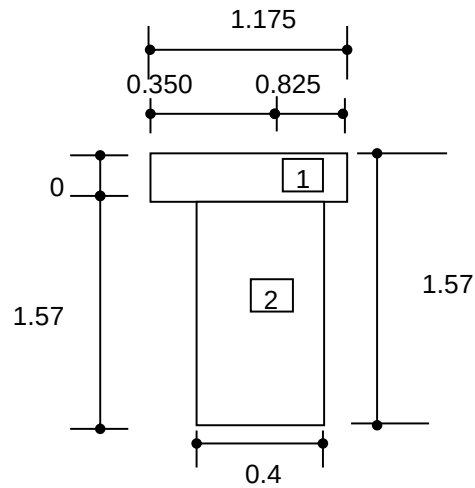
6c. Calculation of Sectional Properties about X-X Axis

Moment of Inertia of Precast Girde Ixx = 9.28E+10 mm⁴
 Moment of Inertia of Deck Slab Ixxslab = b * d³/6 = 4.97E+09 mm⁴
 Total Ixx for Composite Girder = 9.78E+10 mm⁴



SECTIONAL PROPERTIES OF END DIAPHRAGM

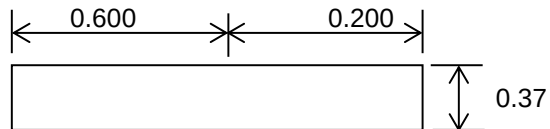
$$\begin{aligned}
 & \boxed{I_z} \\
 \text{Area (Ax)} &= \begin{array}{rcl} 1.175 & \times & 0 \\ + & 1.57 & \times & 0.4 \end{array} \\
 &= 0.628 \text{ m}^2 \\
 y_b &= 0.785 \text{ m} \\
 y_t &= 0.785 \text{ m}
 \end{aligned}$$



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)**

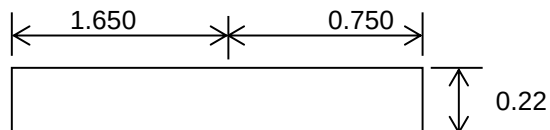
Area Ax =

$$0.800 \quad \times \quad 0.37 \quad = \quad 0.296 \text{ m}^2$$

$$I_x = \frac{0.8 \times 0.37^3}{6} = 0.006754 \text{ m}^4$$

$$I_y = \frac{0.37 \times 0.8^3}{12} = 0.015787 \text{ m}^4$$

$$I_z = \frac{0.8 \times 0.37^3}{12} = 0.003377 \text{ m}^4$$

2)

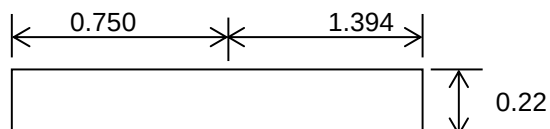
Area Ax =

$$2.400 \quad \times \quad 0.22 \quad = \quad 0.528 \text{ m}^2$$

$$I_x = \frac{2.4 \times 0.22^3}{6} = 0.004259 \text{ m}^4$$

$$I_y = \frac{0.22 \times 2.4^3}{12} = 0.253440 \text{ m}^4$$

$$I_z = \frac{2.4 \times 0.22^3}{12} = 0.002130 \text{ m}^4$$

3)

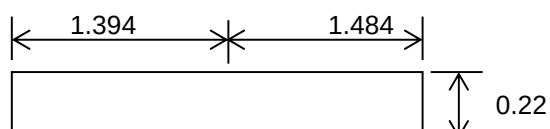
Area Ax =

$$2.144 \quad \times \quad 0.22 \quad = \quad 0.47168 \text{ m}^2$$

$$I_x = \frac{2.144 \times 0.22^3}{6} = 0.003805 \text{ m}^4$$

$$I_y = \frac{0.22 \times 2.144^3}{12} = 0.180682 \text{ m}^4$$

$$I_z = \frac{2.144 \times 0.22^3}{12} = 0.001902 \text{ m}^4$$

4)

Area Ax =

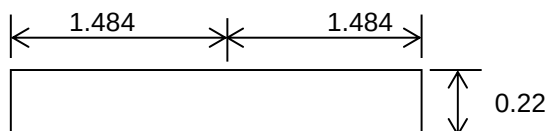
$$2.878 \quad \times \quad 0.22 \quad = \quad 0.63316 \text{ m}^2$$

$$I_x = \frac{2.878}{6} \times 0.22^3 = 0.005107 \text{ m}^4$$

$$I_y = \frac{0.22}{12} \times 2.878^3 = 0.437033 \text{ m}^4$$

$$I_z = \frac{2.878}{12} \times 0.22^3 = 0.002554 \text{ m}^4$$

5)



Area Ax =

$$2.968 \quad \times \quad 0.22 \quad = \quad 0.65296 \text{ m}^2$$

$$I_x = \frac{2.968}{6} \times 0.22^3 = 0.005267 \text{ m}^4$$

$$I_y = \frac{0.22}{12} \times 2.968^3 = 0.479328 \text{ m}^4$$

$$I_z = \frac{2.968}{12} \times 0.22^3 = 0.002634 \text{ m}^4$$

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

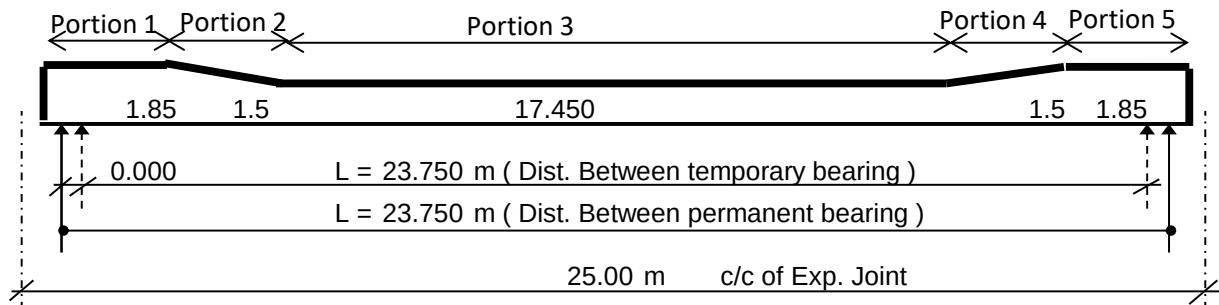
Thickness of wearing coat	=	0.065 m
Load Intensity	=	22 x 0.065
	=	1.43 kN/m ²

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	=	5.225 kN/m
UDL over Inner Girder	=	6.16 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

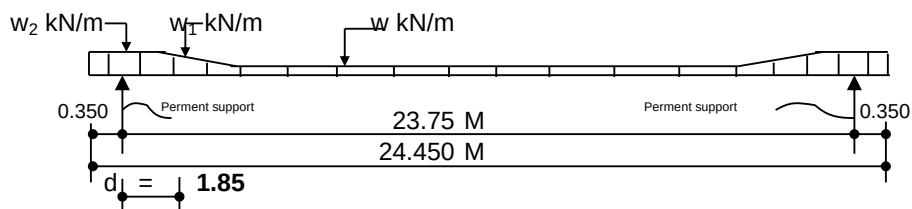
(FOR INTERMEDIATE GIRDERS)

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	=	2800 mm
Thickness of Slab	=	220 mm
Profile correction course thickness for $c\epsilon$	=	0.00 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	=	16.016 kN/m
UDL due to additional weight	=	0.910 kN/m
UDL due to Temp. Live load	=	5.6 kN/m
		<u>22.526 kN/m</u>

UDL , w = 22.526 kN/m (On entire length of girder)

Avg. , w₁ = 22.526 kN/m

UDL , w₂ = 22.526 kN/m

Supp. Reaction , R = **275.380 kN**

a) Moment at	x =	-0.200 m		
=	0.00	kNm	Corresp. Shear =	267.50 kN
b) Moment at	x =	0.000 L		
=	-1.38	kNm	Corresp. Shear =	267.50 kN
c) Moment at	x =	0.069 L		
=	409.33	kNm	Corresp. Shear =	230.33 kN
d) Moment at	x =	0.133 L		
=	729.48	kNm	Corresp. Shear =	196.54 kN
e) Moment at	x =	0.250 L		
=	1189.81	kNm	Corresp. Shear =	133.75 kN
f) Moment at	x =	0.375 L		
=	1487.61	kNm	Corresp. Shear =	66.87 kN

g) Moment at	x =	0.500 L		
=	1586.88 kNm		Corresp. Shear =	0.00 kN
h) Moment at	x =	0.625 L		
=	1487.61 kNm		Corresp. Shear =	66.87 kN
i) Moment at	x =	0.750 L		
=	1189.81 kNm		Corresp. Shear =	133.75 kN
j) Moment at	x =	0.867 L		
=	729.48 kNm		Corresp. Shear =	196.54 kN
k) Moment at	x =	0.931 L		
=	409.33 kNm		Corresp. Shear =	230.33 kN
L) Moment at	x =	1.000 L		
=	-1.38 kNm		Corresp. Shear =	267.50 kN
M) Moment at	x =	Dead end L		
=	0.00 kNm		Corresp. Shear =	267.50 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

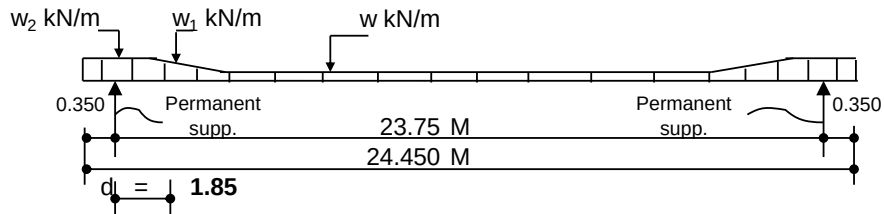
(FOR OUTER GIRDERS)

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

$$\text{UDL , w} = 18.08 \text{ kN/m}$$
$$\text{Avg. , } w_1 = 23.68 \text{ kN/m}$$
$$\text{UDL, } w_2 = 29.28 \text{ kN/m}$$

Supp. Reaction , R = **251.81 kN**

a) Moment at $x = -0.200$ m

= 0.00 kNm Corresp. Shear = 241.56 kN

b) Moment at $x = 0.000 \text{ L}$

= -1.79 kNm Corresp. Shear = 241.56 kN

c) Moment at $x = 0.069 \text{ L}$

= 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 $x = 0.250 \text{ L}$

= 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 $x = 0.500 \text{ L}$

= 1308.27 kNm Corresp. Shear = 0.00 kN

h) Moment at $x = 0.625 \text{ L}$

= 1228.60 kNm Corresp. Shear = 53.67 kN

i) Moment at $x = 0.750 \text{ L}$

= 989.61 kNm Corresp. Shear = 107.34 kN

j) Moment at $x = 0.867 \text{ L}$

=	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	=	2850	mm
Thickness of Slab	=	220	mm
Extra average slab thickness for camber	=	0.00	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	=	16.302	kN/m
UDL due to additional weight	=	0.926	kN/m
UDL due to Temp. Live load	=	<u>5.7</u>	kN/m
		22.928	kN/m

UDL , w = 22.928 kN/m (On entire length of girder)

Avg. , w_1 = 22.928 kN/m (For avg. thickness of web)

UDL , w_2 = 22.928 kN/m

Supp. Reaction , R = **280.298** kN

a) Moment at	x	=	-0.200	m	
=	0.00	kNm	Corresp. Shear =	272.27	kN
b) Moment at	x	=	0.000	L	
=	-1.40	kNm	Corresp. Shear =	272.27	kN
c) Moment at	x	=	0.069	L	
=	416.63	kNm	Corresp. Shear =	234.44	kN
d) Moment at	x	=	0.133	L	
=	742.50	kNm	Corresp. Shear =	200.05	kN
e) Moment at	x	=	0.250	L	
=	1211.06	kNm	Corresp. Shear =	136.14	kN
f) Moment at	x	=	0.375	L	
=	1514.18	kNm	Corresp. Shear =	68.07	kN
g) Moment at	x	=	0.500	L	
=	1615.22	kNm	Corresp. Shear =	0.00	kN

h) Moment at	x =	0.625 L		
=	1514.18 kNm		Corresp. Shear =	68.07 kN
i) Moment at	x =	0.750 L		
=	1211.06 kNm		Corresp. Shear =	136.14 kN
j) Moment at	x =	0.867 L		
=	742.50 kNm		Corresp. Shear =	200.05 kN
k) Moment at	x =	0.931 L		
=	416.63 kNm		Corresp. Shear =	234.44 kN
L) Moment at	x =	1.000 L		
=	-1.40 kNm		Corresp. Shear =	272.27 kN
M) Moment at	x =	Dead end		
=	0.00 kNm		Corresp. Shear =	272.27 kN

3.0 Shuttering Load

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	267.50	0.00	241.56	0.00	5.34	0.00	0.00
0.000 L	267.50	-1.38	241.56	-1.79	5.34	-0.03	0.00
0.069 L	230.33	409.33	193.25	356.93	4.60	8.18	0.00
0.133 L	196.54	729.48	157.73	620.16	3.93	14.57	0.00
0.250 L	133.75	1189.81	107.34	989.61	2.67	23.77	0.00
0.375 L	66.87	1487.61	53.67	1228.60	1.34	29.72	0.00
0.500 L	0.00	1586.88	0.00	1308.27	0.00	31.70	0.00
0.625 L	66.87	1487.61	53.67	1228.60	1.34	29.72	0.00
0.750 L	133.75	1189.81	107.34	989.61	2.67	23.77	0.00
0.867 L	196.54	729.48	157.73	620.16	3.93	14.57	0.00
0.931 L	230.33	409.33	193.25	356.93	4.60	8.18	0.00
1.000 L	267.50	-1.38	241.56	-1.79	5.34	-0.03	0.00
Dead end	267.50	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	25.26	36.56	0.05	66.4	0.2	0.01
0.000 L	25.26	78.24	0.05	63.5	107.3	0.01
0.069 L	19.52	83.44	0.01	54.3	107.3	0.00
0.133 L	10.23	117.04	0.01	54.3	190.6	0.00
0.250 L	6.41	150.27	0.01	39.4	310.6	0.00
0.375 L	2.42	172.00	0.00	22.3	387.3	0.00
0.500 L	2.42	179.18	0.00	5.1	412.8	0.00
0.625 L	2.42	172.00	0.00	12.1	387.3	0.00
0.750 L	6.41	150.27	0.01	29.3	310.8	0.00
0.867 L	10.23	117.04	0.01	45.8	191.8	0.00
0.931 L	19.52	83.44	0.01	56.8	107.2	0.00
1.000 L	25.26	78.24	0.05	63.5	107.3	0.01
Dead end	25.26	36.56	0.05	66.4	0.2	0.01

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	13.11	208.21	46.05	184.27	0.00	0.00	0.00	0.00	89.51	46.05	184.27
0.000 L	327.41	177.86	258.00	184.27	0.00	0.00	0.00	0.00	89.51	327.41	177.86
0.069 L	323.15	156.04	251.16	196.80	0.00	0.00	0.00	0.00	70.32	323.15	156.04
0.133 L	557.21	156.04	546.37	196.80	0.00	0.00	0.00	0.00	70.32	557.21	156.04
0.250 L	934.58	117.81	1147.40	217.77	0.00	0.00	0.00	0.00	43.48	1147.40	217.77
0.375 L	1192.36	71.08	1589.44	72.29	0.00	0.00	0.00	0.00	23.66	1589.44	72.29
0.500 L	1379.62	16.41	1742.19	0.58	0.00	0.00	0.00	0.00	9.36	1742.19	0.58
0.625 L	1165.34	98.24	1529.44	137.07	0.00	0.00	0.00	0.00	10.65	1529.44	137.07
0.750 L	806.91	134.44	1040.98	189.97	0.00	0.00	0.00	0.00	26.50	1040.98	189.97
0.867 L	408.61	141.17	550.21	205.70	0.00	0.00	0.00	0.00	38.63	550.21	205.70
0.931 L	206.36	135.87	255.78	197.79	0.00	0.00	0.00	0.00	72.04	255.78	197.79
1.000 L	208.02	132.78	262.34	185.82	0.00	0.00	0.00	0.00	90.37	262.34	185.82
Dead end	11.08	132.78	44.26	185.82	0.00	0.00	0.00	0.00	90.37	44.26	185.82

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	100.44	8.20	355.31	40.95	0.00	0.00	0.00	0.00	85.34	355.31	40.95
0.000 L	100.44	157.53	208.38	410.88	0.00	0.00	0.00	0.00	85.34	208.38	410.88
0.069 L	102.71	310.36	195.41	400.66	0.00	0.00	0.00	0.00	57.85	195.41	400.66
0.133 L	106.66	606.52	195.41	693.77	0.00	0.00	0.00	0.00	57.85	195.41	693.77
0.250 L	74.37	890.14	51.06	1134.51	0.00	0.00	0.00	0.00	31.79	74.37	890.14
0.375 L	24.82	1016.62	22.59	1162.22	0.00	0.00	0.00	0.00	8.81	24.82	1016.62
0.500 L	23.26	1001.90	82.86	945.08	0.00	0.00	0.00	0.00	4.85	82.86	945.08
0.625 L	71.22	846.39	103.43	697.16	0.00	0.00	0.00	0.00	9.18	103.43	697.16
0.750 L	82.72	605.98	93.51	418.61	0.00	0.00	0.00	0.00	14.96	93.51	418.61
0.867 L	158.75	406.37	80.98	196.74	0.00	0.00	0.00	0.00	20.56	158.75	406.37
0.931 L	211.83	409.54	72.67	90.31	0.00	0.00	0.00	0.00	35.55	211.83	409.54
1.000 L	262.72	157.53	67.72	93.12	0.00	0.00	0.00	0.00	45.95	262.72	157.53
Dead end	262.72	11.23	67.72	18.61	0.00	0.00	0.00	0.00	45.95	262.72	11.23

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	54.07	216.33	417.12	48.07	89.51
0.000 L	384.36	208.79	244.63	482.35	89.51
0.069 L	379.36	183.18	229.40	470.35	70.32
0.133 L	654.14	183.18	229.40	814.45	70.32
0.250 L	1346.99	255.65	87.31	1044.98	43.48
0.375 L	1865.93	84.86	29.13	1193.46	23.66
0.500 L	2045.25	0.68	97.27	1109.47	9.36
0.625 L	1795.48	160.91	121.42	818.43	10.65
0.750 L	1222.06	223.02	109.77	491.43	26.50
0.867 L	645.92	241.48	186.36	477.06	38.63
0.931 L	300.28	232.20	248.68	480.78	72.04
1.000 L	307.98	218.14	308.42	184.93	90.37
Dead end	51.96	218.14	308.42	13.18	90.37

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	272.27	0.00	241.56	0.00	5.34	0.00	0.00
0.000 L	272.27	-1.40	241.56	-1.79	5.34	-0.03	0.00
0.069 L	234.44	416.63	193.25	356.93	4.60	8.18	0.00
0.133 L	200.05	742.50	157.73	620.16	3.93	14.57	0.00
0.250 L	136.14	1211.06	107.34	989.61	2.67	23.77	0.00
0.375 L	68.07	1514.18	53.67	1228.60	1.34	29.72	0.00
0.500 L	0.00	1615.22	0.00	1308.27	0.00	31.70	0.00
0.625 L	68.07	1514.18	53.67	1228.60	1.34	29.72	0.00
0.750 L	136.14	1211.06	107.34	989.61	2.67	23.77	0.00

0.867 L	200.05	742.50	157.73	620.16	3.93	14.57	0.00
0.931 L	234.44	416.63	193.25	356.93	4.60	8.18	0.00
1.000 L	272.27	-1.40	241.56	-1.79	5.34	-0.03	0.00
Dead end	272.27	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	74.4	15.8	69.75	66.0	0.2	1.13
0.000 L	74.4	107.0	69.75	63.1	106.8	1.13
0.069 L	65.0	203.7	53.26	56.8	106.9	0.84
0.133 L	65.0	203.7	53.26	54.4	190.3	0.84
0.250 L	53.1	354.1	31.56	40.2	312.0	0.47
0.375 L	32.1	448.9	16.40	23.0	390.2	0.18
0.500 L	10.5	479.5	5.16	5.5	416.2	0.05
0.625 L	10.5	448.2	5.16	12.1	390.2	0.05
0.750 L	32.1	353.7	16.40	29.6	312.2	0.18
0.867 L	53.1	206.2	31.56	46.3	191.4	0.47
0.931 L	65.0	106.2	53.26	56.8	106.9	0.84
1.000 L	74.4	107.0	69.75	63.1	106.8	1.13
Dead end	74.4	15.8	69.75	66.0	0.2	1.13

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	-1.38	409.33	729.48	1189.81	1487.61	1586.88	1487.61	1189.81	729.48	409.33	-1.38	0.00
Shear Force (kN)	267.50	267.50	230.33	196.54	133.75	66.87	0.00	66.87	133.75	196.54	230.33	267.50	267.50

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)	36.56	78.24	83.44	117.04	150.27	172.00	179.18	172.00	150.27	117.04	83.44	78.24	36.56
Shear Force (kN)	25.26	25.26	19.52	10.23	6.41	2.42	2.42	2.42	6.41	10.23	19.52	25.26	25.26

7 DUE TO SUPERIMPOSED DEAD LOAD -SIDL2 (Applied on Composite Girder)

Bending Moment (kNm)	0.15	107.31	107.31	190.57	310.59	387.30	412.78	387.30	310.77	191.78	107.23	107.31	0.15
Shear Force (kN)	66.41	63.48	54.33	54.33	39.40	22.28	5.09	12.07	29.27	45.83	56.80	63.48	66.41

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)	54.07	384.36	379.36	654.14	1346.99	1865.93	2045.25	1795.48	1222.06	645.92	300.28	307.98	51.96
Corresp. Shear Force (kN)	216.33	208.79	183.18	183.18	255.65	84.86	0.68	160.91	223.02	241.48	232.20	218.14	218.14
Governing Shear Force (kN)	417.12	244.63	229.40	229.40	87.31	29.13	97.27	121.42	109.77	186.36	248.68	308.42	308.42
Corresp. Bending Moment (kNm)	48.07	482.35	470.35	814.45	1044.98	1193.46	1109.47	818.43	491.43	477.06	480.78	184.93	13.18

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	-1.40	416.63	742.50	1211.06	1514.18	1615.22	1514.18	1211.06	742.50	416.63	-1.40	0.00
Shear Force (kN)	272.27	272.27	234.44	200.05	136.14	68.07	0.00	68.07	136.14	200.05	234.44	272.27	272.27

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)	15.77	106.98	203.72	203.72	354.11	448.87	479.54	448.22	353.67	206.20	106.25	106.98	15.77
Shear Force (kN)	74.39	74.39	64.98	64.98	53.05	32.07	10.55	10.55	32.07	53.06	64.98	74.39	74.39

7 DUE TO SUPERIMPOSED DEAD LOAD -SIDL2 (Applied on Composite Girder)

Bending Moment (kNm)	0.25	106.80	106.85	190.28	312.01	390.21	416.24	390.21	312.16	191.39	106.85	106.80	0.25
Shear Force (kN)	66.00	63.15	56.83	54.41	40.21	22.98	5.46	12.08	29.60	46.32	56.83	63.15	66.00

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

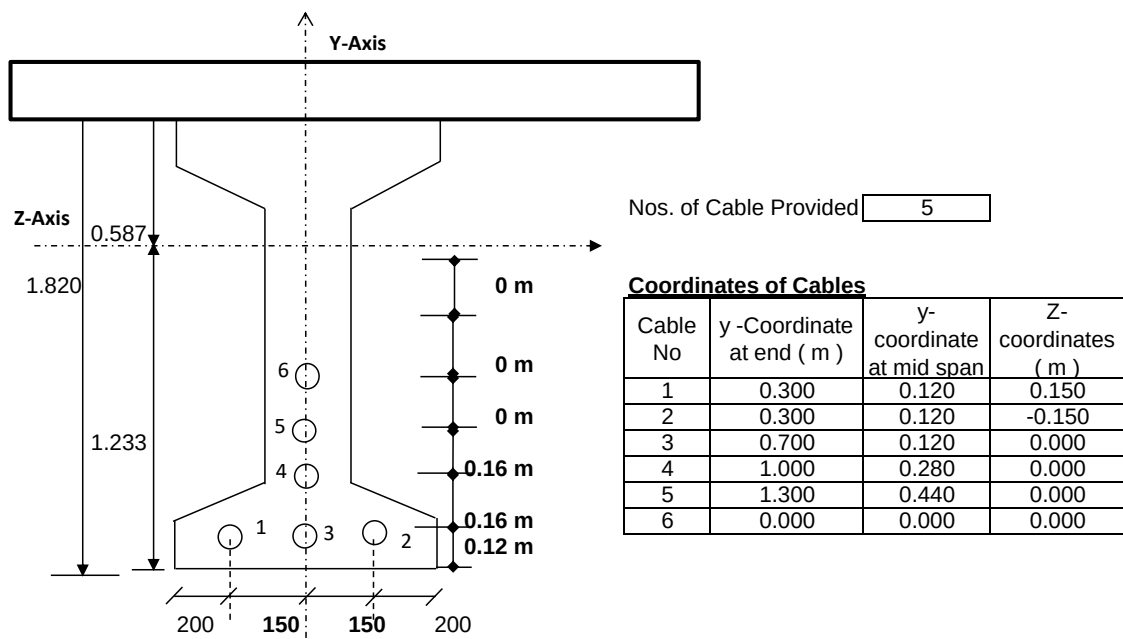


Fig. Section At Mid-Span showing cables in Elevation

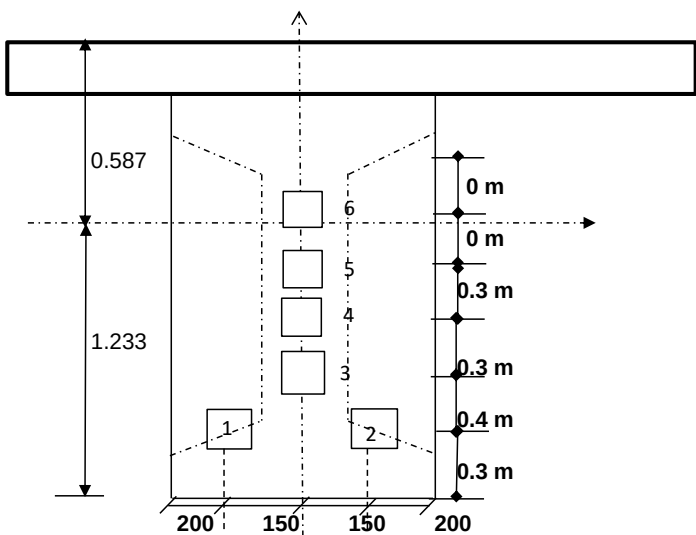


Fig. Section At Jacking End showing cables in Elevation

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

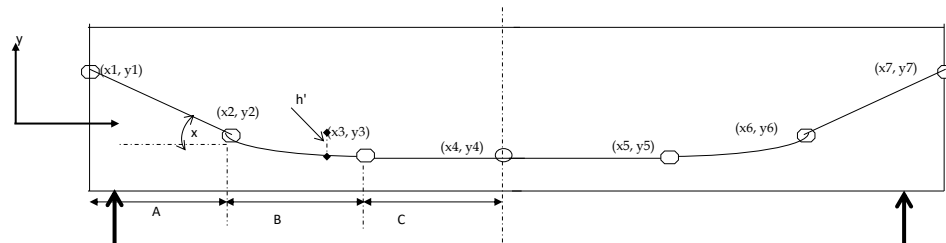


FIG. Showing Cable Profile in Elevation

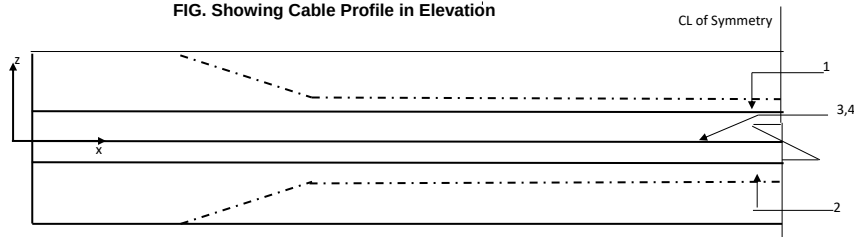


FIG. Showing Cable Profile in Plan

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:

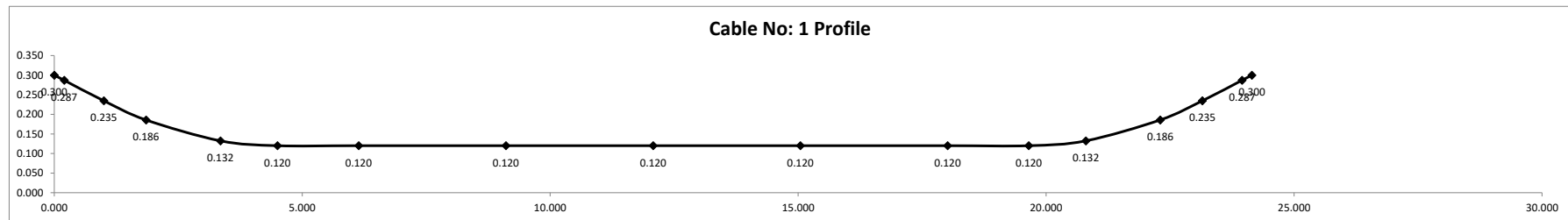
Cable 1

$$\text{Friction loss} = P_o \times \exp(-(\mu x + k \theta)) = P_o \times \text{Friction coeff}$$

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 6	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 7	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 7		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
										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	0.462	0.162
0.000 L	1389.232	90.931	0.287	0.831	0.544	0.462	0.175
0.069 L	1399.724	69.368	0.186	0.829	0.643	0.418	0.232
0.133 L	1412.987	30.389	0.132	0.819	0.686	0.367	0.235
0.250 L	1427.309	0.000	0.120	0.819	0.699	0.367	0.247
0.375 L	1436.243	0.000	0.120	0.819	0.699	0.367	0.247
0.500 L	1445.124	0.000	0.120	0.819	0.699	0.367	0.247
0.625 L	1453.952	0.000	0.120	0.819	0.699	0.367	0.247
0.750 L	1462.729	0.000	0.120	0.819	0.699	0.367	0.247

Half shaded area $s \times E_s \times A / 2$
area of force diagram above null point
equat Half shaded area - area of force diagram above null point to zero

=
=
=

635.13 kN
635.14 kN

0.00

0.867 L	1459.833	31.396	0.132	0.819	0.686	0.367	0.235
0.931 L	1447.101	71.716	0.186	0.829	0.643	0.418	0.232
1.000 L	1437.135	94.066	0.287	0.831	0.544	0.462	0.175
Dead end	1436.559	94.029	0.300	0.831	0.531	0.462	0.162

Prestress after Friction and Slip Loss: Cable 2

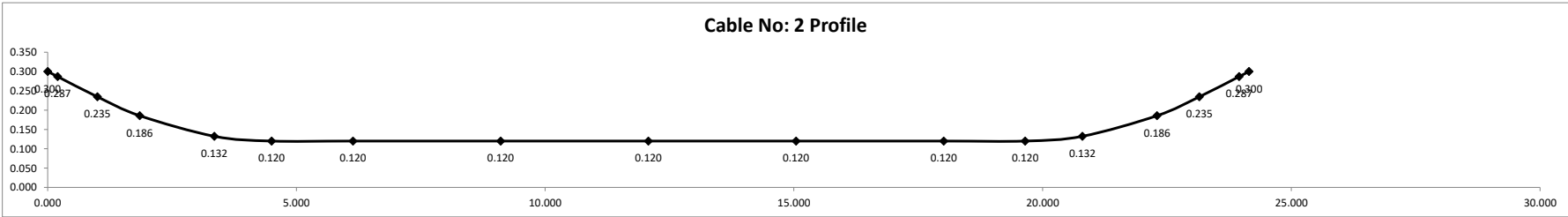
Cable Profile		Sections	x (m)	y(m)	z (m)	θ_v (deg)	θ_H (deg)	θ_S (deg)	cum θ (deg)	L INC (m)	X PRO (m)	Friction coeff.	Force after Friction loss , p (kN)	Force at null point (kN)	Nodal force above null point (kN)	Area of Force Dia. above null (kNm)	Force after Slip loss , p` (kN)	p` hor (kN)
Inclined	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
	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		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
	Node 3		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		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
	Node 4	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
			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
	Node 5	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
Parabolic			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 6		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
	Node 7	Dead end																

635.137

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	0.462	0.162
0.000 L	1389.232	90.932	0.287	0.831	0.544	0.462	0.175
0.069 L	1399.724	69.368	0.186	0.829	0.643	0.418	0.232
0.133 L	1412.987	30.388	0.132	0.819	0.686	0.367	0.235
0.250 L	1427.309	0.000	0.120	0.819	0.699	0.367	0.247
0.375 L	1436.243	0.000	0.120	0.819	0.699	0.367	0.247
0.500 L	1445.124	0.000	0.120	0.819	0.699	0.367	0.247
0.625 L	1453.952	0.000	0.120	0.819	0.699	0.367	0.247
0.750 L	1462.728	0.000	0.120	0.819	0.699	0.367	0.247
0.867 L	1459.833	31.396	0.132	0.819	0.686	0.367	0.235
0.931 L	1447.101	71.716	0.186	0.829	0.643	0.418	0.232
1.000 L	1437.135	94.067	0.287	0.831	0.544	0.462	0.175
Dead end	1436.559	94.029	0.300	0.831	0.531	0.462	0.162

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

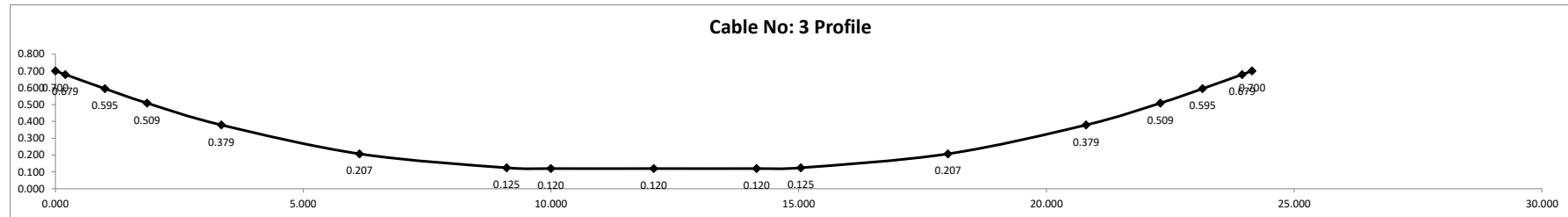
Calculation of Slip loss:

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
Parabolic			1.000	0.595	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
Straight	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
Straight	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	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
Parabolic		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	1323.965
		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
Inclined	Node 6		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
	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**

**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	0.462	-0.238
0.000 L	1249.484	131.764	0.679	0.831	0.152	0.462	-0.217
0.069 L	1257.684	120.102	0.509	0.829	0.320	0.418	-0.091
0.133 L	1267.886	98.793	0.379	0.819	0.440	0.367	-0.012
0.250 L	1285.763	58.191	0.207	0.819	0.611	0.367	0.160
0.375 L	1303.181	13.647	0.125	0.819	0.694	0.367	0.243
0.500 L	1313.686	0.000	0.120	0.819	0.699	0.367	0.247
0.625 L	1323.97	13.86	0.12	0.819	0.694	0.367	0.243
0.750 L	1319.13	59.70	0.21	0.819	0.611	0.367	0.160
0.867 L	1301.94	101.45	0.38	0.819	0.440	0.367	-0.012
0.931 L	1292.23	123.40	0.51	0.829	0.320	0.418	-0.091
1.000 L	1284.52	135.46	0.68	0.831	0.152	0.462	-0.217
Dead end	1284.00	135.40	0.70	0.831	0.131	0.462	-0.238

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

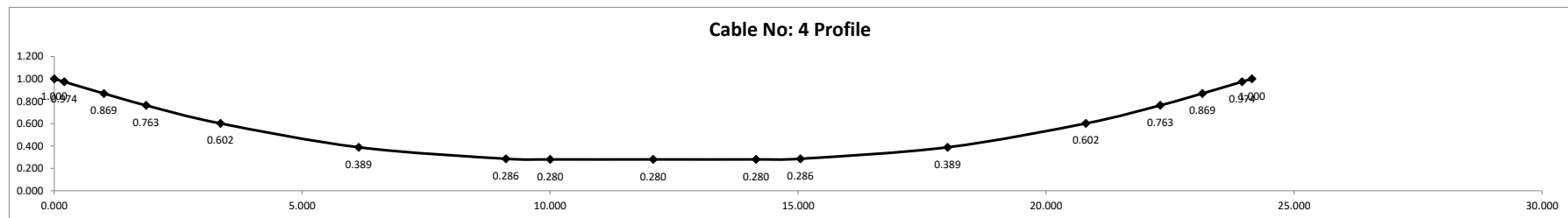
Prestress after Friction and Slip Loss:**Cable 4**

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

**Summary of Forces after Friction and Slip Losses:****Cable 4**

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	1237.432	161.991	1.000	0.831	-0.169	0.462	-0.538
0.000 L	1237.993	162.065	0.974	0.831	-0.143	0.462	-0.511
0.069 L	1247.377	147.871	0.763	0.829	0.066	0.418	-0.345
0.133 L	1259.534	121.831	0.602	0.819	0.217	0.367	-0.234
0.250 L	1280.545	71.943	0.389	0.819	0.430	0.367	-0.021
0.375 L	1300.515	16.907	0.286	0.819	0.533	0.367	0.082
0.500 L	1311.594	0.000	0.280	0.819	0.539	0.367	0.087
0.625 L	1322.36	17.19	0.29	0.819	0.533	0.367	0.082
0.750 L	1310.33	73.62	0.39	0.819	0.430	0.367	-0.021
0.867 L	1290.14	124.79	0.60	0.819	0.217	0.367	-0.234
0.931 L	1278.58	151.57	0.76	0.829	0.066	0.418	-0.345
1.000 L	1269.77	166.22	0.97	0.831	-0.143	0.462	-0.511
Dead end	1269.26	166.16	1.00	0.831	-0.169	0.462	-0.538

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:

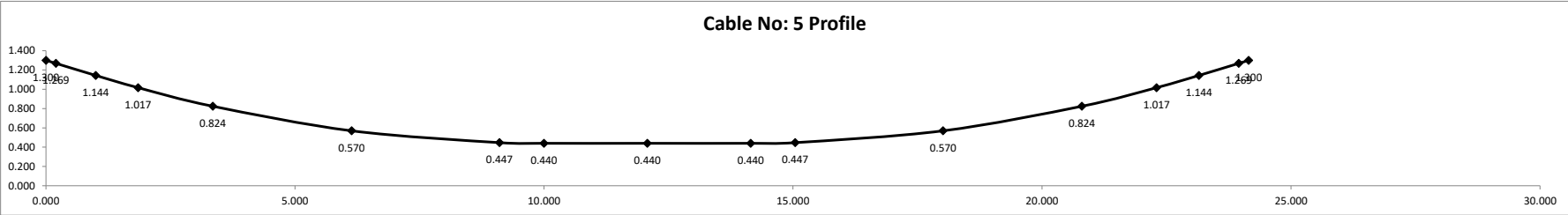
Assuming Slip travel upto 15.6749 m from anchorage end
Prestress force at null point = 1326.20 kN (After friction loss by Interpolation)
Assume Slip = 6 mm
Half shaded area $s \times E_s \times A / 2$ = 577.40 kN
area of force diagram above null point = 577.40 kN
equat Half shaded area - area of force diagram above null point to zero = 0.00

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	0.000	8.887	0.000	8.887	0.000	0.810	1.012	0.998	1401.572	-	79.260	1243.052	1228.129
Parabolic			0.069 L	1.850	1.017	0.000	8.059	0.000	8.059	0.828	0.859	1.872	0.994	1395.733	-	73.421	1248.891	1236.556
			0.133 L	3.350	0.824	0.000	6.590	0.000	6.590	2.297	1.512	3.384	0.987	1385.467	-	63.155	1259.157	1250.836
			0.250 L	6.138	0.570	0.000	3.839	0.000	3.839	5.048	2.799	6.183	0.973	1366.531	-	44.219	1278.093	1275.225
			0.375 L	9.106	0.447	0.000	0.890	0.000	0.890	7.997	2.971	9.154	0.959	1346.598	-	24.286	1298.026	1297.869
			Node 3	10.000	0.440	0.000	0.000	0.000	0.000	8.887	0.894	10.048	0.955	1340.650	-	18.338	1303.974	1303.974
			Node 4	12.075	0.440	0.000	0.000	0.000	0.000	8.887	2.075	12.123	0.951	1335.098	-	12.786	1309.526	1309.526
Straight	Node 5	0.500 L	14.150	0.440	0.000	0.000	0.000	0.000	8.887	2.075	14.198	0.947	1329.569	-	7.257	20.794	1315.055	1315.055
			0.625 L	15.044	0.447	0.000	0.890	0.000	0.890	9.777	0.894	15.092	0.943	1323.696	1322.312	1.384	1320.928	1320.769
			0.750 L	18.013	0.570	0.000	3.839	0.000	3.839	12.726	2.971	18.063	0.929	1304.388	-	0.000	1304.388	1301.460
Parabolic			0.867 L	20.800	0.824	0.000	6.590	0.000	6.590	15.477	2.799	20.862	0.916	1286.560	-	0.000	1286.560	1278.058
			0.931 L	22.300	1.017	0.000	8.059	0.000	8.059	16.946	1.512	22.375	0.909	1277.097	-	0.000	1277.097	1264.484
			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	1271.776	1256.509
Inclined	Node 7	1.000 L	23.950	1.269	0.000	8.887	0.000	8.887	17.774	0.810	24.044	0.904	1269.718	-	0.000	0.000	1269.718	1254.475
			Dead end	24.150	1.300	0.000	8.887	0.000	8.887	17.774	0.202	24.246	0.904	1269.204	-	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	0.462	-0.838
0.000 L	1225.884	191.684	1.269	0.831	-0.437	0.462	-0.806
0.069 L	1236.556	175.091	1.017	0.829	-0.188	0.418	-0.599

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	0.367	-0.457
0.250 L	1275.225	85.575	0.570	0.819	0.249	0.367	-0.202
0.375 L	1297.869	20.153	0.447	0.819	0.372	0.367	-0.080
0.500 L	1309.526	0.000	0.440	0.819	0.379	0.367	-0.073
0.625 L	1320.769	20.509	0.447	0.819	0.372	0.367	-0.080
0.750 L	1301.460	87.336	0.570	0.819	0.249	0.367	-0.202
0.867 L	1278.058	147.661	0.824	0.819	-0.005	0.367	-0.457
0.931 L	1264.484	179.046	1.017	0.829	-0.188	0.418	-0.599
1.000 L	1254.475	196.154	1.269	0.831	-0.437	0.462	-0.806
Dead end	1253.968	196.075	1.300	0.831	-0.469	0.462	-0.838

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	0.462	0.162
Cable 2	2	1388.614	90.891	0.300	0.831	0.531	0.462	0.162
Cable 3	1	1248.922	131.705	0.700	0.831	0.131	0.462	-0.238
Cable 4	2	1237.432	161.991	1.000	0.831	-0.169	0.462	-0.538
Cable 5	1	1225.323	191.596	1.300	0.831	-0.469	0.462	-0.838

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 = -0.535 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' = -0.053 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	0.462	0.175
Cable 2	2	1389.232	90.932	0.287	0.831	0.544	0.462	0.175
Cable 3	1	1249.484	131.764	0.679	0.831	0.152	0.462	-0.217
Cable 4	2	1237.993	162.065	0.974	0.831	-0.143	0.462	-0.511
Cable 5	1	1225.884	191.684	1.269	0.831	-0.437	0.462	-0.806

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 = -0.509 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	=	-0.036	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	0.418	0.232
Cable 2	2	1399.724	69.368	0.186	0.829	0.643	0.418	0.232
Cable 3	1	1257.684	120.102	0.509	0.829	0.320	0.418	-0.091
Cable 4	2	1247.377	147.871	0.763	0.829	0.066	0.418	-0.345
Cable 5	1	1236.556	175.091	1.017	0.829	-0.188	0.418	-0.599

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 = -0.343 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 = 0.054 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	0.367	0.235
Cable 2	2	1412.987	30.388	0.132	0.819	0.686	0.367	0.235
Cable 3	1	1267.886	98.793	0.379	0.819	0.440	0.367	-0.012
Cable 4	2	1259.534	121.831	0.602	0.819	0.217	0.367	-0.234
Cable 5	1	1250.836	144.516	0.824	0.819	-0.005	0.367	-0.457

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 = -0.233 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 = 0.090 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	0.367	0.247
Cable 2	2	1427.309	0.000	0.120	0.819	0.699	0.367	0.247
Cable 3	1	1285.763	58.191	0.207	0.819	0.611	0.367	0.160
Cable 4	2	1280.545	71.943	0.389	0.819	0.430	0.367	-0.021
Cable 5	1	1275.225	85.575	0.570	0.819	0.249	0.367	-0.202

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 = -0.020 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 = 0.164 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	0.367	0.247
Cable 2	2	1436.243	0.000	0.120	0.819	0.699	0.367	0.247
Cable 3	1	1303.181	13.647	0.125	0.819	0.694	0.367	0.243
Cable 4	2	1300.515	16.907	0.286	0.819	0.533	0.367	0.082
Cable 5	1	1297.869	20.153	0.447	0.819	0.372	0.367	-0.080

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' = 0.082 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'	=	0.196	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	0.367	0.247
Cable 2	2.000	1445.124	0.000	0.120	0.819	0.699	0.367	0.247
Cable 3	1.000	1313.686	0.000	0.120	0.819	0.699	0.367	0.247
Cable 4	2.000	1311.594	0.000	0.280	0.819	0.539	0.367	0.087
Cable 5	1.000	1309.526	0.000	0.440	0.819	0.379	0.367	-0.073

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'	=	0.088	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'	=	0.197	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	0.367	0.247
Cable 2	2.000	1453.952	0.000	0.120	0.819	0.699	0.367	0.247
Cable 3	1.000	1323.965	13.865	0.125	0.819	0.694	0.367	0.243
Cable 4	2.000	1322.362	17.191	0.286	0.819	0.533	0.367	0.082
Cable 5	1.000	1320.769	20.509	0.447	0.819	0.372	0.367	-0.080

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'	=	0.082	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' = 0.196 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	0.367	0.247
Cable 2	2.000	1462.728	0.000	0.120	0.819	0.699	0.367	0.247
Cable 3	1.000	1319.133	59.701	0.207	0.819	0.611	0.367	0.160
Cable 4	2.000	1310.328	73.617	0.389	0.819	0.430	0.367	-0.021
Cable 5	1.000	1301.460	87.336	0.570	0.819	0.249	0.367	-0.202

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' = -0.020 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' = 0.164 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	0.367	0.235
Cable 2	2.000	1459.833	31.396	0.132	0.819	0.686	0.367	0.235
Cable 3	1.000	1301.939	101.446	0.379	0.819	0.440	0.367	-0.012
Cable 4	2.000	1290.144	124.792	0.602	0.819	0.217	0.367	-0.234
Cable 5	1.000	1278.058	147.661	0.824	0.819	-0.005	0.367	-0.457

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' = -0.232 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' = 0.091 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	0.418	0.232
Cable 2	2.000	1447.101	71.716	0.186	0.829	0.643	0.418	0.232
Cable 3	1.000	1292.225	123.401	0.509	0.829	0.320	0.418	-0.091
Cable 4	2.000	1278.579	151.570	0.763	0.829	0.066	0.418	-0.345
Cable 5	1.000	1264.484	179.046	1.017	0.829	-0.188	0.418	-0.599

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' = -0.343 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' = 0.055 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	0.462	0.175
Cable 2	2.000	1437.135	94.067	0.287	0.831	0.544	0.462	0.175
Cable 3	1.000	1284.515	135.458	0.679	0.831	0.152	0.462	-0.217
Cable 4	2.000	1269.768	166.224	0.974	0.831	-0.143	0.462	-0.511
Cable 5	1.000	1254.475	196.154	1.269	0.831	-0.437	0.462	-0.806

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'	=	-0.508	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'	=	-0.035	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	0.462	0.162
Cable 2	2.000	1436.559	94.029	0.300	0.831	0.531	0.462	0.162
Cable 3	1.000	1283.999	135.404	0.700	0.831	0.131	0.462	-0.238
Cable 4	2.000	1269.256	166.157	1.000	0.831	-0.169	0.462	-0.538
Cable 5	1.000	1253.968	196.075	1.300	0.831	-0.469	0.462	-0.838

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'	=	-0.534	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'	=	-0.052	m

ANALYSIS OF OUTER LONG. GIRDER FOR TEMPERATURE

Material Properties:

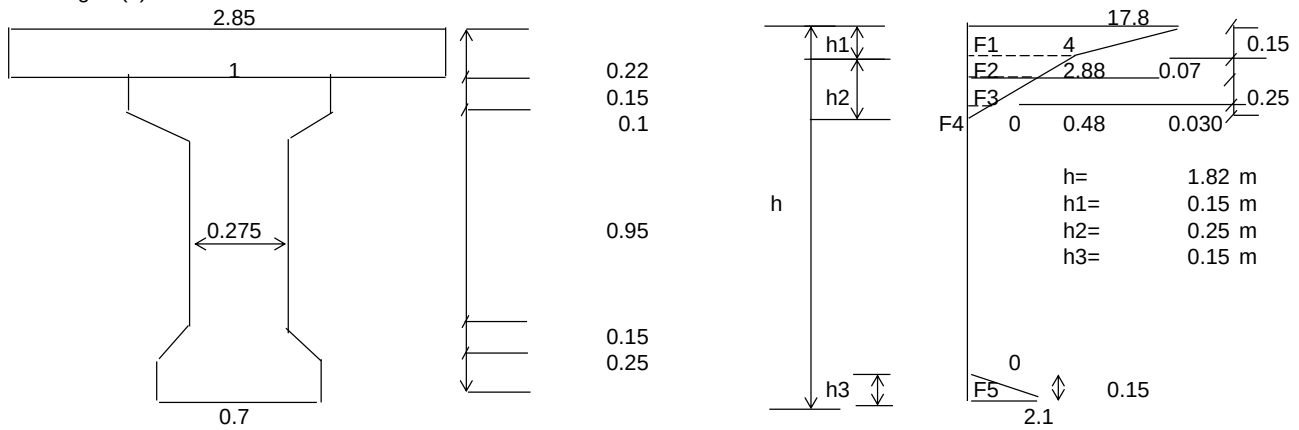
Concrete Grade for the superstructure	=	M 45
E_c for the concrete	=	3.40E+04 N/mm ²
α	=	Coefficient of Thermal expansion as per Clause 215.4 , IRC:6-2017
	=	1.20E-05 / °C

Sectional Properties of Composite Section

A	=	1.350 m ²	Z_{ts}	=	0.836 m ³
y_{ts}	=	0.587 m	Z_{tg}	=	1.3367 m ³
y_{tg}	=	0.367 m	Z_{bg}	=	0.3984 m ³
y_{bg}	=	1.233 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	2.85	0.15	0.428	1901.18	0.059	0.528	1004.184
F2	3.440	2.85	0.07	0.200	280.00	0.183	0.404	113.195
F3	1.680	1.00	0.15	0.150	102.82	0.277	0.310	31.896
F4	0.240	0.64	0.03	0.019	1.87	0.380	0.207	0.388
F5	1.050	0.70	0.15	0.105	44.98	1.770	-1.183	-53.197
SUM				0.901	2330.851			1096.466

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

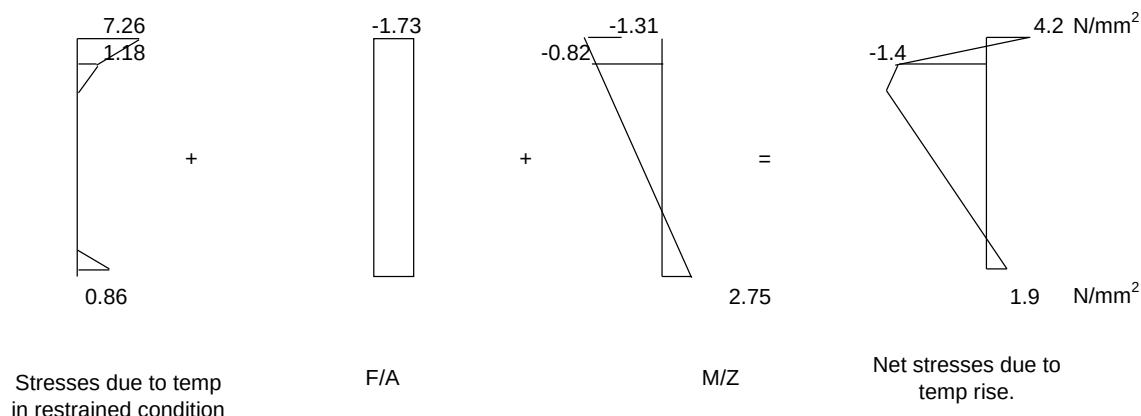
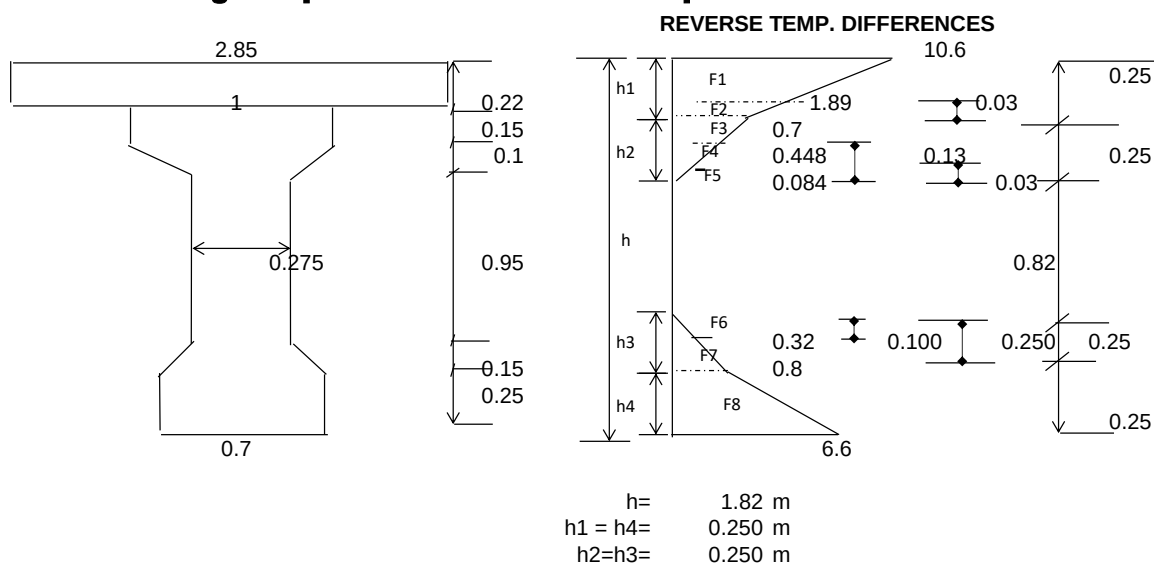
Force/Area	$(-F_t/A)$	=	-1.73 N/mm ²

Bending Stress Due to Direct Tension

At the top of deck slab	$(-M_t / Z_{ts})$	=	-1.31 N/mm ²
At the top of girder	$(-M_t / Z_{tg})$	=	-0.82 N/mm ²
At the bottom of girder	(M_t / Z_{bg})	=	2.75 N/mm ²

Final Temperature Stress:

At the top of deck slab	=	4.22 N/mm ²	σ_{ts}	(-Ft/At)	(- Mt / Zts)
At the Bottom of Deck Slab	=	-1.37 N/mm ²	σ_{tg}	(-Ft/At)	(- Mt / Ztg)
At the bottom of girder	=	1.88 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	2.85	0.22	0.627	1597.32	0.084	0.503	803.361
F2	1.294	2.85	0.03	0.086	45.14	0.233	0.355	16.009
F3	0.574	0.15	0.12	0.018	4.22	0.306	0.282	1.188
F4	0.266	0.64	0.22	0.140	15.22	0.455	0.132	2.016
F5	0.042	0.28	0.03	0.008	0.14	0.48	0.107	0.015
F6	0.160	0.28	0.100	0.028	1.80	1.387	-0.799	-1.435
F7	0.560	0.49	0.250	0.122	27.85	1.484	-0.897	-24.976
F8	3.700	0.70	0.25	0.175	264.18	1.728	-1.140	-301.243
SUM				1.203	1955.854			494.936

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

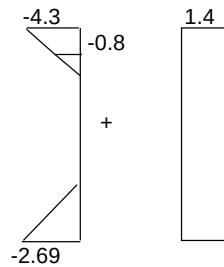
Force/Area	(Ft/A)	=	1.45 N/mm ²

Bending Stress Due to Direct Tension

At the top of deck slab	(Mt / Zts)	=	0.59 N/mm ²
At the top of girder	(Mt / Ztg)	=	0.37 N/mm ²
At the bottom of girder	(- Mt / Zbg)	=	-1.24 N/mm ²

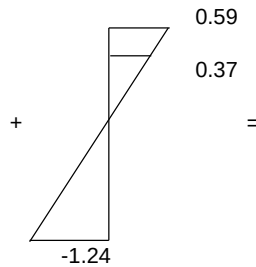
Final Temperature Stress:

At the top of deck slab	=	-2.28 N/mm ²	σ_{ts}	+ (Ft/At) + (Mt / Zts)
At the Bottom of Deck Slab	=	1.05 N/mm ²	σ_{tg}	+ (Ft/At) + (Mt / Ztg)
At the bottom of girder	=	-2.49 N/mm ²	σ_{bg}	+ (Ft/At) - (Mt / Zbg)

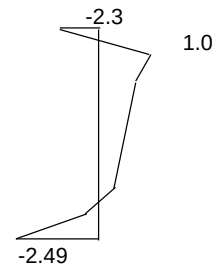


Stress due to temp in restrained condition

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	=	5.95E-05	
Creep Coefficient ϕ (t^∞, t_0)	=	1.83	
Creep reduction factor (refer eq. 7.3 of IRC:112-2011) α	=	$\frac{1 - e^{-\phi}}{\phi}$	= 0.459
Effective area of Deck Slab A_{cf}	=	627000	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	=	1.350	m ²
Distance of c.g. of section from top of slab	=	Yts	=	0.587	m
Distance of c.g. of section from top of girder	=	Ytg	=	0.367	m
Distance of c.g. of section from bottom of girder	=	Ybg	=	1.233	m
Sectional modulus at top of slab	=	Zts	=	0.836	m ³
sectional modulus at top of girder	=	Ztg	=	1.337	m ³
sectional modulus at bottom of girder	=	Zbg	=	0.398	m ³

Stresses in Outer Girder Due to Differential Shrinkage Effect:

Step (1) Stress due to Tensile Force in deck slab

$$\sigma_{\text{Tension}} = 3.400\text{E}+04 \times -5.95\text{E}-05 = -2.022\text{E}+00 \text{ N/mm}^2$$

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

$$F = 581.75 \text{ kN}$$

Restrained Moment @ c.g. of composite section

$$M = 581.75 \times (0.587 - 0.11) = 277.71 \text{ kNm}$$

$$\sigma_{\text{compression}} = \frac{F}{Ac} \pm \frac{M}{Z}$$

$\sigma_{\text{compression}}$	at top of deck slab	=	0.431	+	0.332	=	0.763	N/mm ²
$\sigma_{\text{compression}}$	at bottom of deck slab	=	0.431	+	0.208	=	0.639	N/mm ²
$\sigma_{\text{compression}}$	at top of girder	=	0.431	+	0.208	=	0.639	N/mm ²
$\sigma_{\text{compression}}$	at bottom of girder	=	0.431	+	-0.697	=	-0.266	N/mm ²

Step (3) : Net stresses at various levels of section

Step (1)	+	Step (2)

Total stress at top of deck slab	=	-2.022	+	0.763	=	-1.259	N/mm ²
Total stress at bottom of deck slab	=	-2.022	+	0.639	=	-1.383	N/mm ²
Total stress at top of girder	=	0.000	+	0.639	=	0.639	N/mm ²
Total stress at bottom of girder	=	0.000	+	-0.266	=	-0.266	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
1st Moment of area above and about N.A. of Comp. Section (Ay)p	m ³	0.096	0.096	0.078	0.057	0.057	0.057	0.057	0.057	0.057	0.057	0.078	0.096	0.096

Properties of Composite Section: Precast Girder + Deck Slab

Area of Composite Section, A	m ²	1.798	1.798	1.563	1.350	1.350	1.350	1.350	1.350	1.350	1.350	1.563	1.798	1.798
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	0.582	0.582	0.534	0.491	0.491	0.491	0.491	0.491	0.491	0.491	0.534	0.582	0.582
C.G. of section from top,Yts	m	0.682	0.682	0.638	0.587	0.587	0.587	0.587	0.587	0.587	0.587	0.638	0.682	0.682
C.G. of section from top,Ytg	m	0.462	0.462	0.418	0.367	0.367	0.367	0.367	0.367	0.367	0.367	0.418	0.462	0.462
C.G. of section from bottom, Ybg	m	1.138	1.138	1.182	1.233	1.233	1.233	1.233	1.233	1.233	1.233	1.182	1.138	1.138
Sectional modulus at top, Zts	m ³	0.853	0.853	0.837	0.836	0.836	0.836	0.836	0.836	0.836	0.836	0.837	0.853	0.853
Sectional modulus at top, Ztg	m ³	1.259	1.259	1.279	1.337	1.337	1.337	1.337	1.337	1.337	1.337	1.279	1.259	1.259
Sectional modulus at bottom, Zbg	m ³	0.512	0.512	0.452	0.398	0.398	0.398	0.398	0.398	0.398	0.398	0.452	0.512	0.512
Perimeter of Composite girder section, u	m	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140
Notional size(mm) of the cross section , ho	mm	585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
1st Moment of area above and about N.A. of Comp. Section (Ay)c	m ³	0.455	0.455	0.409	0.363	0.363	0.363	0.363	0.363	0.363	0.363	0.409	0.455	0.455

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	-0.535	-0.509	-0.343	-0.233	-0.020	0.082	0.088	0.082	-0.020	-0.232	-0.343	-0.508	-0.534

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	-0.053	-0.036	0.054	0.090	0.164	0.196	0.197	0.196	0.164	0.091	0.055	-0.035	-0.052

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

$$\epsilon_{cs}(t) = \beta_s(t, t_s) k_L \epsilon_{sh}$$

Eq. 6.14

where

$$\beta_s(t, t_s) = \frac{t - t_s}{t + t_s}$$

$$\epsilon_{sh} = \epsilon_{sh} \cdot \epsilon_{sh}$$

RH	=	80 %
fck	=	45

$$\epsilon_{cd}(t) = \beta_{ds}(t) \epsilon_{cs}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \sqrt{h_0^3}}$$

Eq. 6.15

$$\epsilon_{cs}(t) = \beta_{as}(t) \epsilon_{cw}$$

Eq. 6.12

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

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		585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0422	0.0422	0.0516	0.0635	0.0635	0.0635	0.0635	0.0635	0.0635	0.0635	0.0516	0.0422	0.0422
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.700	0.715	0.715	0.715	0.715	0.715	0.715	0.715	0.700	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		7.449	7.449	9.101	11.436	11.436	11.436	11.436	11.436	11.436	11.436	9.101	7.449	7.449
	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}	$\mu\epsilon$	49.891	49.891	51.543	53.878	53.878	53.878	53.878	53.878	53.878	53.878	51.543	49.891	49.891
At age t (35 days)	notional size (mm) of the cross section, ho		585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0534	0.0534	0.0651	0.0798	0.0798	0.0798	0.0798	0.0798	0.0798	0.0798	0.0651	0.0534	0.0534
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.700	0.715	0.715	0.715	0.715	0.715	0.715	0.715	0.700	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		9.423	9.423	11.484	14.383	14.383	14.383	14.383	14.383	14.383	14.383	11.484	9.423	9.423
	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}	$\mu\epsilon$	54.514	54.514	56.575	59.474	59.474	59.474	59.474	59.474	59.474	59.474	56.575	54.514	54.514
At age t (49 days)	notional size (mm) of the cross section, ho		585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0750	0.0750	0.0910	0.1109	0.1109	0.1109	0.1109	0.1109	0.1109	0.1109	0.0910	0.0750	0.0750
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.700	0.715	0.715	0.715	0.715	0.715	0.715	0.715	0.700	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		13.236	13.236	16.051	19.978	19.978	19.978	19.978	19.978	19.978	19.978	16.051	13.236	13.236
	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}	$\mu\epsilon$	62.208	62.208	65.022	68.949	68.949	68.949	68.949	68.949	68.949	68.949	65.022	62.208	62.208
At age t (63 days)	notional size (mm) of the cross section, ho		585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
	Coeff. For drying shrinkage strain $\beta_{ds}(t, t_s)$		0.0957	0.0957	0.1155	0.1399	0.1399	0.1399	0.1399	0.1399	0.1399	0.1399	0.1155	0.0957	0.0957
	Coeff. Depends on notional size for shrinkage, kh		0.700	0.700	0.700	0.715	0.715	0.715	0.715	0.715	0.715	0.715	0.700	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		16.879	16.879	20.372	25.208	25.208	25.208	25.208	25.208	25.208	25.208	20.372	16.879	16.879
	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}	$\mu\epsilon$	68.590	68.590	72.083	76.919	76.919	76.919	76.919	76.919	76.919	76.919	72.083	68.590	68.590
At age t Infinity	notional size (mm) of the cross section, ho		585.735	585.735	509.148	439.780	439.780	439.780	439.780	439.780	439.780	439.780	509.148	585.735	585.735
	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	0.700	0.715	0.715	0.715	0.715	0.715	0.715	0.715	0.700	0.700	0.700
	Drying shrinkage strain, ϵ_{cd}		176.400	176.400	176.400	180.194	180.194	180.194	180.194	180.194	180.194	180.194	176.400	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	241.400	245.194	245.194	245.194	245.194	245.194	245.194	245.194	241.400	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	$\mu\epsilon$	21.366	21.366	21.997	22.367	22.367	22.367	22.367	22.367	22.367	22.367	21.997	21.366	21.366
	from 28 days to 35 days	$\mu\epsilon$	4.623	4.623	5.031	5.596	5.596	5.596	5.596	5.596	5.596	5.596	5.031	4.623	4.623
	from 35 days to 49 days	$\mu\epsilon$	7.693	7.693	8.447	9.475	9.475	9.475	9.475	9.475	9.475	9.475	8.447	7.693	7.693
	from 49 days to 63 days	$\mu\epsilon$	6.383	6.383	7.061	7.969	7.969	7.969	7.969	7.969	7.969	7.969	7.061	6.383	6.383
	from 63 days to Infinity	$\mu\epsilon$	172.810	172.810	169.317	168.275	168.275	168.275	168.275	168.275	168.275	168.275	169.317	172.810	172.810

CREEP MODEL (micro strain)

(Refer Cl: 6.4.2.7 of IRC:112-2011)

$$\epsilon(t, t_s) = \beta(t, t_s) \cdot \epsilon(t_s) \quad \text{where } \beta(t, t_s) = \left[\frac{t - t_s}{t_s} \right]^{0.3}, \quad \epsilon(t_s) = \phi_{sh} \cdot \beta(fcm) \cdot \beta(t_s)$$

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

$$\phi_{RH} = 1 + \frac{1 - RH/100}{0.1 \sqrt[3]{h_o}} \text{ for } f_{cm} \leq 45 \text{ MPa} \quad \text{Eq. A2-16} \quad \beta(t_o) = \frac{1}{0.1 + t_o^{0.20}}$$

$$\phi_{RH} = \left[1 + \frac{1 - RH/100}{0.1 \sqrt[3]{h_o}} \alpha_1 \right] \alpha_2 \text{ for } f_{cm} \leq 45 \text{ MPa} \quad \text{Eq. A2-17}$$

$$\beta_H = 1.5[1 + (0.012RH)^{0.8}] h_o + 250 \leq 1500 \text{ for } f_{cm} \leq 35 \quad \text{Eq. A2-22}$$

$$\beta_H = 1.5[1 + (0.012RH)^{0.8}] h_o + 250 \alpha_3 \leq 1500 \alpha_3 \text{ for } f_{cm} \geq 35 \quad \text{Eq. A2-23}$$

$$t_o = t_{o,T} \left(\frac{9}{2 + t_{o,T}^{1.2}} + 1 \right)^{0.5} \geq 0.5 \quad \text{Eq. A2-25}$$

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	585.74	585.74	509.15	439.78	439.78	439.78	439.78	439.78	439.78	439.78	509.15	585.74	585.74
(7 days)	φ _{RH} , refer eq. A2-16 & A2-17	1.157	1.157	1.172	1.188	1.188	1.188	1.188	1.188	1.188	1.188	1.172	1.157	1.157
	β _H , Refer Eq. A2-22 & A2-23, where fcm ≤ 45	1337.824	1337.824	1337.824	1199.021	1199.021	1199.021	1199.021	1199.021	1199.021	1199.021	1337.824	1337.824	1337.824
to	Temperature adjusted conc. Age t _{o,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)	1.782	1.782	1.804	1.830	1.830	1.830	1.830	1.830	1.830	1.830	1.804	1.782	1.782
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.286	0.286	0.286	0.296	0.296	0.296	0.296	0.296	0.296	0.296	0.286	0.286	0.286
(28 days)	Creep Coefficient, φ(t,t _o)	0.510	0.510	0.516	0.541	0.541	0.541	0.541	0.541	0.541	0.541	0.516	0.510	0.510
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.312	0.312	0.312	0.322	0.322	0.322	0.322	0.322	0.322	0.322	0.312	0.312	0.312
(35 days)	Creep Coefficient, φ(t,t _o)	0.555	0.555	0.562	0.589	0.589	0.589	0.589	0.589	0.589	0.589	0.562	0.555	0.555
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.351	0.351	0.351	0.362	0.362	0.362	0.362	0.362	0.362	0.362	0.351	0.351	0.351
(49 days)	Creep Coefficient, φ(t,t _o)	0.625	0.625	0.633	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.633	0.625	0.625
Creep coeff. For age between														
(7 days)														
to	β(t,t _o)	0.381	0.381	0.381	0.393	0.393	0.393	0.393	0.393	0.393	0.393	0.381	0.381	0.381
(63 days)	Creep Coefficient, φ(t,t _o)	0.679	0.679	0.688	0.720	0.720	0.720	0.720	0.720	0.720	0.720	0.688	0.679	0.679
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)	1.781	1.781	1.804	1.829	1.829	1.829	1.829	1.829	1.829	1.829	1.804	1.781	1.781

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

$$f_{cm}(t) = \beta_{cc}(t) \times f_{cm} \quad \text{where} \quad f_{cm} = f_{ck} + 10 \quad \& \quad \beta_{cc}(t) = \exp \left\{ s \left[1 - \left(\frac{28}{t+1} \right)^{1/2} \right] \right\} \quad \text{for Ordinary portland cement} \quad S = 0.25$$

Concrete Tensile Strength Development with Time is given by Eq. 6.7 of IRC:112-2011

$$f_{ctm}(t) = \beta_{cc}(t)^a \times f_{ctm} \quad \text{where} \quad a = 1 \text{ for } t < 28 \text{ days} \& \frac{2}{3} \text{ for } t \geq 28 \text{ days}, \quad \beta_{cc}(t) = \exp \left\{ s \left[1 - \left(\frac{28}{t+1} \right)^{1/2} \right] \right\} \quad \& \quad f_{ctm} = 0.259 (f_{ck})^{2/3} \text{ for } f_{ck} \leq 60 \text{ mpa}, \quad f_{ctm} = 2.27 \ln[1 + (f_{cm}/12.5)] \text{ for } f_{ck} > 60 \text{ mpa}$$

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 clausue 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)	%	0.783	0.783	0.803	0.831	0.817	0.804	0.797	0.791	0.798	0.811	0.783	0.763	0.763
Stress loss due to Shrinkage = ecs x Ep x Ac	kN	19.205	19.205	19.841	20.739	20.739	20.739	20.739	20.739	20.739	20.739	19.841	19.205	19.205

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	%	0.739	0.739	0.742	0.770	0.757	0.745	0.739	0.733	0.740	0.751	0.724	0.720	0.720
	kN	18.121	18.121	18.351	19.219	19.219	19.219	19.219	19.219	19.219	19.219	18.351	18.121	18.121

TOTAL PRESTRESS LOSS (From 7 days to 28 days)

Loss due to (SH + CR + RLEX)	%	3.411	3.414	3.495	3.626	3.729	3.843	3.938	4.031	3.927	3.776	3.649	3.571	3.568
	kN	83.638	83.744	86.397	90.516	94.662	99.093	102.429	105.715	102.033	96.563	92.470	89.858	89.757

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	83.638	83.744	86.397	90.516	94.662	99.093	102.429	105.715	102.033	96.563	92.470	89.858	89.757
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	-13.876	-11.706	5.872	19.788	40.797	52.841	55.207	56.358	44.022	21.164	6.340	-12.499	-14.830
P_{hor} / A	N/mm ²	0.071	0.072	0.092	0.125	0.131	0.137	0.142	0.146	0.141	0.134	0.099	0.077	0.077
M_p/Z_{tg}	N/mm ²	-0.040	-0.034	0.019	0.070	0.144	0.186	0.195	0.199	0.155	0.075	0.020	-0.036	-0.043
M_p/Z_{bg}	N/mm ²	-0.044	-0.037	0.020	0.073	0.151	0.195	0.204	0.208	0.163	0.078	0.022	-0.039	-0.047
$P_{hor} / A - M_p/Z_{tg}$	N/mm ²	0.112	0.106	0.073	0.055	-0.013	-0.049	-0.053	-0.052	-0.014	0.059	0.078	0.113	0.120
$P_{hor} / A + M_p/Z_{bg}$	N/mm ²	0.028	0.035	0.113	0.198	0.282	0.332	0.345	0.354	0.304	0.212	0.121	0.037	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	3.166	2.982	3.174	3.659	3.155	3.099	3.320	3.082	3.144	3.688	3.217	3.050	3.238
At Bottom of Precast Girder,fbg	N/mm ²	0.785	0.986	1.875	2.979	3.614	3.774	3.596	3.897	3.774	3.105	1.954	1.015	0.809
Stress in Concrete at c.g. of Stage I Cable	N/mm ²	2.269	2.197	2.493	3.234	3.503	3.654	3.548	3.752	3.622	3.323	2.554	2.249	2.322
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	2.978 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	1.553	1.608	2.171	3.097	3.556	3.702	3.567	3.810	3.694	3.206	2.240	1.647	1.590
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	2.726 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	3.423	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	2.522	1.447	1.512	-0.094	-0.643	-0.474	-0.709	-0.186	1.463	1.423	2.560	2.955
At Bottom of Precast Girder,fbg	N/mm ²	8.200	8.621	12.702	16.800	18.731	19.514	19.472	19.851	19.260	17.359	13.133	8.908	8.477

Stress at c.g of Stage I Cables	N/mm ²	4.901	4.919	7.349	11.065	14.169	15.918	15.984	16.181	14.553	11.402	7.568	5.058	5.038

Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg}) = 10.316 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	6.494	6.720	10.144	14.152	16.341	17.351	17.353	17.643	16.792	14.615	10.479	6.935	6.701

Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg}) = 12.440 N/mm²

Increase in Average Stress at cg. of Stage-I Cables = 7.337 N/mm²

Increase in Average Stress at cg. of Stage-II Cables = 9.714 N/mm²

ADDN. LOSS OF PRESTRESS DUE TO ELASTIC SHORTENING FROM STAGE-I CABLES

m = Eps / Ec,eff at 28 days = 5.735 Ec,eff = Ecm / (1+ ϕ) = 34000 N/mm²
 Eps = 195000 N/mm²
 A_{cable} = 1974 mm²
 Elastic Shortening Loss due to Stage-I Cables = 1.0 x σ_{cg_avg} x m x A_{cable of stage I} = 83.070 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
Mp = P_{hor} x 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 / Ec,eff at 28 days = 5.735 A_{cable of stage II} = 3158.4 mm²
 σ_{cg_avg} = 9.714 N/mm²
 Elastic Shortening Loss due to Stage-II Cables = 0.5 x σ_{cg_avg} x m x A_{cable of stage II} = 87.980 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
Mp = P_{hor} x 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	2.801	2.427	1.414	1.507	-0.014	-0.523	-0.351	-0.589	-0.105	1.458	1.390	2.466	2.850
At Bottom of Precast Girder, fbg	N/mm ²	8.010	8.419	12.358	16.320	18.162	18.904	18.859	19.240	18.691	16.879	12.789	8.706	8.286

Stress at c.g of Stage I Cables	N/mm ²	4.764	4.783	7.154	10.764	13.757	15.438	15.500	15.701	14.141	11.101	7.372	4.922	4.901

Average stress in concrete at c.g. of Stage- I

=

10.023

N/mm²Cable($\sigma_{cq \text{ avg}}$)

Stress at c.g of Stage II Cables	N/mm ²	6.331	6.552	9.871	13.755	15.855	16.819	16.819	17.111	16.306	14.218	10.205	6.766	6.538

Average stress in concrete at c.g. of Stage- II

=

12.088

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/mm²Area of cables in stage -I Prestress, $A_{cable} = 1974.000$ mm²

Differential shrinkage strain = ecs	μC	4.623	4.623	5.031	5.596	5.596	5.596	5.596	5.596	5.596	5.596	5.031	4.623	4.623
shrinkage loss between (28 days to 35days)	%	0.073	0.073	0.078	0.086	0.085	0.084	0.083	0.082	0.083	0.084	0.076	0.071	0.071
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	1.780	1.780	1.937	2.154	2.154	2.154	2.154	2.154	2.154	2.154	1.937	1.780	1.780

Shrinkage Loss for Stage II Cables

=

 $E_p = 1.95E+05$ N/mm²Area of cables in stage -II Prestress, $A_{cable} = 3158.400$ mm²

Differential shrinkage strain = ecs	μC	4.623	4.623	5.031	5.596	5.596	5.596	5.596	5.596	5.596	5.596	5.031	4.623	4.623
shrinkage loss between (28 days to 35days)	%	0.073	0.072	0.078	0.086	0.085	0.084	0.084	0.083	0.083	0.084	0.076	0.070	0.070
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	2.847	2.847	3.099	3.446	3.446	3.446	3.446	3.446	3.446	3.446	3.099	2.847	2.847

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 $\sigma_{cp,Avg,-I}$ = 10.023 N/mm² Modulus of elasticity of prestressing steel, E_p = 195000 N/mm²
 Average stress at c.g. of stage-II Cables $\sigma_{cp,Avg,-II}$ = 12.088 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 = $\phi(t_2, t_0)$

$\phi(t_2, t_0)$		0.555	0.555	0.562	0.589	0.589	0.589	0.589	0.589	0.589	0.589	0.562	0.555	0.555
$\phi(t_1, t_0)$		0.510	0.510	0.516	0.541	0.541	0.541	0.541	0.541	0.541	0.541	0.516	0.510	0.510
Creep loss for Stage I Cables	%	0.209	0.209	0.210	0.217	0.213	0.210	0.208	0.207	0.209	0.212	0.205	0.203	0.204
	kN	5.119	5.119	5.184	5.418	5.418	5.418	5.418	5.418	5.418	5.418	5.184	5.119	5.119

$\phi(t_2, t_0)$		0.555	0.555	0.562	0.589	0.589	0.589	0.589	0.589	0.589	0.589	0.562	0.555	0.555
$\phi(t_1, t_0)$		0.510	0.510	0.516	0.541	0.541	0.541	0.541	0.541	0.541	0.541	0.516	0.510	0.510
Creep loss for Stage II Cables	%	0.252	0.251	0.253	0.262	0.258	0.256	0.254	0.252	0.252	0.254	0.245	0.244	0.244
	kN	9.879	9.879	10.004	10.455	10.455	10.455	10.455	10.455	10.455	10.455	10.004	9.879	9.879

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 28 days to 35 days**

Total Prestress Loss for Stage-I Cables	%	0.352	0.352	0.361	0.379	0.379	0.379	0.381	0.382	0.381	0.379	0.361	0.352	0.352
	kN	8.626	8.630	8.919	9.458	9.612	9.777	9.902	10.024	9.887	9.683	9.145	8.858	8.855
Total Prestress Loss for Stage-II Cables	%	1.940	1.943	1.999	2.079	2.156	2.215	2.261	2.333	2.347	2.281	2.195	2.141	2.138
	kN	76.177	76.321	79.153	83.122	87.264	90.475	93.017	96.641	97.350	94.029	89.657	86.838	86.698

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	8.626	8.630	8.919	9.458	9.612	9.777	9.902	10.024	9.887	9.683	9.145	8.858	8.855
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	-1.431	-1.206	0.606	2.068	4.143	5.214	5.337	5.344	4.266	2.122	0.627	-1.232	-1.463

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	76.177	76.321	79.153	83.122	87.264	90.475	93.017	96.641	97.350	94.029	89.657	86.838	86.698
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	24.031	25.384	36.825	45.027	53.718	58.542	60.348	62.517	59.935	51.016	41.807	28.990	27.461

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	84.803	84.951	88.072	92.580	96.877	100.252	102.919	106.666	107.237	103.712	98.803	95.697	95.552
M_p (Stage I & Stage II Cables)	kNm	22.600	24.177	37.431	47.094	57.861	63.756	65.685	67.861	64.201	53.138	42.434	27.758	25.998
P_{hor} / A	N/mm ²	0.072	0.073	0.094	0.128	0.134	0.139	0.142	0.148	0.148	0.143	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	0.070	0.120	0.166	0.204	0.225	0.231	0.239	0.226	0.187	0.136	0.081	0.076
M_p / Z_{bg}	N/mm ²	0.071	0.076	0.129	0.174	0.214	0.235	0.243	0.251	0.237	0.196	0.147	0.087	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	-0.089	-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	0.223	0.302	0.348	0.374	0.385	0.398	0.385	0.340	0.252	0.169	0.163

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	2.425	1.467	1.596	0.140	-0.332	-0.151	-0.393	0.056	1.554	1.447	2.465	2.844
At Bottom of Precast Girder,fbg	N/mm ²	7.866	8.270	12.107	15.964	17.727	18.420	18.357	18.733	18.218	16.486	12.508	8.537	8.123

Stress at c.g of Stage I Cables	N/mm ²	4.705	4.723	7.047	10.575	13.465	15.074	15.121	15.319	13.822	10.890	7.252	4.854	4.835

Average stress in concrete at c.g. of **Stage- I Cable**($\sigma_{ca, avg}$) = 9.822 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	6.231	6.449	9.688	13.476	15.494	16.408	16.391	16.679	15.913	13.908	10.002	6.650	6.425

Average stress in concrete at c.g. of **Stage- II Cable**($\sigma_{ca, avg}$) = 11.824 N/mm²**Stress in Girder Due to Green Weight of Deck slab****(At 35 days)**

Moment due to Green Weight + Temp. LL	kNm	0.00	-1.40	416.63	742.50	1211.06	1514.18	1615.22	1514.18	1211.06	742.50	416.63	-1.40	0.00
M_{DL1} / Z_{tg}	N/mm ²	0.000	-0.004	1.339	2.616	4.267	5.335	5.691	5.335	4.267	2.616	1.339	-0.004	0.000
$-M_{DL1} / Z_{bg}$	N/mm ²	0.000	0.004	-1.439	-2.741	-4.471	-5.590	-5.963	-5.590	-4.471	-2.741	-1.439	0.004	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	2.421	2.806	4.212	4.406	5.002	5.540	4.942	4.323	4.169	2.786	2.461	2.844
At Bottom of Precast Girder,fbg	N/mm ²	7.866	8.275	10.668	13.223	13.256	12.830	12.394	13.142	13.747	13.745	11.069	8.542	8.123

Stress at c.g of Stage I Cables	N/mm ²	4.705	4.722	6.929	9.843	11.111	11.433	11.195	11.678	11.466	10.157	7.133	4.854	4.835

Average stress in concrete at c.g. of **Stage- I****Cable**($\sigma_{cq \text{ avg}}$) = 8.466 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	6.231	6.450	8.881	11.663	12.132	11.990	11.666	12.262	12.551	12.092	9.192	6.651	6.425

Average stress in concrete at c.g. of **Stage- II****Cable**($\sigma_{cq \text{ avg}}$) = 9.860 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	7.693	7.693	8.447	9.475	9.475	9.475	9.475	9.475	9.475	9.475	8.447	7.693	7.693
shrinkage loss between (35 days to 49days)	%	0.121	0.121	0.132	0.146	0.144	0.141	0.140	0.139	0.140	0.143	0.128	0.118	0.118
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	2.961	2.961	3.252	3.647	3.647	3.647	3.647	3.647	3.647	3.647	3.252	2.961	2.961

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	7.693	7.693	8.447	9.475	9.475	9.475	9.475	9.475	9.475	9.475	8.447	7.693	7.693
shrinkage loss between (35 days to 49days)	%	0.121	0.121	0.131	0.146	0.144	0.143	0.142	0.141	0.141	0.142	0.127	0.117	0.117
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	4.738	4.738	5.203	5.836	5.836	5.836	5.836	5.836	5.836	5.836	5.203	4.738	4.738

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}	=	$\{\phi(t2, to) - \phi(t1, to)\} \times E_p / E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of stage-I Cables	$\sigma_{cp,Avg-I}$	=	8.466 N/mm ²	Modulus of elasticity of prestressing steel, E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg-II}$	=	9.860 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		=	$\phi(t1, to)$	Area of prestressing strand of Stage -II Prestress, A_{ps}		=	3158.40 mm ²
Creep coeff for age between from 7 to 35		=	$\phi(t2, to)$				

$\phi(t2, to)$		0.625	0.625	0.633	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.633	0.625	0.625
$\phi(t1, to)$		0.555	0.555	0.562	0.589	0.589	0.589	0.589	0.589	0.589	0.589	0.562	0.555	0.555
Creep loss for Stage I Cables	%	0.273	0.273	0.274	0.284	0.279	0.275	0.272	0.270	0.273	0.277	0.268	0.266	0.266
	kN	6.699	6.699	6.784	7.082	7.082	7.082	7.082	7.082	7.082	7.082	6.784	6.699	6.699

$\phi(t2, to)$		0.625	0.625	0.633	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.633	0.625	0.625
$\phi(t1, to)$		0.555	0.555	0.562	0.589	0.589	0.589	0.589	0.589	0.589	0.589	0.562	0.555	0.555
Creep loss for Stage II Cables	%	0.318	0.318	0.319	0.330	0.326	0.323	0.321	0.319	0.318	0.320	0.309	0.308	0.308
	kN	12.483	12.483	12.641	13.197	13.197	13.197	13.197	13.197	13.197	13.197	12.641	12.483	12.483

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 35 days to 49 days**

Total Prestress Loss for Stage-I Cables	%	0.532	0.532	0.548	0.577	0.580	0.583	0.587	0.592	0.587	0.581	0.552	0.536	0.536
	kN	13.033	13.041	13.546	14.411	14.713	15.035	15.278	15.518	15.249	14.851	13.988	13.486	13.479
Total Prestress Loss for Stage-II Cables	%	0.719	0.719	0.740	0.777	0.785	0.791	0.796	0.806	0.808	0.799	0.762	0.742	0.741
	kN	28.232	28.257	29.305	31.044	31.763	32.320	32.761	33.390	33.513	32.937	31.128	30.082	30.058

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	13.033	13.041	13.546	14.411	14.713	15.035	15.278	15.518	15.249	14.851	13.988	13.486	13.479
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	-2.162	-1.823	0.921	3.150	6.341	8.018	8.235	8.273	6.579	3.255	0.959	-1.876	-2.227

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	28.232	28.257	29.305	31.044	31.763	32.320	32.761	33.390	33.513	32.937	31.128	30.082	30.058
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	8.906	9.398	13.634	16.816	19.553	20.913	21.255	21.600	20.633	17.870	14.515	10.043	9.520

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	41.265	41.298	42.851	45.455	46.475	47.355	48.039	48.908	48.762	47.788	45.116	43.568	43.536
M_p (Stage I & Stage II Cables)	kNm	6.744	7.575	14.555	19.967	25.893	28.930	29.490	29.872	27.212	21.125	15.474	8.167	7.294
P_{hor} / A	N/mm ²	0.035	0.035	0.046	0.063	0.064	0.065	0.066	0.068	0.067	0.066	0.048	0.037	0.037
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	0.020	0.022	0.047	0.070	0.091	0.102	0.104	0.105	0.096	0.074	0.050	0.024	0.021
M_p / Z_{bg}	N/mm ²	0.021	0.024	0.050	0.074	0.096	0.107	0.109	0.110	0.100	0.078	0.053	0.026	0.023
$P_{hor} / A - M_p / Z_{ts}$	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	0.016	0.013	-0.001	-0.007	-0.027	-0.036	-0.037	-0.038	-0.028	-0.008	-0.002	0.013	0.016
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.056	0.059	0.096	0.137	0.160	0.172	0.175	0.178	0.168	0.144	0.102	0.063	0.060

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	2.779	2.408	2.807	4.220	4.433	5.039	5.578	4.979	4.351	4.178	2.787	2.447	2.828
At Bottom of Precast Girder,fbg	N/mm ²	7.810	8.216	10.571	13.086	13.096	12.657	12.218	12.964	13.579	13.601	10.967	8.479	8.063

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.674	4.691	6.879	9.761	10.997	11.298	11.057	11.539	11.345	10.070	7.080	4.821	4.803
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	8.386 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	6.188	6.406	8.807	11.551	11.996	11.840	11.513	12.107	12.408	11.974	9.114	6.604	6.379
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	9.760 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	-0.011	-0.018	-0.022	-0.024	-0.022	-0.018	-0.011	-0.006	0.000	0.000
$-M/Z_{bg}$	N/mm ²	0.000	0.000	0.018	0.037	0.060	0.075	0.080	0.075	0.060	0.037	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	2.779	2.408	2.800	4.209	4.416	5.017	5.554	4.957	4.334	4.167	2.781	2.447	2.828
At Bottom of Precast Girder,fbg	N/mm ²	7.810	8.216	10.590	13.123	13.156	12.732	12.298	13.039	13.639	13.637	10.985	8.479	8.063

Stress at c.g of Stage I Cables	N/mm ²	4.674	4.691	6.885	9.779	11.037	11.355	11.119	11.596	11.386	10.088	7.087	4.821	4.803
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	8.409 N/mm ²												
Stress at c.g of Stage II Cables	N/mm ²	6.188	6.406	8.819	11.579	12.046	11.904	11.582	12.171	12.458	12.002	9.126	6.604	6.379
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	9.790 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	$\mu\epsilon$	6.383	6.383	7.061	7.969	7.969	7.969	7.969	7.969	7.969	7.969	7.061	6.383	6.383
shrinkage loss between (49 days to 63days)	%	0.100	0.100	0.110	0.123	0.121	0.119	0.118	0.117	0.118	0.120	0.107	0.098	0.098
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	2.457	2.457	2.718	3.068	3.068	3.068	3.068	3.068	3.068	3.068	2.718	2.457	2.457

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	$\mu\epsilon$	6.383	6.383	7.061	7.969	7.969	7.969	7.969	7.969	7.969	7.969	7.061	6.383	6.383
shrinkage loss between (49 days to 63days)	%	0.100	0.100	0.110	0.123	0.121	0.120	0.119	0.118	0.118	0.119	0.106	0.097	0.097
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	3.931	3.931	4.349	4.908	4.908	4.908	4.908	4.908	4.908	4.908	4.349	3.931	3.931

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	=	100.00	%		
Relaxation Loss between 49days & 63days	=	100.00	-	100.00	
	=	0.00	% 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	%	0.000	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}	=	$\{\phi(t_2, t_0) - \phi(t_1, t_0)\} \times E_p / E_{c,eff} \times \sigma_{cp} \times A_p$	Modulus of elasticity of concrete	$E_{c,eff} = E_{cm}$	=	34000 N/mm ²
Average stress at c.g. of stage-I Cables	$\sigma_{cp,Avg-I}$	=	8.409 N/mm ²	Modulus of elasticity of prestressing steel , E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg-II}$	=	9.790 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		=	$\phi(t_1, t_0)$	Area of prestressing strand of Stage -II Prestress, A_{ps}		=	3158.40 mm ²
Creep coeff for age between from 7 to 49		=	$\phi(t_2, t_0)$				

$\phi(t_2, t_0)$		0.679	0.679	0.688	0.720	0.720	0.720	0.720	0.720	0.720	0.720	0.688	0.679	0.679
$\phi(t_1, t_0)$		0.625	0.625	0.633	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.633	0.625	0.625
Creep loss for Stage I Cables	%	0.211	0.211	0.212	0.219	0.215	0.212	0.210	0.208	0.210	0.213	0.206	0.205	0.205
	kN	5.168	5.168	5.233	5.455	5.455	5.455	5.455	5.455	5.455	5.455	5.233	5.168	5.168

$\phi(t_2, t_0)$		0.679	0.679	0.688	0.720	0.720	0.720	0.720	0.720	0.720	0.720	0.688	0.679	0.679
$\phi(t_1, t_0)$		0.625	0.625	0.633	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.633	0.625	0.625
Creep loss for Stage II Cables	%	0.245	0.245	0.246	0.254	0.251	0.249	0.247	0.245	0.245	0.246	0.239	0.237	0.237
	kN	9.625	9.625	9.747	10.160	10.160	10.160	10.160	10.160	10.160	10.160	9.747	9.625	9.625

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 49 days to 63 days**

Total Prestress Loss for Stage-I Cables	%	0.311	0.311	0.322	0.341	0.336	0.331	0.328	0.325	0.328	0.333	0.314	0.303	0.303
	kN	7.624	7.624	7.951	8.522	8.522	8.522	8.522	8.522	8.522	8.522	7.951	7.624	7.624
Total Prestress Loss for Stage-II Cables	%	0.487	0.487	0.502	0.529	0.531	0.533	0.535	0.539	0.539	0.536	0.509	0.494	0.494
	kN	19.111	19.124	19.878	21.128	21.491	21.772	21.995	22.312	22.374	22.083	20.798	20.044	20.032

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	7.624	7.624	7.951	8.522	8.522	8.522	8.522	8.522	8.522	8.522	7.951	7.624	7.624
Eccentricity of Stage I cable from N.A. (ec)	m	-0.535	-0.509	-0.343	-0.233	-0.020	0.082	0.088	0.082	-0.020	-0.232	-0.343	-0.508	-0.534
$M_p = P_{hor} \times ec$	kNm	-4.077	-3.878	-2.729	-1.984	-0.174	0.698	0.747	0.697	-0.170	-1.979	-2.724	-3.873	-4.072

Stage II Cables

Prestress After SH+CR+RELEX Loss(P_{hor})	kN	19.111	19.124	19.878	21.128	21.491	21.772	21.995	22.312	22.374	22.083	20.798	20.044	20.032
Eccentricity of Stage II cable from N.A. (ec)	m	-0.053	-0.036	0.054	0.090	0.164	0.196	0.197	0.196	0.164	0.091	0.055	-0.035	-0.052
$M_p = P_{hor} \times ec$	kNm	-1.020	-0.694	1.074	1.909	3.529	4.261	4.342	4.363	3.676	2.014	1.146	-0.702	-1.044

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	26.735	26.748	27.829	29.651	30.013	30.295	30.517	30.834	30.896	30.606	28.748	27.669	27.657
M_p (Stage I & Stage II Cables)	kNm	-5.098	-4.572	-1.655	-0.075	3.355	4.958	5.089	5.059	3.506	0.035	-1.579	-4.575	-5.116
P_{dor} / A	N/mm ²	0.015	0.015	0.018	0.022	0.022	0.022	0.023	0.023	0.023	0.023	0.018	0.015	0.015
M_p / Z_{ts}	N/mm ²	-0.006	-0.005	-0.002	0.000	0.004	0.006	0.006	0.006	0.004	0.000	-0.002	-0.005	-0.006
M_p / Z_{tg}	N/mm ²	-0.004	-0.004	-0.001	0.000	0.003	0.004	0.004	0.004	0.003	0.000	-0.001	-0.004	-0.004
M_p / Z_{bg}	N/mm ²	-0.010	-0.009	-0.004	0.000	0.008	0.012	0.013	0.013	0.009	0.000	-0.003	-0.009	-0.010
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.021	0.020	0.020	0.022	0.018	0.017	0.017	0.017	0.019	0.023	0.020	0.021	0.021

$P_{hor} / A - M_p / Z_{tg}$	N/mm ²	0.019	0.019	0.019	0.022	0.020	0.019	0.019	0.019	0.020	0.023	0.020	0.019	0.019
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.005	0.006	0.014	0.022	0.031	0.035	0.035	0.036	0.032	0.023	0.015	0.006	0.005

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	2.760	2.389	2.781	4.187	4.396	4.998	5.535	4.938	4.313	4.144	2.761	2.428	2.809
At Bottom of Precast Girder,fbg	N/mm ²	7.805	8.210	10.575	13.101	13.125	12.697	12.263	13.003	13.607	13.614	10.971	8.472	8.057

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.661	4.677	6.869	9.758	11.009	11.323	11.086	11.564	11.357	10.066	7.070	4.807	4.788
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	8.387	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	6.179	6.396	8.804	11.558	12.017	11.871	11.548	12.137	12.427	11.980	9.110	6.593	6.370
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	9.768	N/mm ²											

STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT**(At 63 days)**

Moment due to SIDL1	kNm	15.77	106.98	203.72	203.72	354.11	448.87	479.54	448.22	353.67	206.20	106.25	106.98	15.77
M/Z_{ts}	N/mm ²	0.018	0.125	0.243	0.244	0.424	0.537	0.574	0.536	0.423	0.247	0.127	0.125	0.018
M/Z_{tg}	N/mm ²	0.013	0.085	0.159	0.152	0.265	0.336	0.359	0.335	0.265	0.154	0.083	0.085	0.013
$-M/Z_{bg}$	N/mm ²	-0.031	-0.209	-0.451	-0.511	-0.889	-1.127	-1.204	-1.125	-0.888	-0.518	-0.235	-0.209	-0.031

STRESS IN COMPOSITE GIRDER DUE TO SIDL2 DUE TO WEARING COAT**(At 63 days)**

Moment due to SIDL2	kNm	0.25	106.80	106.85	190.28	312.01	390.21	416.24	390.21	312.16	191.39	106.85	106.80	0.25
M/Z_{ts}	N/mm ²	0.000	0.125	0.128	0.228	0.373	0.467	0.498	0.467	0.373	0.229	0.128	0.125	0.000
M/Z_{tg}	N/mm ²	0.000	0.085	0.084	0.142	0.233	0.292	0.311	0.292	0.234	0.143	0.084	0.085	0.000
$-M/Z_{bg}$	N/mm ²	0.000	-0.209	-0.237	-0.478	-0.783	-0.980	-1.045	-0.980	-0.784	-0.480	-0.237	-0.209	0.000

NET STRESS IN GIRDER**(AT 63 days)****JUST AFTER SIDL PLACEMENT**

At Top of Deck Slab,fts	N/mm ²	0.019	0.251	0.371	0.471	0.797	1.004	1.071	1.003	0.796	0.476	0.254	0.251	0.019
At Top of Precast Girder,ftg	N/mm ²	2.772	2.559	3.024	4.481	4.894	5.626	6.205	5.565	4.811	4.442	2.928	2.598	2.821
At Bottom of Precast Girder,fbg	N/mm ²	7.774	7.792	9.888	12.112	11.453	10.591	10.014	10.899	11.936	12.616	10.499	8.054	8.026

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.657	4.616	6.624	9.250	9.863	9.705	9.348	9.947	10.211	9.553	6.901	4.745	4.785
Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg})	=	7.708	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	6.162	6.161	8.328	10.791	10.620	10.058	9.609	10.326	11.031	11.205	8.783	6.358	6.352
Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg})	=	8.907	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	172.810	172.810	169.317	168.275	168.275	168.275	168.275	168.275	168.275	168.275	169.317	172.810	172.810
shrinkage loss between (63 days to Infinity)	%	2.713	2.712	2.637	2.595	2.552	2.512	2.491	2.470	2.493	2.533	2.572	2.643	2.644
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	66.520	66.520	65.175	64.774	64.774	64.774	64.774	64.774	64.774	64.774	65.175	66.520	66.520

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	172.810	172.810	169.317	168.275	168.275	168.275	168.275	168.275	168.275	168.275	169.317	172.810	172.810
shrinkage loss between (63 days to Infinity)	%	2.710	2.709	2.634	2.593	2.561	2.537	2.519	2.502	2.499	2.514	2.553	2.624	2.625
Stress loss due to Shrinkage = ecs x E_p x A_{cable}	kN	106.431	106.431	104.280	103.639	103.639	103.639	103.639	103.639	103.639	103.639	104.280	106.431	106.431

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	$\sigma_{cp,Avg,-I}$	=	7.708 N/mm ²	Modulus of elasticity of prestressing steel, E_{ps}		=	195000 N/mm ²
Average stress at c.g. of stage-II Cables	$\sigma_{cp,Avg,-II}$	=	8.907 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		=	$\phi(t_2, t_0)$				

$\phi(t_2, t_0)$		1.781	1.781	1.804	1.829	1.829	1.829	1.829	1.829	1.829	1.829	1.804	1.781	1.781
$\phi(t_1, t_0)$		0.679	0.679	0.688	0.720	0.720	0.720	0.720	0.720	0.720	0.720	0.688	0.679	0.679
Creep loss for Stage I Cables	%	3.922	3.920	3.939	3.878	3.813	3.754	3.722	3.691	3.726	3.785	3.842	3.821	3.822
	kN	96.151	96.151	97.368	96.795	96.795	96.795	96.795	96.795	96.795	96.795	97.368	96.151	96.151

$\phi(t_2, t_0)$		1.781	1.781	1.804	1.829	1.829	1.829	1.829	1.829	1.829	1.829	1.804	1.781	1.781
$\phi(t_1, t_0)$		0.679	0.679	0.688	0.720	0.720	0.720	0.720	0.720	0.720	0.720	0.688	0.679	0.679
Creep loss for Stage II Cables	%	4.527	4.525	4.547	4.477	4.422	4.381	4.350	4.320	4.314	4.342	4.407	4.383	4.384
	kN	177.761	177.761	180.011	178.951	178.951	178.951	178.951	178.951	178.951	178.951	180.011	177.761	177.761

TOTAL PRESTRESS LOSS (Shrinkage , Relaxation & Creep Loss)**From 63 days to Infinity**

Total Prestress Loss for Stage-I Cables	%	10.829	10.833	10.906	10.969	11.149	11.357	11.545	11.729	11.523	11.231	11.169	11.099	11.094
	kN	265.495	265.729	269.571	273.818	283.024	292.861	300.268	307.565	299.388	287.244	283.055	279.304	279.081
Total Prestress Loss for Stage-II Cables	%	11.515	11.520	11.598	11.653	11.781	11.880	11.960	12.110	12.139	12.002	11.921	11.844	11.839
	kN	452.160	452.542	459.140	465.833	476.798	485.297	492.028	501.620	503.496	494.706	486.948	480.383	480.010

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	265.495	265.729	269.571	273.818	283.024	292.861	300.268	307.565	299.388	287.244	283.055	279.304	279.081
Eccentricity of Stage I cable from N.A. (ec)	m	-0.535	-0.509	-0.343	-0.233	-0.020	0.082	0.088	0.082	-0.020	-0.232	-0.343	-0.508	-0.534
$M_p = P_{hor} \times ec$	kNm	-141.980	-135.162	-92.530	-63.730	-5.771	23.981	26.309	25.143	-5.962	-66.696	-96.987	-141.877	-149.054

Stage II Cables

Prestress After SH+CR+RLEX Loss(P_{hor})	kN	452.160	452.542	459.140	465.833	476.798	485.297	492.028	501.620	503.496	494.706	486.948	480.383	480.010
Eccentricity of Stage II cable from N.A. (ec)	m	-0.053	-0.036	0.054	0.090	0.164	0.196	0.197	0.196	0.164	0.091	0.055	-0.035	-0.052
$M_p = P_{hor} \times ec$	kNm	-24.145	-16.415	24.807	42.079	78.299	94.968	97.137	98.084	82.727	45.113	26.823	-16.824	-25.020

For Stage I & Stage II Cables

P_{hor} (Stage I & Stage II Cables)	kN	717.654	718.271	728.711	739.651	759.822	778.158	792.296	809.185	802.884	781.950	770.003	759.687	759.091
M_p (Stage I & Stage II Cables)	kNm	-166.125	-151.577	-67.722	-21.651	72.527	118.949	123.446	123.227	76.764	-21.584	-70.164	-158.701	-174.074
P_{hor} / A	N/mm ²	0.399	0.399	0.466	0.548	0.563	0.576	0.587	0.599	0.595	0.579	0.493	0.422	0.422
M_p / Z_{ts}	N/mm ²	-0.195	-0.178	-0.081	-0.026	0.087	0.142	0.148	0.147	0.092	-0.026	-0.084	-0.186	-0.204
M_p / Z_{ta}	N/mm ²	-0.132	-0.120	-0.053	-0.016	0.054	0.089	0.092	0.092	0.057	-0.016	-0.055	-0.126	-0.138
M_p / Z_{bg}	N/mm ²	-0.325	-0.296	-0.150	-0.054	0.182	0.299	0.310	0.309	0.193	-0.054	-0.155	-0.310	-0.340
$P_{hor} / A - M_p / Z_{ts}$	N/mm ²	0.594	0.577	0.547	0.574	0.476	0.434	0.439	0.452	0.503	0.605	0.576	0.609	0.626
$P_{hor} / A - M_p / Z_{ta}$	N/mm ²	0.531	0.520	0.519	0.564	0.509	0.487	0.494	0.507	0.537	0.595	0.547	0.549	0.560
$P_{hor} / A + M_p / Z_{bg}$	N/mm ²	0.074	0.103	0.316	0.493	0.745	0.875	0.897	0.909	0.787	0.525	0.337	0.112	0.082

NET STRESS IN GIRDER

(At Infinity

At Top of Deck Slab,fts	N/mm ²	-0.575	-0.327	-0.176	-0.102	0.321	0.570	0.632	0.551	0.294	-0.129	-0.322	-0.358	-0.607
At Top of Precast Girder,ftg	N/mm ²	2.241	2.039	2.505	3.917	4.386	5.138	5.711	5.058	4.274	3.846	2.381	2.049	2.261
At Bottom of Precast Girder,fbg	N/mm ²	7.699	7.689	9.572	11.619	10.708	9.716	9.117	9.990	11.148	12.091	10.161	7.942	7.944

Stress in Concrete at c.g. of Stage I Cable	N/mm ²	4.298	4.260	6.211	8.730	9.176	8.899	8.522	9.110	9.484	9.002	6.464	4.368	4.405

Average stress in concrete at c.g. of Stage- I Cable(σ_{cg_avg}) = 7.148 N/mm²

Stress at c.g of Stage II Cables	N/mm ²	5.940	5.928	7.965	10.285	9.905	9.225	8.755	9.460	10.276	10.668	8.398	6.110	6.117

Average stress in concrete at c.g. of Stage- II Cable(σ_{cg_avg}) = 8.387 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	54.07	432.20	822.57	822.57	1574.18	2107.23	2289.01	2041.84	1463.15	809.77	410.92	413.64	51.96
At Top of Deck Slab, fts	N/mm ²	0.063	0.507	0.982	0.984	1.883	2.521	2.738	2.442	1.750	0.969	0.491	0.485	0.061
At Top of Precast Girder,ftg	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
At Bottom of Precast Girder,fbg	N/mm ²	-0.106	-0.845	-1.821	-2.065	-3.952	-5.290	-5.746	-5.125	-3.673	-2.033	-0.910	-0.809	-0.102

Stress in Composite Girder due to CWLL + FPLL**(At Infinity)**

At Top of Deck Slab, fts	N/mm ²	0.063	0.507	0.982	0.984	1.883	2.521	2.738	2.442	1.750	0.969	0.491	0.485	0.061
At Top of Precast Girder,ftg	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
At Bottom of Precast Girder,fbg	N/mm ²	-0.106	-0.845	-1.821	-2.065	-3.952	-5.290	-5.746	-5.125	-3.673	-2.033	-0.910	-0.809	-0.102

NET STRESS IN GIRDER**(At Infinity)****JUST AFTER CWLL+FPLL**

At Top of Deck Slab, fts	N/mm ²	-0.512	0.180	0.806	0.881	2.204	3.090	3.370	2.993	2.044	0.839	0.169	0.127	-0.547
At Top of Precast Girder,ftg	N/mm ²	2.284	2.382	3.148	4.533	5.563	6.715	7.423	6.586	5.369	4.452	2.702	2.378	2.302
At Bottom of Precast Girder,fbg	N/mm ²	7.594	6.844	7.750	9.554	6.756	4.426	3.371	4.865	7.475	10.059	9.252	7.134	7.843

Stress at c.g of Stage I Cables	N/mm ²	4.285	4.136	5.562	7.671	6.467	4.835	4.080	5.172	6.965	7.958	6.139	4.250	4.392
Average stress in concrete at c.g. of Stage- I Cable($\sigma_{ca\ avg}$)	=	5.532	N/mm ²											
Stress at c.g of Stage II Cables	N/mm ²	5.882	5.453	6.704	8.684	6.605	4.672	3.802	5.050	7.208	9.091	7.767	5.655	6.061
Average stress in concrete at c.g. of Stage- II Cable($\sigma_{ca\ avg}$)	=	6.357	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	%	0.783	0.783	0.803	0.831	0.817	0.804	0.797	0.791	0.798	0.811	0.783	0.763	0.763
	kN	19.205	19.205	19.841	20.739	20.739	20.739	20.739	20.739	20.739	20.739	19.841	19.205	19.205
Creep Loss	%	0.739	0.739	0.742	0.770	0.757	0.745	0.739	0.733	0.740	0.751	0.724	0.720	0.720
	kN	18.121	18.121	18.351	19.219	19.219	19.219	19.219	19.219	19.219	19.219	18.351	18.121	18.121
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)	%	3.411	3.414	3.495	3.626	3.729	3.843	3.938	4.031	3.927	3.776	3.649	3.571	3.568
	kN	83.638	83.744	86.397	90.516	94.662	99.093	102.429	105.715	102.033	96.563	92.470	89.858	89.757

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	%	0.073	0.073	0.078	0.086	0.085	0.084	0.083	0.082	0.083	0.084	0.076	0.071	0.071
	kN	1.780	1.780	1.937	2.154	2.154	2.154	2.154	2.154	2.154	2.154	1.937	1.780	1.780
Creep Loss	%	0.209	0.209	0.210	0.217	0.213	0.210	0.208	0.207	0.209	0.212	0.205	0.203	0.204
	kN	5.119	5.119	5.184	5.418	5.418	5.418	5.418	5.418	5.418	5.418	5.184	5.119	5.119
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)	%	0.352	0.352	0.361	0.379	0.379	0.379	0.381	0.382	0.381	0.379	0.361	0.352	0.352
	kN	8.626	8.630	8.919	9.458	9.612	9.777	9.902	10.024	9.887	9.683	9.145	8.858	8.855

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 35 days to 49 days**

Shrinkage Loss	%	0.121	0.121	0.132	0.146	0.144	0.141	0.140	0.139	0.140	0.143	0.128	0.118	0.118
	kN	2.961	2.961	3.252	3.647	3.647	3.647	3.647	3.647	3.647	3.647	3.252	2.961	2.961
Creep Loss	%	0.273	0.273	0.274	0.284	0.279	0.275	0.272	0.270	0.273	0.277	0.268	0.266	0.266
	kN	6.699	6.699	6.784	7.082	7.082	7.082	7.082	7.082	7.082	7.082	6.784	6.699	6.699
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)	%	0.532	0.532	0.548	0.577	0.580	0.583	0.587	0.592	0.587	0.581	0.552	0.536	0.536
	kN	13.033	13.041	13.546	14.411	14.713	15.035	15.278	15.518	15.249	14.851	13.988	13.486	13.479

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 49 days to 63 days**

Shrinkage Loss	%	0.100	0.100	0.110	0.123	0.121	0.119	0.118	0.117	0.118	0.120	0.107	0.098	0.098
	kN	2.457	2.457	2.718	3.068	3.068	3.068	3.068	3.068	3.068	3.068	2.718	2.457	2.457
Creep Loss	%	0.211	0.211	0.212	0.219	0.215	0.212	0.210	0.208	0.210	0.213	0.206	0.205	0.205
	kN	5.168	5.168	5.233	5.455	5.455	5.455	5.455	5.455	5.455	5.455	5.233	5.168	5.168
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)	%	0.311	0.311	0.322	0.341	0.336	0.331	0.328	0.325	0.328	0.333	0.314	0.303	0.303
	kN	7.624	7.624	7.951	8.522	8.522	8.522	8.522	8.522	8.522	8.522	7.951	7.624	7.624

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 63 days to Infinity**

Shrinkage Loss	%	2.713	2.712	2.637	2.595	2.552	2.512	2.491	2.470	2.493	2.533	2.572	2.643	2.644
	kN	66.520	66.520	65.175	64.774	64.774	64.774	64.774	64.774	64.774	64.774	65.175	66.520	66.520
Creep Loss	%	3.922	3.920	3.939	3.878	3.813	3.754	3.722	3.691	3.726	3.785	3.842	3.821	3.822
	kN	96.151	96.151	97.368	96.795	96.795	96.795	96.795	96.795	96.795	96.795	97.368	96.151	96.151
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)	%	10.829	10.833	10.906	10.969	11.149	11.357	11.545	11.729	11.523	11.231	11.169	11.099	11.094
	kN	265.495	265.729	269.571	273.818	283.024	292.861	300.268	307.565	299.388	287.244	283.055	279.304	279.081

TOTAL PRESTRESS LOSS**From 7 days to Infinity**

STAGE I CABLES	%	20.783	20.744	20.157	19.442	17.968	16.591	15.866	15.163	15.951	17.306	17.979	18.525	18.561
	kN	553.043	552.037	536.939	518.551	480.888	445.744	427.287	409.412	429.464	463.988	481.212	495.202	496.115

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	%	0.073	0.072	0.078	0.086	0.085	0.084	0.084	0.083	0.083	0.084	0.076	0.070	0.070
	kN	2.847	2.847	3.099	3.446	3.446	3.446	3.446	3.446	3.446	3.446	3.099	2.847	2.847
Creep Loss	%	0.252	0.251	0.253	0.262	0.258	0.256	0.254	0.252	0.252	0.254	0.245	0.244	0.244
	kN	9.879	9.879	10.004	10.455	10.455	10.455	10.455	10.455	10.455	10.455	10.004	9.879	9.879
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)	%	1.940	1.943	1.999	2.079	2.156	2.215	2.261	2.333	2.347	2.281	2.195	2.141	2.138
	kN	76.177	76.321	79.153	83.122	87.264	90.475	93.017	96.641	97.350	94.029	89.657	86.838	86.698

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 35 days to 49 days**

Shrinkage Loss	%	0.121	0.121	0.131	0.146	0.144	0.143	0.142	0.141	0.141	0.142	0.127	0.117	0.117
	kN	4.738	4.738	5.203	5.836	5.836	5.836	5.836	5.836	5.836	5.836	5.203	4.738	4.738
Creep Loss	%	0.318	0.318	0.319	0.330	0.326	0.323	0.321	0.319	0.318	0.320	0.309	0.308	0.308
	kN	12.483	12.483	12.641	13.197	13.197	13.197	13.197	13.197	13.197	13.197	12.641	12.483	12.483
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)	%	0.719	0.719	0.740	0.777	0.785	0.791	0.796	0.806	0.808	0.799	0.762	0.742	0.741
	kN	28.232	28.257	29.305	31.044	31.763	32.320	32.761	33.390	33.513	32.937	31.128	30.082	30.058

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 49 days to 63 days**

Shrinkage Loss	%	0.100	0.100	0.110	0.123	0.121	0.120	0.119	0.118	0.118	0.119	0.106	0.097	0.097
	kN	3.931	3.931	4.349	4.908	4.908	4.908	4.908	4.908	4.908	4.908	4.349	3.931	3.931
Creep Loss	%	0.245	0.245	0.246	0.254	0.251	0.249	0.247	0.245	0.245	0.246	0.239	0.237	0.237
	kN	9.625	9.625	9.747	10.160	10.160	10.160	10.160	10.160	10.160	10.160	9.747	9.625	9.625
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)	%	0.487	0.487	0.502	0.529	0.531	0.533	0.535	0.539	0.539	0.536	0.509	0.494	0.494
	kN	19.111	19.124	19.878	21.128	21.491	21.772	21.995	22.312	22.374	22.083	20.798	20.044	20.032

LOSS OF PRESTRESS DUE TO SHRINKAGE, CREEP & RELAXATION OF STEEL**From 63 days to Infinity**

Shrinkage Loss	%	2.710	2.709	2.634	2.593	2.561	2.537	2.519	2.502	2.499	2.514	2.553	2.624	2.625
	kN	106.431	106.431	104.280	103.639	103.639	103.639	103.639	103.639	103.639	103.639	104.280	106.431	106.431
Creep Loss	%	4.527	4.525	4.547	4.477	4.422	4.381	4.350	4.320	4.314	4.342	4.407	4.383	4.384
	kN	177.761	177.761	180.011	178.951	178.951	178.951	178.951	178.951	178.951	178.951	180.011	177.761	177.761
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)	%	11.515	11.520	11.598	11.653	11.781	11.880	11.960	12.110	12.139	12.002	11.921	11.844	11.839
	kN	452.160	452.542	459.140	465.833	476.798	485.297	492.028	501.620	503.496	494.706	486.948	480.383	480.010

TOTAL PRESTRESS LOSS**From 28 days to Infinity**

STAGE II CABLES	%	16.006	15.968	15.369	14.630	13.587	12.793	12.190	11.629	11.520	12.032	12.725	13.290	13.325
	kN	690.959	689.344	663.610	631.885	587.455	553.666	528.032	504.177	499.551	521.339	550.903	575.026	576.513

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	0.073	0.055	-0.013	-0.049	-0.053	-0.052	-0.014	0.059	0.078	0.113	0.120
At Bottom of Precast Girder,fbg	N/mm ²	0.028	0.035	0.113	0.198	0.282	0.332	0.345	0.354	0.304	0.212	0.121	0.037	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	-0.089	-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	0.223	0.302	0.348	0.374	0.385	0.398	0.385	0.340	0.252	0.169	0.163
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	0.016	0.013	-0.001	-0.007	-0.027	-0.036	-0.037	-0.038	-0.028	-0.008	-0.002	0.013	0.016
	At Bottom of Precast Girder,fbg	N/mm ²	0.056	0.059	0.096	0.137	0.160	0.172	0.175	0.178	0.168	0.144	0.102	0.063	0.060
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	0.021	0.020	0.020	0.022	0.018	0.017	0.017	0.017	0.019	0.023	0.020	0.021	0.021
	At Bottom of Deck Slab, fbs	N/mm ²	0.019	0.019	0.019	0.022	0.020	0.019	0.019	0.019	0.020	0.023	0.020	0.019	0.019
	At Top of Precast Girder,ftg	N/mm ²	0.019	0.019	0.019	0.022	0.020	0.019	0.019	0.019	0.020	0.023	0.020	0.019	0.019
	At Bottom of Precast Girder,fbg	N/mm ²	0.005	0.006	0.014	0.022	0.031	0.035	0.035	0.036	0.032	0.023	0.015	0.006	0.005
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	0.594	0.577	0.547	0.574	0.476	0.434	0.439	0.452	0.503	0.605	0.576	0.609	0.626
	At Bottom of Deck Slab, fbs	N/mm ²	0.531	0.520	0.519	0.564	0.509	0.487	0.494	0.507	0.537	0.595	0.547	0.549	0.560
	At Top of Precast Girder,ftg	N/mm ²	0.531	0.520	0.519	0.564	0.509	0.487	0.494	0.507	0.537	0.595	0.547	0.549	0.560
	At Bottom of Precast Girder,fbg	N/mm ²	0.074	0.103	0.316	0.493	0.745	0.875	0.897	0.909	0.787	0.525	0.337	0.112	0.082

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	-0.004	1.339	2.616	4.267	5.335	5.691	5.335	4.267	2.616	1.339	-0.004	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.004	-1.439	-2.741	-4.471	-5.590	-5.963	-5.590	-4.471	-2.741	-1.439	0.004	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	-0.011	-0.018	-0.022	-0.024	-0.022	-0.018	-0.011	-0.006	0.000	0.000
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.018	0.037	0.060	0.075	0.080	0.075	0.060	0.037	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	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26	-1.26
	At Bottom of Deck Slab, fbs	N/mm ²	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38	-1.38
	At Top of Precast Girder,ftg	N/mm ²	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64	0.64
	At Bottom of Precast Girder,fbg	N/mm ²	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27
6	STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT (G2)		(At 63 days)												
	At Top of Deck Slab, fts	N/mm ²	0.018	0.125	0.243	0.244	0.424	0.537	0.574	0.536	0.423	0.247	0.127	0.125	0.018

At Bottom of Deck Slab, fbs	N/mm ²	0.013	0.085	0.159	0.152	0.265	0.336	0.359	0.335	0.265	0.154	0.083	0.085	0.013
At Top of Precast Girder,ftg	N/mm ²	0.013	0.085	0.159	0.152	0.265	0.336	0.359	0.335	0.265	0.154	0.083	0.085	0.013
At Bottom of Precast Girder,fbg	N/mm ²	-0.031	-0.209	-0.451	-0.511	-0.889	-1.127	-1.204	-1.125	-0.888	-0.518	-0.235	-0.209	-0.031

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	0.125	0.128	0.228	0.373	0.467	0.498	0.467	0.373	0.229	0.128	0.125	0.000			
	At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.085	0.084	0.142	0.233	0.292	0.311	0.292	0.234	0.143	0.084	0.085	0.000			
	At Top of Precast Girder,ftg	N/mm ²	0.000	0.085	0.084	0.142	0.233	0.292	0.311	0.292	0.234	0.143	0.084	0.085	0.000			
	At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.209	-0.237	-0.478	-0.783	-0.980	-1.045	-0.980	-0.784	-0.480	-0.237	-0.209	0.000			

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	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	0.000	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

9	Stress in Composite Girder due to CWLL (WITH IMPACT)														
				(Q2)			(At Infinity								
	At Top of Deck Slab, fts	N/mm ²	0.063	0.507	0.982	0.984	1.883	2.521	2.738	2.442	1.750	0.969	0.491	0.485	0.061
	At Bottom of Deck Slab, fbs	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
	At Top of Precast Girder,ftg	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
	At Bottom of Precast Girder,fbg	N/mm ²	-0.106	-0.845	-1.821	-2.065	-3.952	-5.290	-5.746	-5.125	-3.673	-2.033	-0.910	-0.809	-0.102

10	Stress due to Positive Temperature Diff. (Q5,1)														
	At Top of Deck Slab, fts	N/mm ²	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22	4.22
	At Bottom of Deck Slab, fbs	N/mm ²	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37
	At Top of Precast Girder,ftg	N/mm ²	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37	-1.37
	At Bottom of Precast Girder,fbg	N/mm ²	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88	1.88

11	Stress due to Reverse Temperature Diff. (Q5,2)														
	At Top of Deck Slab, fts	N/mm ²	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28	-2.28
	At Bottom of Deck Slab, fbs	N/mm ²	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
	At Top of Precast Girder,ftg	N/mm ²	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
	At Bottom of Precast Girder,fbg	N/mm ²	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49	-2.49

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	3.295	2.889	0.518	-0.530	-3.693	-5.212	-5.334	-5.285	-3.787	-0.573	0.503	2.942	3.359
At Bottom of Precast Girder,fbg	N/mm ²	8.460	8.905	14.562	20.064	23.650	25.475	25.757	25.856	24.223	20.651	15.010	9.198	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	3.295	2.884	1.665	1.655	-0.206	-0.884	-0.725	-0.956	-0.301	1.612	1.650	2.937	3.359
At Bottom of Precast Girder,fbg	N/mm ²	8.460	8.910	13.329	17.774	19.996	20.939	20.927	21.320	20.570	18.361	13.777	9.204	8.742

Maximum Compressive Stress in Girder = 21.320 N/mm² < **21.600** N/mm² (Allowable compressive stress) **SAFE**

Minimum Compressive Stress in Girder = -0.956 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	3.302	2.892	0.492	-0.568	-3.763	-5.298	-5.423	-5.376	-3.865	-0.616	0.472	2.943	3.365
At Bottom of Precast Girder,fbg	N/mm ²	8.604	9.053	14.786	20.365	23.998	25.849	26.142	26.254	24.609	20.990	15.262	9.367	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	-0.009	2.512	4.852	7.837	9.768	10.411	9.768	7.837	4.852	2.512	-0.009	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.010	-2.700	-5.085	-8.213	-10.236	-10.911	-10.236	-8.213	-5.085	-2.700	0.010	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	3.302	2.882	3.004	4.284	4.074	4.470	4.988	4.392	3.972	4.236	2.984	2.933	3.365
At Bottom of Precast Girder,fbg	N/mm ²	8.604	9.063	12.085	15.281	15.785	15.613	15.231	16.018	16.396	15.906	12.562	9.378	8.906

Maximum Compressive Stress in Girder = 16.396 N/mm² < **21.600** N/mm² (Allowable compressive stress) **SAFE**

Minimum Compressive Stress in Girder = 2.882 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	0.491	-0.576	-3.790	-5.335	-5.461	-5.414	-3.893	-0.625	0.471	2.956	3.381
At Bottom of Precast Girder,fbg	N/mm ²	8.660	9.112	14.882	20.502	24.158	26.022	26.317	26.432	24.777	21.135	15.363	9.430	8.966

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	-0.009	2.512	4.852	7.837	9.768	10.411	9.768	7.837	4.852	2.512	-0.009	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.010	-2.700	-5.085	-8.213	-10.236	-10.911	-10.236	-8.213	-5.085	-2.700	0.010	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	-0.011	-0.018	-0.022	-0.024	-0.022	-0.018	-0.011	-0.006	0.000	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	0.000	0.018	0.037	0.060	0.075	0.080	0.075	0.060	0.037	0.018	0.000	0.000

Stress due to Differential Shrinkage (G10)

At Top of Deck Slab, fts	N/mm ²	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259
At Bottom of Deck Slab, fbs	N/mm ²	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383
At Top of Precast Girder,ftg	N/mm ²	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639	0.639
At Bottom of Precast Girder,fbg	N/mm ²	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266	-0.266

Resultant stress in Girder 1.0 G8 + 1.0 G1 +1.0 G10

At Top of Deck Slab, fts	N/mm ²	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259
At Top of Deck Slab, fts	N/mm ²	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383
At Top of Precast Girder,ftg	N/mm ²	3.956	3.534	3.636	4.904	4.668	5.050	5.566	4.970	4.564	4.855	3.615	3.586	4.020
At Bottom of Precast Girder,fbg	N/mm ²	8.394	8.856	11.933	15.188	15.738	15.594	15.220	16.004	16.357	15.820	12.415	9.174	8.700

Maximum Compressive Stress in Girder	=	16.357	N/mm ²	<	21.600 N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.534	N/mm ²	>	-3.3 N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.383	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	0.021	0.020	0.020	0.022	0.018	0.017	0.017	0.017	0.019	0.023	0.020	0.021	0.021
At Bottom of Deck Slab, fbs	N/mm ²	0.019	0.019	0.019	0.022	0.020	0.019	0.019	0.019	0.020	0.023	0.020	0.019	0.019
At Top of Precast Girder,ftg	N/mm ²	3.336	2.923	0.510	-0.554	-3.770	-5.316	-5.442	-5.395	-3.873	-0.602	0.490	2.975	3.401
At Bottom of Precast Girder,fbg	N/mm ²	8.665	9.118	14.896	20.524	24.188	26.056	26.352	26.467	24.808	21.157	15.378	9.437	8.971

X2	=	X1	+	(G1a)	+	(G10)								
At Top of Deck Slab, fts	N/mm ²	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259	-1.259
At Bottom of Deck Slab, fbs	N/mm ²	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383	-1.383
At Top of Precast Girder,ftg	N/mm ²	0.639	0.629	3.144	5.480	8.458	10.384	11.026	10.384	8.458	5.480	3.144	0.629	0.639
At Bottom of Precast Girder,fbg	N/mm ²	-0.266	-0.256	-2.948	-5.314	-8.419	-10.428	-11.097	-10.428	-8.419	-5.314	-2.948	-0.256	-0.266

STRESS IN COMPOSITE GIRDER DUE TO SIDL1 WITHOUT WEARING COAT (G2)

At Top of Deck Slab, fts	N/mm ²	0.018	0.125	0.243	0.244	0.424	0.537	0.574	0.536	0.423	0.247	0.127	0.125	0.018
At Bottom of Deck Slab, fbs	N/mm ²	0.013	0.085	0.159	0.152	0.265	0.336	0.359	0.335	0.265	0.154	0.083	0.085	0.013
At Top of Precast Girder,ftg	N/mm ²	0.013	0.085	0.159	0.152	0.265	0.336	0.359	0.335	0.265	0.154	0.083	0.085	0.013
At Bottom of Precast Girder,fbg	N/mm ²	-0.031	-0.209	-0.451	-0.511	-0.889	-1.127	-1.204	-1.125	-0.888	-0.518	-0.235	-0.209	-0.031

STRESS IN COMPOSITE GIRDER DUE TO SIDL2 DUE TO WEARING COAT (G3)

At Top of Deck Slab, fts	N/mm ²	0.000	0.125	0.128	0.228	0.373	0.467	0.498	0.467	0.373	0.229	0.128	0.125	0.000
At Bottom of Deck Slab, fbs	N/mm ²	0.000	0.085	0.084	0.142	0.233	0.292	0.311	0.292	0.234	0.143	0.084	0.085	0.000
At Top of Precast Girder,ftg	N/mm ²	0.000	0.085	0.084	0.142	0.233	0.292	0.311	0.292	0.234	0.143	0.084	0.085	0.000
At Bottom of Precast Girder,fbg	N/mm ²	0.000	-0.209	-0.237	-0.478	-0.783	-0.980	-1.045	-0.980	-0.784	-0.480	-0.237	-0.209	0.000

Stress due to Positive Temperature Diff. (Q5,1)

At Top of Deck Slab, fts	N/mm ²	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224	4.224
At Bottom of Deck Slab, fbs	N/mm ²	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372
At Top of Precast Girder,ftg	N/mm ²	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372	-1.372
At Bottom of Precast Girder,fbg	N/mm ²	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883	1.883

Resultant stress in Girder Case 1 : WITHOUT CWLL PLACEMENT & TEMPERATURE RISE CASE

At Top of Deck Slab, fts	N/mm ²	1.315	1.547	1.666	1.769	2.091	2.296	2.364	2.295	2.091	1.774	1.550	1.547	1.316
At Bottom of Deck Slab, fbs	N/mm ²	-2.175	-2.018	-1.944	-1.890	-1.688	-1.560	-1.518	-1.560	-1.688	-1.886	-2.020	-2.018	-2.174
At Top of Precast Girder,ftg	N/mm ²	3.165	2.899	3.075	4.398	4.363	4.873	5.432	4.794	4.260	4.352	2.978	2.951	3.229
At Bottom of Precast Girder,fbg	N/mm ²	9.497	9.574	12.389	15.350	15.227	14.652	14.136	15.065	15.847	15.975	13.088	9.892	9.803

Maximum Compressive Stress in Girder	=	15.975	N/mm ²	<	21.600	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	2.899	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.175	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	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284	-2.284
At Bottom of Deck Slab, fbs	N/mm ²	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049
At Top of Precast Girder,ftg	N/mm ²	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049	1.049
At Bottom of Precast Girder,fbg	N/mm ²	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487	-2.487

Resultant stress in Girder Case 2 : WITHOUT CWLL PLACEMENT & TEMPERATURE FALLING CASE

At Top of Deck Slab, fts	N/mm ²	-2.590	-2.359	-2.239	-2.136	-1.815	-1.609	-1.542	-1.610	-1.814	-2.131	-2.355	-2.358	-2.589
At Bottom of Deck Slab, fbs	N/mm ²	-0.723	-0.566	-0.492	-0.438	-0.236	-0.108	-0.065	-0.108	-0.236	-0.434	-0.568	-0.565	-0.722
At Top of Precast Girder,ftg	N/mm ²	4.617	4.352	4.527	5.850	5.815	6.325	6.884	6.246	5.712	5.804	4.431	4.404	4.681
At Bottom of Precast Girder,fbg	N/mm ²	6.876	6.952	9.768	12.728	12.605	12.031	11.515	12.443	13.226	13.353	10.466	7.271	7.182

Maximum Compressive Stress in Girder	=	13.353	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	4.352	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.590	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	0.063	0.507	0.982	0.984	1.883	2.521	2.738	2.442	1.750	0.969	0.491	0.485	0.061
At Bottom of Deck Slab, fbs	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
At Top of Precast Girder,ftg	N/mm ²	0.043	0.343	0.643	0.615	1.178	1.576	1.712	1.528	1.095	0.606	0.321	0.329	0.041
At Bottom of Precast Girder,fbg	N/mm ²	-0.106	-0.845	-1.821	-2.065	-3.952	-5.290	-5.746	-5.125	-3.673	-2.033	-0.910	-0.809	-0.102

Resultant stress in Girder Case 3 : WITH CWLL PLACEMENT & TEMPERATURE RISING CASE

At Top of Deck Slab, fts	N/mm ²	1.379	2.053	2.649	2.753	3.974	4.816	5.102	4.738	3.841	2.742	2.041	2.032	1.377
At Bottom of Deck Slab, fbs	N/mm ²	-2.132	-1.675	-1.301	-1.274	-0.511	0.016	0.195	-0.033	-0.593	-1.281	-1.699	-1.689	-2.133
At Top of Precast Girder,ftg	N/mm ²	3.208	3.243	3.718	5.014	5.541	6.450	7.144	6.321	5.354	4.958	3.300	3.280	3.270
At Bottom of Precast Girder,fbg	N/mm ²	9.392	8.729	10.568	13.285	11.275	9.363	8.390	9.939	12.175	13.942	12.178	9.084	9.702

Maximum Compressive Stress in Girder	=	13.942	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.208	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.133	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	-2.527	-1.852	-1.257	-1.152	0.068	0.911	1.196	0.832	-0.064	-1.163	-1.864	-1.873	-2.528
At Bottom of Deck Slab, fbs	N/mm ²	-0.680	-0.223	0.151	0.178	0.941	1.469	1.647	1.420	0.859	0.172	-0.247	-0.237	-0.681
At Top of Precast Girder,ftg	N/mm ²	4.660	4.695	5.170	6.466	6.993	7.902	8.596	7.773	6.807	6.410	4.752	4.732	4.722
At Bottom of Precast Girder,fbg	N/mm ²	6.770	6.107	7.947	10.664	8.653	6.741	5.769	7.318	9.553	11.320	9.556	6.462	7.080

Maximum Compressive Stress in Girder	=	11.320	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	4.660	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-2.528	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	0.615	0.597	0.567	0.596	0.494	0.451	0.456	0.469	0.522	0.628	0.597	0.629	0.648
At Bottom of Deck Slab, fbs	N/mm ²	0.550	0.538	0.538	0.586	0.528	0.506	0.513	0.526	0.558	0.618	0.567	0.568	0.580
At Top of Precast Girder,ftg	N/mm ²	3.868	3.443	1.029	0.011	-3.262	-4.828	-4.947	-4.888	-3.336	-0.007	1.038	3.524	3.961
At Bottom of Precast Girder,fbg	N/mm ²	8.740	9.221	15.212	21.017	24.933	26.931	27.249	27.376	25.596	21.682	15.716	9.549	9.053

Resultant stress in Girder Case 1 : WITHOUT CWLL PLACEMENT & TEMPERATURE RISE CASE

At Top of Deck Slab, fts	N/mm ²	1.971	2.183	2.270	2.402	2.616	2.775	2.848	2.794	2.646	2.442	2.186	2.219	2.007
At Bottom of Deck Slab, fbs	N/mm ²	-1.589	-1.444	-1.371	-1.267	-1.127	-1.022	-0.972	-1.000	-1.095	-1.229	-1.416	-1.412	-1.556
At Top of Precast Girder,ftg	N/mm ²	4.083	3.764	3.697	4.963	4.545	4.878	5.432	4.812	4.463	4.947	3.630	3.852	4.185
At Bottom of Precast Girder,fbg	N/mm ²	10.446	10.599	14.227	17.945	18.465	18.220	17.758	18.711	19.194	18.668	14.997	10.960	10.790

Maximum Compressive Stress in Girder	=	19.194	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.630	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.589	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	-1.935	-1.722	-1.635	-1.503	-1.289	-1.130	-1.057	-1.111	-1.259	-1.464	-1.719	-1.687	-1.898
At Bottom of Deck Slab, fbs	N/mm ²	-0.137	0.008	0.081	0.185	0.325	0.430	0.480	0.452	0.357	0.223	0.036	0.040	-0.104
At Top of Precast Girder,ftg	N/mm ²	5.535	5.216	5.149	6.415	5.998	6.330	6.884	6.264	5.916	6.399	5.082	5.305	5.638
At Bottom of Precast Girder,fbg	N/mm ²	7.824	7.978	11.605	15.324	15.843	15.599	15.136	16.089	16.573	16.046	12.375	8.338	8.169

Maximum Compressive Stress in Girder	=	16.573	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	5.082	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

Resultant stress in Girder Case 3 : WITH CWLL PLACEMENT & TEMPERATURE RISING CASE

At Top of Deck Slab, fts	N/mm ²	2.034	2.690	3.252	3.386	4.499	5.295	5.586	5.236	4.396	3.410	2.677	2.704	2.068
At Bottom of Deck Slab, fbs	N/mm ²	-1.546	-1.101	-0.728	-0.652	0.051	0.554	0.741	0.527	0.000	-0.623	-1.095	-1.084	-1.515
At Top of Precast Girder,ftg	N/mm ²	4.126	4.107	4.340	5.579	5.723	6.454	7.144	6.340	5.558	5.553	3.951	4.181	4.227

At Bottom of Precast Girder,fbg	N/mm ²	10.340	9.754	12.406	15.880	14.513	12.931	12.012	13.586	15.522	16.635	14.087	10.151	10.689

Maximum Compressive Stress in Girder	=	16.635	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.951	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.546	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	-1.871	-1.215	-0.653	-0.519	0.594	1.390	1.681	1.331	0.491	-0.495	-1.228	-1.202	-1.837
At Bottom of Deck Slab, fbs	N/mm ²	-0.094	0.351	0.724	0.801	1.503	2.007	2.193	1.979	1.452	0.829	0.358	0.369	-0.062
At Top of Precast Girder,ftg	N/mm ²	5.578	5.559	5.792	7.031	7.175	7.906	8.596	7.792	7.010	7.005	5.403	5.633	5.679
At Bottom of Precast Girder,fbg	N/mm ²	7.718	7.133	9.784	13.259	11.892	10.309	9.390	10.964	12.900	14.013	11.465	7.529	8.067

Maximum Compressive Stress in Girder	=	14.013	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	5.403	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.871	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 $G_8 = 0.9$
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	1.848	2.064	2.157	2.283	2.517	2.685	2.757	2.700	2.541	2.316	2.067	2.093	1.877
At Bottom of Deck Slab, fbs	N/mm ²	-1.699	-1.552	-1.479	-1.384	-1.233	-1.123	-1.074	-1.106	-1.207	-1.353	-1.529	-1.526	-1.672
At Top of Precast Girder,ftg	N/mm ²	3.309	3.075	3.491	4.961	5.198	5.843	6.421	5.790	5.131	4.948	3.422	3.148	3.393
At Bottom of Precast Girder,fbg	N/mm ²	8.698	8.755	11.184	13.742	13.478	12.834	12.308	13.236	14.075	14.331	11.854	9.050	8.980

Maximum Compressive Stress in Girder	=	14.331	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.075	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.699	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	-1.996	-1.781	-1.692	-1.562	-1.339	-1.175	-1.102	-1.158	-1.312	-1.526	-1.778	-1.750	-1.963
At Bottom of Deck Slab, fbs	N/mm ²	-0.192	-0.046	0.027	0.127	0.272	0.380	0.429	0.399	0.301	0.161	-0.021	-0.017	-0.162
At Top of Precast Girder,ftg	N/mm ²	5.148	4.871	5.046	6.414	6.324	6.813	7.378	6.753	6.249	6.400	4.978	4.952	5.241
At Bottom of Precast Girder,fbg	N/mm ²	6.950	7.055	10.084	13.222	13.350	12.906	12.411	13.352	14.013	13.878	10.803	7.383	7.264

Maximum Compressive Stress in Girder	=	14.013	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	4.871	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.996	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	1.911	2.571	3.139	3.267	4.400	5.205	5.495	5.143	4.292	3.285	2.558	2.578	1.938
At Bottom of Deck Slab, fbs	N/mm ²	-1.656	-1.209	-0.836	-0.769	-0.055	0.453	0.638	0.422	-0.112	-0.747	-1.208	-1.197	-1.631
At Top of Precast Girder,ftg	N/mm ²	3.352	3.418	4.134	5.577	6.375	7.420	8.133	7.317	6.225	5.554	3.744	3.476	3.435
At Bottom of Precast Girder,fbg	N/mm ²	8.592	7.910	9.363	11.677	9.527	7.544	6.562	8.110	10.402	12.299	10.944	8.241	8.878

Maximum Compressive Stress in Girder	=	12.299	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.352	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE
Minimum stress in Deck slab	=	-1.656	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	-1.994	-1.334	-0.766	-0.638	0.495	1.300	1.590	1.238	0.386	-0.621	-1.347	-1.327	-1.967
At Bottom of Deck Slab, fbs	N/mm ²	-0.204	0.243	0.616	0.683	1.397	1.905	2.090	1.874	1.340	0.705	0.244	0.255	-0.178
At Top of Precast Girder,ftg	N/mm ²	4.804	4.871	5.586	7.029	7.828	8.872	9.586	8.769	7.677	7.006	5.196	4.928	4.887
At Bottom of Precast Girder,fbg	N/mm ²	5.970	5.288	6.742	9.055	6.905	4.923	3.941	5.489	7.781	9.677	8.322	5.620	6.257

Maximum Compressive Stress in Girder	=	9.677	N/mm ²	<	21.6	N/mm ²	(Allowable compressive stress)	SAFE
Minimum Compressive Stress in Girder	=	3.941	N/mm ²	>	-3.3	N/mm ²	(Allowable tensile stress)	SAFE

Design Calculation				RODIC		Final Stress
Minimum stress in Deck slab	=	-1.994	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	-1.9	562.5	1002.4	1634.9	2044.1	2180.5	2044.1	1634.9	1002.4	562.5	-1.9	0.0
	1.35 x SIDL1	21.3	144.4	275.0	275.0	478.0	606.0	647.4	605.1	477.5	278.4	143.4	144.4	21.3
	1.75 x SIDL2	0.4	186.9	187.0	333.0	546.0	682.9	728.4	682.9	546.3	334.9	187.0	186.9	0.4
	1.5 LL	81.1	648.3	1233.8	1233.8	2361.3	3160.8	3433.5	3062.8	2194.7	1214.7	616.4	620.5	77.9
	TOTAL M_u	102.8	975.3	2740.2	3681.5	6356.2	8152.4	8756.0	8053.5	6189.4	3667.5	1991.1	947.5	99.7
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}	367.6	367.6	316.5	270.1	183.8	91.9	0.0	91.9	183.8	270.1	316.5	367.6	367.6
	1.35 x SIDL1	100.4	100.4	87.7	87.7	71.6	43.3	14.2	14.2	43.3	71.6	87.7	100.4	100.4
	1.75 x SIDL2	115.5	110.5	99.5	95.2	70.4	40.2	9.5	21.1	51.8	81.1	99.5	110.5	115.5
	1.5 LL	401.0	368.9	403.2	403.2	405.8	189.5	42.5	241.4	334.5	379.5	396.7	393.8	393.8
	TOTAL V_u	1310.6	1273.5	1167.7	1069.1	876.5	437.4	66.3	441.1	758.3	1015.2	1161.3	1298.4	1303.4

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	94.2	94.2	71.9	71.9	42.6	22.1	7.0	7.0	22.1	42.6	71.9	94.2	94.2
	1.75 x SIDL2	2.0	2.0	1.5	1.5	0.8	0.3	0.1	0.1	0.3	0.8	1.5	2.0	2.0
	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	169.0	169.0	146.2	104.1	62.1	32.4	19.7	18.7	38.5	52.5	83.1	110.2	110.2

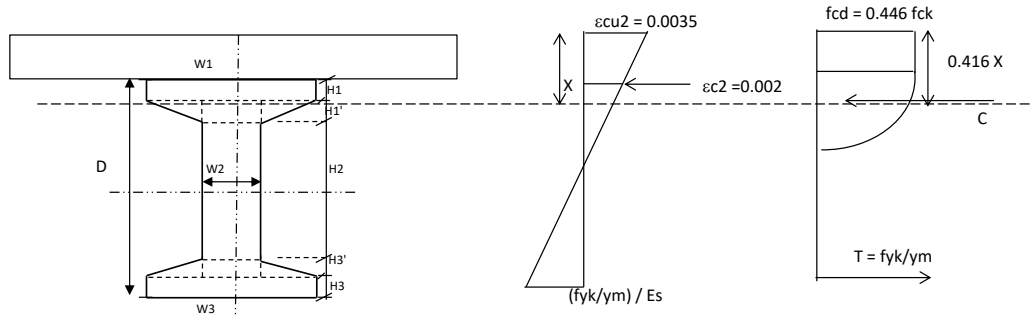
b) Maximum Shear force and Corressponding Bending Moment

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}	367.6	367.6	316.5	270.1	183.8	91.9	0.0	91.9	183.8	270.1	316.5	367.6	367.6
	1.35 x SIDL1	100.4	100.4	87.7	87.7	71.6	43.3	14.2	14.2	43.3	71.6	87.7	100.4	100.4
	1.75 x SIDL2	115.5	110.5	99.5	95.2	70.4	40.2	9.5	21.1	51.8	81.1	99.5	110.5	115.5
	1.5 LL	625.7	554.6	554.6	348.0	303.0	67.7	145.9	182.1	229.7	315.4	418.1	462.6	483.2
	TOTAL V_u	1535.3	1459.2	1319.2	1013.9	773.7	315.5	169.7	381.8	653.5	951.1	1182.7	1367.2	1392.8
BENDING MOMENTS (kNm) CORRESSPONDING 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	-1.9	562.5	1002.4	1634.9	2044.1	2180.5	2044.1	1634.9	1002.4	562.5	-1.9	0.0
	1.35 x SIDL1	21.3	144.4	275.0	275.0	478.0	606.0	647.4	605.1	477.5	278.4	143.4	144.4	21.3
	1.75 x SIDL2	0.4	186.9	187.0	333.0	546.0	682.9	728.4	682.9	546.3	334.9	187.0	186.9	0.4
	1.5 LL	72.1	723.5	896.6	1521.5	2358.6	2703.2	2513.9	2019.0	1350.1	775.8	775.4	339.6	19.8

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	93.8	1050.5	2403.0	3969.1	6353.6	7694.8	7836.4	7009.7	5344.7	3228.7	2150.1	666.6	41.5

COMPOSITE SECTION PROPERTIES

Sectional area, A	m ²	1.798	1.798	1.563	1.350	1.350	1.350	1.350	1.350	1.350	1.350	1.563	1.798	1.798
Moment of Inertia, Iz	m ⁴	0.582	0.582	0.534	0.491	0.491	0.491	0.491	0.491	0.491	0.491	0.534	0.582	0.582
C.G. of section from top, Yt	m	0.682	0.682	0.638	0.587	0.587	0.587	0.587	0.587	0.587	0.587	0.638	0.682	0.682
C.G. of section from bottom, Yb	m	1.138	1.138	1.182	1.233	1.233	1.233	1.233	1.233	1.233	1.233	1.182	1.138	1.138
Sectional modulus at top, Zt	m ³	0.853	0.853	0.837	0.836	0.836	0.836	0.836	0.836	0.836	0.836	0.837	0.853	0.853
Sectional modulus at bottom, Zb	m ³	0.512	0.512	0.452	0.398	0.398	0.398	0.398	0.398	0.398	0.398	0.452	0.512	0.512
Effective width of Deck slab	m	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850
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

$$C = T$$

$$\text{Assume Depth of Neutral Axis, } X = 155.968 \text{ mm}$$

$$\text{Depth of Rectangular portion} = \frac{3}{7} X = 66.844 \text{ mm}$$

$$\text{Depth of Parabolic Portion} = \frac{4}{7} X = 89.125 \text{ mm}$$

$$\begin{aligned} C &= 7222.01 \text{ kN} \\ T &= 7222.01 \text{ kN} \\ C - T &= 0.00 \end{aligned}$$

Material Safety Factor:

ym for steel
ym for concrete

$$\begin{aligned} &= 1.15 \\ &= 1.5 \end{aligned}$$

$$\begin{aligned} E_p &= 1.95E+05 \text{ N/mm}^2 \\ E_s &= 2.00E+05 \text{ N/mm}^2 \\ E_{cm} &= 3.40E+04 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Width of deck slab} &= bs = 2850 \text{ mm} \\ \text{Thickness of Deck slab} &= ds = 220 \text{ mm} \\ \text{width of top flange of girder} &= w1 = 1000 \text{ mm} \end{aligned}$$

Material Stress

$$\begin{aligned} f_{ck} &= 45 \text{ N/mm}^2 & f_{cd} &= 20.07 \text{ N/mm}^2 \\ f_{yk} &= 500 \text{ N/mm}^2 & f_{yd} &= 434.783 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} \epsilon_{cu2} &= 0.0035 \\ \epsilon_{c2} &= 0.002 \end{aligned}$$

$$\begin{aligned} &\text{As per clause 7.9.5 of IRC:112-2011} \\ &\text{Partial safety factor for prestressing force } \gamma_p = 1.00 \end{aligned}$$

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	427	409	429	464	481	495	496
Final prestressing force = P1 . γ_p	kN	2255.78	2256.79	2271.89	2290.27	2327.94	2363.08	2381.54	2399.41	2379.36	2344.84	2327.61	2313.62	2312.71
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	690.959	689.344	663.610	631.885	587.455	553.666	528.032	504.177	499.551	521.339	550.903	575.026	576.513
Final prestressing force = P2 . γ_p	kN	3803.161	3804.776	3830.510	3862.235	3906.665	3940.454	3966.088	3989.943	3994.569	3972.781	3943.217	3919.094	3917.607
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													
Pd = P1... γ_p + P2... γ_p	kN	6058.943	6061.564	6102.396	6152.509	6234.601	6303.535	6347.627	6389.357	6373.930	6317.618	6270.831	6232.717	6230.317
eff. C.g. of cables from soffit (e_{ef})	m	0.695	0.675	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	1.145	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 = $0.0035 * (X - d_s) / X = -0.0014$

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 = $20.07 \times 2850.00 \times 66.84 \times 0.001 = 3823.416 \text{ kN}$

Comp. Force due to upper part of Parabolic portion C2 = $(0.00 \times 2850.000 \times 89.125 + 0.667 \times 20.1 \times 2850.000 \times 89.125) \times 0.001 = 3398.59 \text{ kN}$

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 = C1 + C2 + C3 = 7222.01 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 = $(P1/A_{cable}) / E_p$		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 = $(P2/A_{cable}) / E_p$		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 = $\epsilon_2 = 0.0035 (d-X) / X$		0.022	0.022	0.026	0.028	0.031	0.033	0.033	0.033	0.031	0.028	0.026	0.022	0.022
Minimum strain in tendons = $\epsilon_1 + \epsilon_2$		0.034	0.034	0.038	0.041	0.044	0.045	0.045	0.045	0.044	0.041	0.038	0.035	0.034
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	7656.3	7804.2	8981.1	9806.2	10710.5	11127.5	11150.6	11125.9	10712.3	9814.1	8990.7	7815.5	7667.8
M_u		102.8	975.3	2740.2	3681.5	6356.2	8152.4	8756.0	8053.5	6189.4	3667.5	1991.1	947.5	99.7
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	168.954	168.954	146.180	104.122	62.127	32.446	19.666	18.711	38.459	52.489	83.116	110.162	110.162
Area of composite section , A_k	m ²	1.798	1.798	1.563	1.350	1.350	1.350	1.350	1.350	1.350	1.350	1.563	1.798	1.798
perimeter of composite section , u_k	m	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140	6.140
$t_{ef,i}$	m	0.293	0.293	0.255	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.255	0.293	0.293
Shear stress due to pure Torsion , $\tau_{t,i} = T_{Ed} / (2 \cdot A_k \cdot t_{ef,i})$	N/mm ²	0.160	0.160	0.184	0.175	0.105	0.055	0.033	0.032	0.065	0.088	0.104	0.105	0.105
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	32.88	32.88	22.80	10.60	6.33	3.30	2.00	1.91	3.92	5.35	12.96	21.44	21.44
Mean compr. stress at c.g. sec. $\sigma_{cp} = N_{Ed} / A_c \leq 0.2 f_{cd}$	N/mm ²	3.369	3.371	3.904	4.014	4.014	4.014	4.014	4.014	4.014	4.014	4.012	3.466	3.465
Coefficient α_{cw} (eq.10.9 of IRC:112-2011)		1.168	1.168	1.195	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.173	1.173
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	4366.42	4366.69	3374.47	2529.16	2529.16	2529.16	2529.16	2529.16	2529.16	2529.16	3389.63	4384.42	4384.17
Maximum shear capacity $V_{Rd,max}$	kN	3598.82	3668.56	2544.42	1353.17	1477.97	1535.51	1538.70	1535.29	1478.21	1354.27	2558.59	3688.77	3618.87
$T_{Ed}/T_{Rd,max} + V_{Ed}/V_{Rd,max} \leq 1.0$		0.048	0.048	0.052	0.049	0.029	0.015	0.009	0.009	0.018	0.025	0.030	0.031	0.031
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	1658.58	1658.58	1650.87	1361.36	812.29	424.22	257.13	244.64	502.85	686.27	938.66	1081.43	1081.43

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, $V_{Ed} = V_U + V_{Ed,I}$	kN	1568.2	1492.1	1342.0	1024.5	780.0	318.8	171.7	383.8	657.4	956.4	1195.6	1388.7	1414.3
Coexisting ULS Moment, $M_{Ed} = M_U$	kNm	93.8	1050.5	2403.0	3969.1	6353.6	7694.8	7836.4	7009.7	5344.7	3228.7	2150.1	666.6	41.5
Final Prestressing Force, $N_{Ed} = P_d$	kN	6059	6062	6102	6153	6235	6304	6348	6389	6374	6318	6271	6233	6230
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	1.145	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	634.03	643.68	574.05	378.41	405.75	418.29	418.98	418.24	405.81	378.66	584.86	655.87	646.04
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	675.61	686.96	591.15	366.74	396.61	410.36	411.12	410.31	396.67	367.00	602.03	699.27	687.75
Max ($V_{Rd,c}$, min $V_{Rd,c}$)	kN	675.61	686.96	591.15	378.41	405.75	418.29	418.98	418.24	405.81	378.66	602.03	699.27	687.75
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.	NO REQ.	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 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	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	200 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	200 c/c	200 c/c	200 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	544.86	545.36	478.25	351.90	180.10	42.94	0.00	44.35	188.72	371.01	505.53	577.45
Net Shear Force = $V_{Rd,s} = V_{Ed} - V_{pd}$	kN	1023.3	946.8	863.7	672.6	599.9	275.9	171.7	339.4	468.7	585.4	690.1	811.2
Area of shear Reinf. A_{sw}	mm ²	226	226	226	226	226	226	226	226	226	226	226	226
Coefficient , α_{cw}		1.168	1.168	1.195	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.173
Lever arm , $Z = d - 0.416 X$	m	1.060	1.081	1.244	1.358	1.483	1.541	1.544	1.541	1.483	1.359	1.245	1.082
$1 \geq \cot \theta \geq 2.5$		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	3598.82	3668.56	2544.42	1353.17	1477.97	1535.51	1538.70	1535.29	1478.21	1354.27	2558.59	3688.77
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	234	258	326	457	559	1263	2034	1027	716	525	408	302
Check for Provided Spacing		SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE	SAFE

Minimum Shear Reinforcement

Min Shear Reinf. ratio , $\rho_{min} = 0.072 f_{ck}^{0.5} / f_{yk}$	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 , $\rho_w = A_{sw} / (S \cdot bw \cdot \sin \alpha)$	Refer Eq. 16.4 of IRC:112-2011	0.0016	0.0016	0.0027	0.0057	0.0057	0.0057	0.0057	0.0057	0.0057	0.0057	0.0027	0.0016	0.0016
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	1.138	1.138	1.182	1.233	1.233	1.233	1.233	1.233	1.233	1.233	1.182	1.138	1.138
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	0.582	0.582	0.534	0.491	0.491	0.491	0.491	0.491	0.491	0.491	0.534	0.582	0.582
First moment of area above and about N.A. of comp. Sect. ($A_y p$)	m ³	0.096	0.096	0.078	0.057	0.057	0.057	0.057	0.057	0.057	0.057	0.078	0.096	0.096

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	0.455	0.455	0.409	0.363	0.363	0.363	0.363	0.363	0.363	0.409	0.455	0.455
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	544.86	545.36	478.25	351.90	180.10	42.94	0.00	44.35	188.72	371.01	505.53	576.97
Design shear on precast sec. Before composite, V_{Ed1}	kN	693.7	693.7	577.4	483.0	328.7	164.3	0.0	164.3	328.7	483.0	577.4	693.7
Design shear on precast sec. after composite, V_{Ed2}	kN	841.6	765.6	741.8	530.9	445.0	151.2	169.7	217.5	324.8	468.1	605.3	699.2
Design Moment on precast sec. Before composite, M_{Ed1}	kNm	0.0	-4.3	1044.3	1839.6	2970.9	3702.8	3946.7	3702.8	2970.9	1839.6	1044.3	-4.3
Design Moment on precast sec. after composite, M_{Ed2}	kNm	102.8	979.6	1695.9	1841.9	3385.3	4449.7	4809.3	4350.7	3218.5	1827.9	946.8	99.7

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	-5.39	-3.87	-0.60	0.49	2.98	3.40
at bottom of precast girder fb.gir	N/mm ²	8.67	9.12	14.90	20.52	24.19	26.06	26.35	26.47	24.81	21.16	15.38	9.44	8.97
Stress due to prestress at centroidal axis, fcp	N/mm ²	4.88	4.71	4.27	4.29	2.65	1.89	1.86	1.92	2.71	4.39	4.38	4.84	5.01

Stress due to Moment M_{Ed1}

at top of precast girder ft.gir	N/mm ²	0.000	-0.013	3.356	6.481	10.467	13.045	13.905	13.045	10.467	6.481	3.356	-0.013	0.000
at bottom of precast girder fb.gir	N/mm ²	0.000	0.014	-3.607	-6.792	-10.969	-13.671	-14.571	-13.671	-10.969	-6.792	-3.607	0.014	0.000
Stress due to prestress at centroidal axis, fcp	N/mm ²	0.000	-0.005	1.538	3.434	5.545	6.911	7.367	6.911	5.545	3.434	1.538	-0.005	0.000

Stress due to Moment M_{Ed2}

at top of precast girder ft.gir	N/mm ²	0.082	0.778	1.326	1.378	2.533	3.329	3.598	3.255	2.408	1.368	0.741	0.756	0.079
at bottom of precast girder fb.gir	N/mm ²	-0.201	-1.915	-3.755	-4.623	-8.498	-11.170	-12.072	-10.921	-8.079	-4.589	-2.096	-1.861	-0.195
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	3.42	3.69	5.19	7.31	9.23	11.06	12.06	10.91	9.00	7.25	4.59	3.72	3.48
at bottom of precast girder fb.gir	N/mm ²	8.46	7.22	7.53	9.11	4.72	1.22	-0.29	1.88	5.76	9.78	9.67	7.59	8.78
Stress due to prestress at centroidal axis, fcp	N/mm ²	4.88	4.71	5.80	7.72	8.19	8.80	9.22	8.83	8.26	7.83	5.91	4.84	5.01

Check for Tension crack in the section

Tensile stress in the section	N/mm ²	3.42	3.69	5.19	7.31	4.72	1.22	-0.29	1.88	5.76	7.25	4.59	3.72	3.48

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	Uncracked	Uncracked	Uncracked	Uncracked	Uncracked	Uncracked	Uncracked	Uncracked

check at composite section centroid

Direct stress at centroid σ_{c1}	N/mm ²	4.88	4.71	5.80	7.72	8.19	8.80	9.22	8.83	8.26	7.83	5.91	4.84	5.01
Shear stress due to VEd1 , $\tau_{Ed1} = V_{Ed1}/(I_p b) \cdot (Ay)p$	N/mm ²	0.36	0.36	0.45	0.62	0.42	0.21	0.00	0.21	0.42	0.62	0.45	0.36	0.36
Shear stress due to VEd2 , $\tau_{Ed2} = V_{Ed2}/(I_c b) \cdot (Ay)c$	N/mm ²	0.94	0.85	1.38	1.96	1.64	0.56	0.63	0.80	1.20	1.73	1.12	0.75	0.78
Total shear stress $\tau_{Ed} = \tau_{Ed1} + \tau_{Ed2}$	N/mm ²	1.30	1.21	1.83	2.58	2.07	0.77	0.63	1.02	1.62	2.35	1.58	1.11	1.14
From Mohrs Circle Centre $\sigma_{co} = \sigma_{c1}/2$	N/mm ²	2.44	2.35	2.90	3.86	4.10	4.40	4.61	4.42	4.13	3.91	2.96	2.42	2.51
$\text{sqrt}(\tau_{Ed}^2 + (\sigma_{c1} - \sigma_{co})^2)$ radius σ_r	N/mm ²	2.76	2.65	3.43	4.64	4.59	4.47	4.65	4.53	4.44	4.57	3.35	2.66	2.75
Larger Principle stress $\sigma_3 = \text{min of } (0.6 f_{ck}, \sigma_{co} + \sigma_r)$	N/mm ²	5.20	5.00	6.33	8.50	8.69	8.87	9.27	8.95	8.56	8.48	6.31	5.08	5.26
smaller principle stress $\sigma_1 = \sigma_{co} - \sigma_r$	N/mm ²	-0.32	-0.30	-0.53	-0.78	-0.49	-0.07	-0.04	-0.12	-0.31	-0.65	-0.39	-0.24	-0.25
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	2.04	1.95	1.94	1.93	1.92	1.93	1.94	1.95	2.04	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 Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd	Not Reqd

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	1023.30	946.75	863.72	672.61	599.94	275.88	171.70	339.40	468.65	585.43	690.11	811.24	837.31
Effective width of flange at interface, bf	m	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850	2.850
Thickness of slab at interface, hf	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	1.060	1.081	1.244	1.358	1.483	1.541	1.544	1.541	1.483	1.359	1.245	1.082	1.062
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	0.97	0.88	0.69	0.50	0.40	0.18	0.11	0.22	0.32	0.43	0.55	0.75	0.79

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} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK	< v_{Edi} OK

Equating the $v_{Edi} = v_{Rdi}$ to get interface reinf. ρ_{reqd}		0.004	0.003	0.003	0.002	0.002	0.001	0.000	0.001	0.001	0.002	0.002	0.003	0.003
$\rho_{min} = 0.15\%$ 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	0.003	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.003
Spacing required, S_{reqd}		217	239	302	424	292	302	302	302	302	487	378	280	266
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	210 c/c	230 c/c	300 c/c	420 c/c	290 c/c	300 c/c	300 c/c	300 c/c	300 c/c	290 c/c	420 c/c	300 c/c	210 c/c
Area of shear connectors, A_s	mm ²	3830	3497	2681	1915	1560	1508	1508	1508	1560	1915	2681	3497	3830
$\rho_{provided} = A_s / A_i$		0.004	0.003	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.003	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	975.3	975.3	2740.2	3681.5	6356.2	8152.4	8756.0	8053.5	6189.4	3667.5	1991.1	947.5	947.5
Lever arm of composite section, z	m	1.1	1.1	1.2	1.4	1.5	1.5	1.5	1.5	1.5	1.4	1.2	1.1	1.1
Design force along height, F_d	kN	920.0	902.5	2203.5	2711.3	4285.9	5291.1	5671.1	5227.6	4172.7	2698.9	1599.4	875.5	892.4
Change in Force along length Δx , ΔF_d	kN	-17.4	1300.9	507.8	1574.6	1005.2	380.0	380.0	443.4	1054.9	1473.8	1099.5	723.9	-16.9
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	-0.21	1.92	0.77	1.20	0.72	0.27	0.27	0.32	0.76	1.12	1.66	1.07	-0.20
$\cot \theta_f$		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	1183	508	847	508	192	192	224	533	793	1099	658	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	191.3	445.4	266.9	445.4	1178.2	1178.2	1009.6	424.4	285.2	205.7	343.7	0.0
Spacing Provided	mm	0	190	440	260	440	1170	1170	1000	420	280	200	340	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

$$\begin{aligned} \text{Area of steel Required as per Clause 13.5.3} &= \frac{432.42}{250.00} \times 1000 \\ &= 1729.67 \text{ mm}^2 \end{aligned}$$

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

$$\begin{aligned} \text{Nos. of turns reqd.} &= \frac{1729.67}{201.06} \times 315.00 \\ &= 8.603 \end{aligned}$$

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 .

$$\begin{aligned} \text{Force} &= 0.04 P_k = 0.04 \times 2203.00 \\ &= 88.12 \text{ kN} \end{aligned}$$

$$\text{Area of steel Required as per Clause 13.5.3} = \frac{88.12}{250} \times 1000$$

$$\begin{aligned} &= 352.48 \text{ mm}^2 \\ \text{Adopt } &\mathbf{16 \text{ mm dia bar for spalling Reinforcement}} \end{aligned}$$

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 x 5672.3 x 40		=	2268.92	mm ²
Torsional Reinforcement required		=	1658.58	mm ²
Governing Reinforcement (max of above)		=	2268.92	mm ²
Adopt diameter of surface reinf.		=	12	mm
Nos. of Bars requd.		=	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	=	3
c/c distance of Long. Girder	=	2.8 m
Depth of Girder excluding slab	=	1.6 m
Depth of Girder with slab	=	1.82 m
Nos. of bearing under long. Girder	=	3
Spacing between brg.	=	2.8 m
Nos. of jack	=	2 Nos
Spacing between jacks	=	2.8 m
Thickness of end cross girder	=	0.4 m
Depth of cross girder	=	1.57 m
Length of End cross girder	=	5.6 m
Weight of end diaphragm	=	0.40 x 1.57 x 5.60 x 25 = 87.92 KN = 15.7 kN/m

Girder Reaction :

Support reaction under each girder	Unit	OG1	IG1	OG2			
Self-Weight of girder	KN	251.81	251.81	251.81			
Deck slab weight	KN	280.30	275.38	280.30			
Total DL	KN	532.11	527.19	532.11			
from staad pro							
SIDL 1 (except surfacing)	KN	130.16	64.70	130.16			
SIDL2 (weight of surfacing)	KN	72.31	70.54	72.31			
Foothpath	KN	0.00	0.00	0.00			
CWLL	1-70R	328.65	472.67	-2.32	98.81	84.909	172.81
	1-CLASSA	323.763	317.893	130.404	100.509	83.564	43.868

STAAD-PRO RESULTS :

Jacking Up Condition

	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (KNm)	Maximum Shear Force - Cantilever portion (KN)
Total DL	0	906	635
SIDL 1 (except surfacing)	0	195	83.2
SIDL2 (weight of surfacing)	0	101	69.7
CWLL	0	0	0

BEARING REACTION

		Bearing 1	Bearing 2	Bearing 3	
Total DL	KN	532.11	527.19	532.11	
SIDL 1 (except surfacing)	KN	130.16	64.70	130.16	
SIDL2 (weight of surfacing)	KN	72.31	70.54	72.31	
Foothpath	KN	0.00	0.00	0.00	
CWLL	KN	328.65	472.67	-2.32	
		1063.23	1135.10	732.26	

JACK REACTION

		JACK 1	JACK 2
Total DL	KN	1013	989
SIDL 1 (except surfacing)	KN	97.8	97.8
SIDL2 (weight of surfacing)	KN	107.58	107.58
Total		1218.38	1194.38

there fore , Capacity of each jack = 1462.056 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	0	906	635
SIDL 1 (except surfacing)	1.00	0	195	83.2
SIDL2 (weight of surfacing)	1.20	0	121.2	83.64
CWLL	1.00	0	0	0
Total SLS Moment		0	1222.2	801.84

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	0	906	635
SIDL 1 (except surfacing)	1.00	0	195	83.2
SIDL2 (weight of surfacing)	1.20	0	121.2	83.64
CWLL	0.75	0	0	0
Total SLS Moment		0	1222.2	801.84

SLS MOMENT - QUASI-PERMANENT COMBINATION FOR CRACK CHECK **Jacking Up Condition**

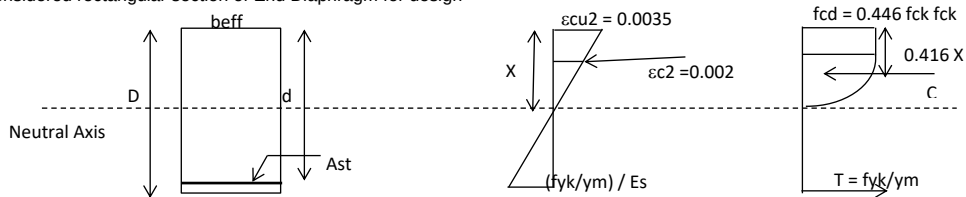
	Load Factor	Maximum Sagging Moment (kNm)	Maximum Hogging Moment (kNm)	Maximum Shear Force (kN)
Total DL	1.00	0	906	635
SIDL 1 (except surfacing)	1.00	0	195	83.2
SIDL2 (weight of surfacing)	1.20	0	121.2	83.64
CWLL	0.00	0	0	0
Total SLS Moment		0	1222.2	801.84

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	0	1223.1	857.25
SIDL 1 (except surfacing)	1.35	0	263.25	112.32
SIDL2 (weight of surfacing)	1.75	0	176.75	121.975
CWLL	1.5	0	0	0
Total ULS Moment		0	1663.1	1091.545

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:

ym for steel = 1.15
 ym for concrete = 1.5

Concrete Strain
 ϵ_{cu2} = 0.0035
 ϵ_{c2} = 0.002

Material Stress

fck = 45 N/mm²
 fyk = 500 N/mm²
 fctm = 3.3 N/mm²

fcd = 20.07 N/mm²
 fyd = 434.78 N/mm²

DESIGN OF SECTION FOR HOGGING MOMENT

Assume	Depth of Neutral Axis , X	=	242.334 mm

Compression Force due to rectangular portion C1 = 0.446 fck . 3/7X . beff = 833.77 KN
 Comp. Force due to upper part of Parabolic portion C2 = 2/3 . 0.446 fck . 4/7X . beff = 741.13 KN
 Total Compressive Force = C = C1+ C2 = 1574.894 KN eq 1
 Maximum Spacing between jack , L = 2.80 m
 L/D = 1.78 < 2.5 **DEEP BEAM**
 Provide Reinforcement in = 3 layers

Provide	4 Nos of	25 mm dia bars	(= 1963.495 mm ²)	in 1st Layer
Provide	4 Nos of	20 mm dia bars	(= 1256.637 mm ²)	in 2nd Layer
Provide	2 Nos of	16 mm dia bars	(= 402.124 mm ²)	in 3rd Layer

Total Area of steel provided , Ast = 3622.256 mm² **OK**
 Minimum area of Long. Steel reqd. , Ast.min = 1185.465 mm²
 Percentage of steel provided Pt = 0.577 %
 Spacing between reinf. c/c = 67.8 mm

Effective cover to bottom reinf. = 112 mm
 Effective depth provided dpro = 1458.00 mm

Total Tensile Force T = fyk/ym x Ast = 1574.894 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 . d² = 1663.1 kNm
 effective depth reqd. dreq = 743.815 mm
 Provided effective depth dpro = 1458.000 mm **OK**

Check that steel will yield

conc. Strain at level of tensile steel $0.0035 / x \cdot (d-x) = 0.0176$
 Yield strain of tensile reinf. $= 0.002 < 0.0176$ **Steel Yield**

Moment Capacity of section $M_R = T \cdot (d - 0.412 X) = 2138.956 \text{ kNm} > 1663.1 \text{ SAFE}$

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, $A_{ct} = 486266.399 \text{ mm}^2$
 $\sigma_{ct} = f_{yk} = 500 \text{ N/mm}^2$

$A_{s,min} = \text{min. Area of reinf. Within tensile zone} = 1185.465 \text{ mm}^2$

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. $A_{st,min} = 1185.465 \text{ mm}^2$

DESIGN OF SECTION FOR SAGGING (SPAN) MOMENT

Assume **Depth of Neutral Axis, X** $= 131.361 \text{ mm}$

Compression Force due to rectangular portion C1 $= 0.446 f_{ck} \cdot 3/7 X \cdot b \cdot e_{eff} = 451.955 \text{ KN}$
 Comp. Force due to upper part of Parabolic portion C2 $= 2/3 \cdot 0.446 f_{cd} \cdot 4/7 X \cdot b \cdot e_{eff} = 401.738 \text{ KN}$
 Total Compressive Force $= C = C1 + C2 = 853.694 \text{ KN}$ eq 1

Provide Reinforcement in $= 1$ layers

Provide	4 Nos of	25 mm dia bars	(= 1963.50 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, $A_{st} = 1963.50 \text{ mm}^2$ **OK**
 Minimum area of Long. Steel req'd., $A_{st,min} = 1185.46 \text{ mm}^2$
 Percentage of steel provided $P_t = 0.313 \%$
 Spacing between reinf. $c/c = 87.8 \text{ mm}$

Effective cover to bottom reinf. $= 89.5 \text{ mm}$
 Effective depth provided $d_{pro} = 1480.50 \text{ mm}$

Total Tensile Force $T = f_{yk} / \gamma_m \times A_{st} = 853.694 \text{ KN}$ 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 $= 0.167 f_{ck} \cdot b \cdot d^2 = 0 \text{ kNm}$
 effective depth req'd. $d_{req} = 0.00 \text{ mm}$
 Provided effective depth $d_{pro} = 1480.50 \text{ mm}$ **OK**

Check that steel will yield

conc. Strain at level of tensile steel $0.0035 / x \cdot (d-x) = 0.0359$
 Yield strain of tensile reinf. $= 0.002 < 0.0359$ **Steel Yield**

Moment Capacity of section $M_R = T \cdot (d - 0.412 X) = 1217.691 \text{ kNm} > 0 \text{ SAFE}$

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	1091.545
Width of section	b	mm	400
Effective depth	d	mm	1458.00
f_{ck}		N/mm ²	45.00
$K = 1 + \text{SQRT} (200 / d) \leq 2.0$			1.370
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	3622
$\rho_1 = A_{sl} / (b_w \cdot d) \leq 0.02$			0.006
steel ratio limited to 0.02			OK
$V_{Rd,c}$		KN	267.40
v_{min}		N/mm ²	0.33

min $V_{Rd,c}$	KN	194.56
Max ($V_{Rd,c}$, min $V_{Rd,c}$)	KN	267.40
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	1091.5
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	1312.20
$1 \geq \cot \theta \geq 2.5$		2.5
θ = smallest angle before concrete crushing starts hence θ	deg	22
$V_{Rd,max}$	KN	2179.52
Section check for shear capacity , $V_{Ed} \leq V_{Rd,max}$		SAFE
Reqd spacing of shear Reinf. S	mm	272
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	1094
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, $E_{eff} = E_{cm}$	=	34000	N/mm ²
Modulus of elasticity of steel, E_s	=	200000	N/mm ²
Modular Ratio (m) = E_s / E_{eff}	=	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. $\phi = \frac{E_{cm}}{(1 + \phi)} = 2.31$
 $E_{eff} = \frac{E_{cm}}{(1 + \phi)} = 1.03E+04$

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	1222.200	0.000	1222.200	0.000	1222.200	0.000	
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	98.00	75.50	98.00	75.50	98.00	75.50	
Effective depth, d	mm	1472.00	1494.50	1472.00	1494.50	1472.00	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	3622.26	1963.50	3622.26	1963.50	3622.26	1963.50	
dc		346.31	266.32	565.58	447.53	346.31	266.32	
Cracked Second Moment of area, I_{NA}	mm ⁴	5.53E+09	3.39E+09	4.21E+09	2.77E+09	5.53E+09	3.39E+09	
section modulus, $Z_t = I_{NA} / d_c$	mm ³	1.60E+07	1.27E+07	7.45E+06	6.18E+06	1.60E+07	1.27E+07	
section modulus, $Z_b = I_{NA} / (d-dc)$	mm ³	4.91E+06	2.76E+06	4.65E+06	2.64E+06	4.91E+06	2.76E+06	
Maximum compressive stress in concrete = $M/Z_t \times E_{eff} / E_s$	N/mm ²	13.01	0.00	8.42	0.00	13.01	0.00	< 21.6 SAFE
Maximum Tensile stress in concrete = $M/Z_b \times E_{eff} / E_s$	N/mm ²	42.28	0.00	13.49	0.00	42.28	0.00	
Maximum Tensile stress in steel = M/Z_b	N/mm ²	248.73	0.00	262.89	0.00	248.73	0.00	< 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	490	377.5
Spacing provided	mm	67.8	87.8
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	1472.00	1494.50
Depth of Neutral axis, $y_t=dc$	mm	346.31	266.32
$h_{c,eff} = \text{Min of } 2.5 (D - d), D - dc/3, D/2$	mm	245.00	188.75
$A_{c,eff} = b * h_{c,eff}$	mm ²	98000	75500
$\rho_{p,eff} = A_s / A_{c,eff}$		0.0370	0.0260
$S_{r \max}$	mm	250.98	299.42
$(\epsilon_{sm} - \epsilon_{cm})$	$= \frac{\sigma_{sc} - k_1 f_{t,eff} (1 + \alpha_e \rho_{p,eff})}{E_s}$		
Stress in tension steel, σ_{sc} (in Quasi-Permanent Case)	N/mm²	248.73	0.00
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	-0.0004
Crack Width, W_k	mm	0.24	-0.11
Limited Crack width	mm	0.30	0.30
Check for Crack width		< 0.3 mm SAFE	SAFE

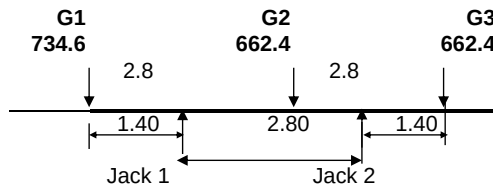
DESIGN OF DIAPHRAGM-DEEP BEAM DESIGN**(Jacking condition)**

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)



$$\begin{aligned} \text{No of Jacks} &= 2 \\ \text{Overall Depth of Diaphragm} &= 1.57 \text{ m} \\ \text{Width of diaphragm} &= 0.4 \text{ m} \end{aligned}$$

SUMMARY OF GIRDER REACTIONS FROM STAAD OUTPUT

SUPPORT NO	DL (kN)	SIDL (kN)	TOTAL REACTION IN KN	LOAD APPLIED IN STAAD (Say) IN KN
18 G1	532.11	202.47	734.6	734.6
23 G2	527.19	135.24	662.4	662.4
48 G3	532.11	202.47	662.4	662.4

from STAAD output

$$\begin{aligned} \text{Maximum Sagging Moments} &= 305.55 \text{ KN-M} \quad \text{TAKEN 25\% OF HOGGING MOMENT} \\ \text{Maximum Hogging Moments} &= 1222.2 \text{ KN-M} \\ \text{Maximum Shear} &= 801.84 \text{ KN} \end{aligned}$$

DESIGN

$$L/D = 1.783 < 2.5 \text{ Design as deep beam as per IS : 456 -2000}$$

$$\begin{aligned} \text{Liver arm} &= 0.2 (1 + 1.5D) \quad \text{As per clause 29.2 of IS : 456 -2000} \\ &= 0.2 (2.800 + 1.5 \times 1.57) = 1.031 \text{ m} \end{aligned}$$

$$\text{Maximum sagging B M} = 305.55 \text{ kN-m}$$

$$\begin{aligned} \text{Ast required} &= \frac{305.55 \times 1000000}{240 \times 1031.000} \\ &= 1235 \text{ mm}^2 \\ \text{Minimum steel} &= 0.2 b d \\ &= 0.2 / 100 \times 400 \times 1501.5 = 1201.2 \text{ mm}^2 \\ &= \frac{d}{1501.5} = \frac{1.57}{1501.5} = 0.04 - 0.0125 - 0.016 \end{aligned}$$

Provide	4 Nos of	25 mm dia bars	(= 1966.25 mm ²)	in 1st Layer
Provide	0 Nos of	20 mm dia bars	(= 0.00 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. 0.253 m from the tension face (soffit) of the girder ,

$$\text{Design hogging B M} = 1222.20 / 1 = 1222.20 \text{ kN-m}$$

$$\begin{aligned} I / D &= 2.800 / 1.57 = 1.783 > 1.0 \\ \text{Lever arm} &= 0.2 (1 + 1.5D) = 1.03 \quad \text{As per cl: 29.2 of IS:456-1978} \\ \text{Net liver arm} &= 1.031 \end{aligned}$$

$$\begin{aligned} \text{Ast required} &= \frac{1222.20 \times 1000000}{240 \times 1031} \\ &= 4939.4 \text{ mm}^2 \end{aligned}$$

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

$$0.5 (I / D - 0.5) = 0.64$$

$$\text{Ast (Required) for this zone} = 0.64 \times 4939 = 3170 \text{ mm}^2$$

Provide	4 Nos of	25 mm dia bars	(= 1966 mm ²)	in 1st layers
Provide	4 Nos of	20 mm dia bars	(= 1258 mm ²)	in 2st layers
Provide	2 Nos of	16 mm dia bars	(= 403 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.
$$= 4939 - \frac{3627 \text{ in } (' = 0.471 \text{ m })}{3170} = 1770 \text{ mm}^2$$

Provide	20 Nos	of	12 mm dia bars	(=	2265 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 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 = **1377.69 N/mm² OK**

CHECK FOR BONDING

Bonding stress = 3.00 N/mm² (IRC: 112-2011 Table 15.3)

Ld $\frac{\phi \sigma_s}{4\tau_{bd}}$ = 1458.05 mm (IRC: 112-2011 Cl 15.2.3.3)

**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 Thickness	=	220 mm
Total Width of Slab	=	8.5 m

CALCULATION OF DEAD LOAD & SUPERIMPOSED DEAD LOAD

1. Deck slab

Self weight of slab	=	5.500 kN/m ²

2. Wearing coat 50 thick + additional load

Load = 2.2 kN/m ²	with start and end offset of 0.5m

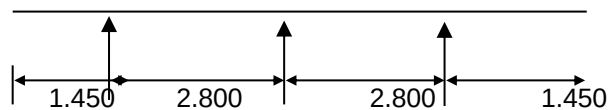
3. Crash barrier

load = 8.0 kN

4. PCC Profile Correction

Thk	=	0.000 m	at edge	0 kN/m ²
		0.000 m	at median	0 kN/m ²
load=	0 KN/m ²			

IDEALIZATION OF DECK SLAB



Load Factors (As per IRC:6-2017)

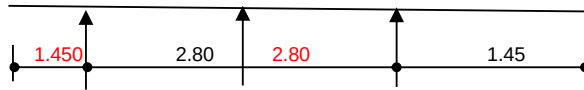
Table B.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

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 B.3 Partial Safety Factor For Verification of Serviceability Limit State

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 = 2.80 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

$$= 2.6$$

Average Thickness of Profile Correction = 0.000 m

Thickness of wearing coat = 65 mm

Thickness of deck slab = 220 mm

Effective depth of deck slab = 172 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

Thickness of Web of girder = 0.28 m

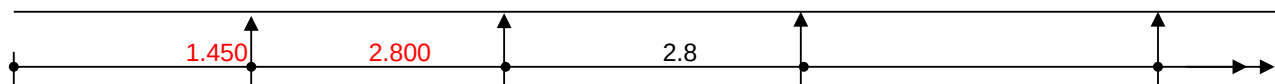
Top Flange width of girder = 1 m

Effective support length from centre of web = 0.3 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)

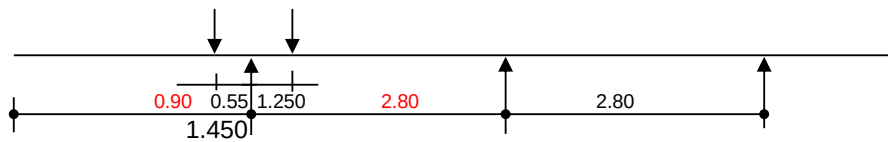


Length of dispersion along the span =

$$0.5 + 2 \times (0.065 + 0.172 + 0.000) = 0.974 \text{ m}$$

Load Nos.	Load (kN)	a (m)	b _{ef} (m)	b _{ef1} (m)	b _{eff2} (m)	Load (kN/m ²)
1	57	0.246	0.675	0.675	0.974	86.70
2	57	0.946	2.008	1.604	0.974	36.5

$$b_{ef1} = (b_{ef} + 1.2)/2, \text{ if } b_{ef} > 1.2$$



CLASS 70R LOADING

$$\text{Max. load on a wheel} = \frac{200}{4} = 50 \text{ kN}$$

$$\text{Max. tyre pressure} = 5.273 \text{ kg/cm}^2$$

$$\text{Contact width parallel to span} = 410 \text{ mm}$$

$$\text{Contact area} = \frac{5000}{5.273} = 948 \text{ cm}^2$$

$$= 94823 \text{ mm}^2$$

$$\text{Contact width perpendicular to span} = \frac{94823}{410} = 231 \text{ mm}$$

$$\text{Distance between two loads perpendicular to span} = 1.22 \text{ m}$$

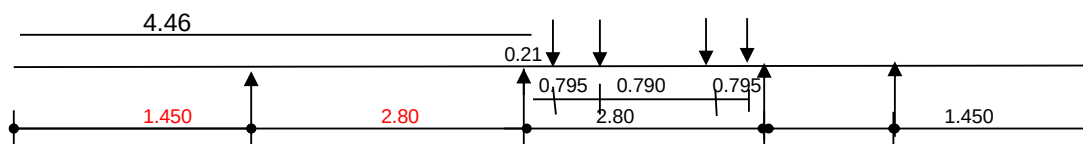
$$b1 = 0.231 + 0.065 \times 2 = 0.361 \text{ m}$$

Loading 1 M type (for maximum sagging moment)

$$\text{Length of dispersion along the span} = 0.884 \text{ m}$$

Load Nos.	Load (kN)	a (m)	bef (m)	bef1 (m)	bef2(m)	Load (kN/m ²)
1	50	0.210	0.866	0.866	0.840	68.7
2	50	1.005	2.036	1.628	0.793	38.7
3	50	1.005	2.036	1.628	0.793	38.7
4	50	0.21	0.866	0.866	0.840	68.7

$$bef1 = (bef + 1.22)/2, \text{ if } bef > 1.22$$

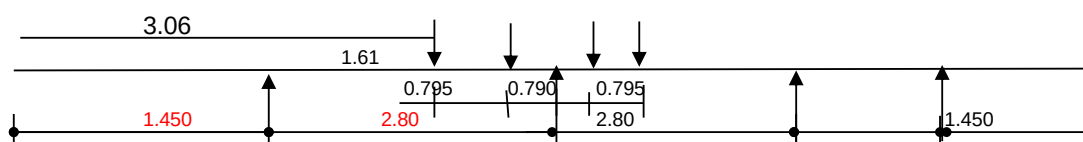


Loading 2 M type (Inner Support Hogging Moment)

$$\text{Length of dispersion along the span} = 0.884 \text{ m}$$

Load Nos.	Load (kN)	a (m)	bef (m)	bef1 (m)	bef2(m)	Load (kN/m ²)
1	50	1.190	2.140	1.680	0.840	35.4
2	50	0.395	1.243	1.232	0.793	51.2
3	50	0.395	1.243	1.232	0.793	51.2
4	50	1.190	2.140	1.680	0.840	35.4

$$bef1 = (bef + 1.22)/2, \text{ if } bef > 1.22$$



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 Moderate 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	4.912	1.040	1.180	8.825	5.112
Wearing Coat	0.223	1.040	0.832	0.990	0.154
Class A	12.542	0.000	0.000	46.537	0.000
Class 70R Wheel	0.000	0.971	32.800	0.000	47.468
SV Loading	0.000	0.000	0.000	0.000	0.000

shear reduction factor = 0.000

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	6.631	1.404	1.593	11.914	6.901
Wearing Coat	1.750	0.390	1.820	1.456	1.733	0.270
Class A	1.500	28.220	0.000	0.000	104.708	0.000
Class 70R Wheel	1.500	0.000	1.821	61.500	0.000	89.003
SV Loading	1.000	0.000	0.000	0.000	0.000	0.000
Total =		35.241	5.045	64.549	118.355	96.173

Support Reinforcement Calculation:

Thickness of slab = 220 mm at Interior span

Thickness of slab = 220 mm (at cantilever)

Clear Cover to outer steel = 40 mm

Maximum Diameter of Reinforcement = 16 mm

Effective Depth Provided (deff) = 172 mm

Design bending moment = 35.24 kNm/m

Mulim = $0.165 \times f_{ck} \times b \times d^2$ = 35.24 kNm/m (Equation derived based on IRC:112-2011)

Effective Depth of Cap Required (dreq) = $\text{SQRT} \left(\frac{35.24 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$

Effective Depth of Cap Required (dreq) = 78.117 mm

Total Depth Required (Dreq) = 126.12 mm

Total Depth Provided (Dprov) = 220.00 mm

OK

$R = M_{RD} / (b \times d^2)$ = 1.19

Design Moment of Resistance M_{RD} = $0.87 f_y A_{st} \times (d - (\lambda x_u / 2))$

35.24 = $0.87 f_y A_{st} \times (d - (\lambda x_u / 2))$

$$\begin{aligned}\eta &= 1 \\ f_{cd} &= 15.61 \text{ N/mm}^2 \\ f_y &= 500 \text{ N/mm}^2 \\ \lambda &= 0.8\end{aligned}$$

$$M_{RD} = 0.87 f_y A_{st} \left(d - \left(0.87 f_y A_{st} / (b \cdot n \cdot f_{cd} \cdot 2) \right) \right)$$

$$\text{Area of Steel Required, } A_{st_{red}} = 484.3841 \text{ mm}^2/\text{m}$$

$$A_{st_{req}} = 484.3841 \text{ mm}^2/\text{m}$$

Minimum Longitudinal Reinforcement :

$$A_{s, \text{Min}} = 0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \cdot d \quad \text{- Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

$$\text{Whichever is higher} \quad \text{OR} \quad = 0.0015 \times b \cdot d \quad \text{'-Refer Clause 16.9 of IRC:112-2011'}$$

$$b = 1000.00 \text{ mm}$$

$$d = 172.00 \text{ mm}$$

$$A_{st \text{ min}} = 258.00 \text{ mm}^2/\text{m}$$

$$\text{Governing Reinf. } A_{st} = 484.38 \text{ mm}^2/\text{m}$$

$$\text{Provide } 16 \text{ mm dia @ } 150 \text{ mm c/c} + 0 \text{ mm dia @ } 100 \text{ mm c/c}$$

$$\text{Area provided} = 1340.41 \text{ mm}^2/\text{m} > 484.38 \text{ mm}^2/\text{m} \quad \text{OK}$$

$$\text{Percentage of Steel (pt\%)} = 0.78 \%$$

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

$$S_{max} = 2h = 344.00$$

$$\text{OR} = 250.00 \text{ mm} \quad \text{whichever is max}$$

Provided Spacing is less than Smax, Hence OK

Moment of Resistance of Section corresponding to Provided Ast

$$M_{RD} = 0.87 f_y A_{st} \left(d - \left(0.87 f_y A_{st} / (b \cdot n \cdot f_{cd} \cdot 2) \right) \right)$$

$$= 89.40 \text{ kNm} > 35.24 \text{ kNm}$$

SAFE

Distribution reinforcement:

$$\text{Effective Depth in Long. Direction} = 159 \text{ mm}$$

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$$\frac{20.00}{100.00} \times 1340.41 = 268.0826 \text{ mm}^2/\text{m}$$

$$\text{Provide } 10 \text{ dia bar } 200 \text{ mm c/c at Trans direction in top face. (Providing } = 392.69908 \text{ mm}^2 \text{)}$$

Span Reinforcement Calculation:

$$\text{Clear Cover to outer steel} = 40 \text{ mm}$$

$$\text{Maximum Diameter of Reinforcement} = 16 \text{ mm}$$

$$\text{Effective Depth Provided (deff)} = 172 \text{ mm}$$

$$\text{Design bending moment} = 64.55 \text{ kNm/m}$$

$$\text{Mult} = 0.165 \times f_{ck} \times b \times d^2 = 64.55 \text{ kNm/m}$$

$$\text{Effective Depth of Cap} = \text{SQRT} \left(\frac{64.55 \times 1000000}{0.165 \times 35.00 \times 1000} \right)$$

$$\text{Required (dreq)} = 105.7228 \text{ mm}$$

$$\text{Effective Depth of Cap Required (dreq)} = 105.7228 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 153.72 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 220.00 \text{ mm} \quad \text{OK}$$

$$R = \frac{M_u}{(b \cdot d^2)} = 2.18$$

Ast Required:

$$M_{RD} = 0.87f_y A_{st} * (d - (0.87f_y A_{st} / (b * n * f_{cd} * 2)))$$

$$\text{Area of Steel Required, } A_{st_{red}} = 920.674 \text{ mm}^2/\text{m}$$

$$A_{st_{req}} = 920.674 \text{ mm}^2/\text{m}$$

Minimum Longitudinal Reinforcement :

$$\text{As. Min} = 0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \cdot d \quad \text{- Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

Whichever is higher OR

$$= 0.0015 \times b \cdot d \quad \text{-Refer Clause 16.9 of IRC:112-2011'}$$

$$b = 1000.00 \text{ mm}$$

$$d = 172.00 \text{ mm}$$

$$A_{st \text{ min}} = 258.00 \text{ mm}^2/\text{m}$$

$$\text{Governing Reinf. Ast} = 920.67 \text{ mm}^2/\text{m}$$

Provide 16 mm dia @ 150 mm c/c + 0 mm dia @ 100 mm c/c

Area provided=	1340.41 mm ² /m	>	920.67 mm ² /m	OK
Percentage of Steel (pt%)	=	0.78	%	

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

$$S_{max} = 2h = 344.00 \text{ mm}$$

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

$$M_{RD} = 0.87f_y A_{st} * (d - (0.87f_y A_{st} / (b * n * f_{cd} * 2)))$$

$$= 89.40 \text{ kNm} > 64.55 \text{ kNm}$$

SAFE

Distribution reinforcement:

Effective Depth in Long. Direction = 165 mm

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$$\frac{20.00}{100.00} \times 1340.41 = 268.0825731 \text{ mm}^2/\text{m}$$

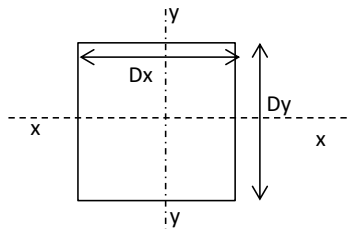
Provide 10 dia bar 200 direction in top face. (Providing = 392.69908 mm²)

Stress Check of Deck Slab

Design Forces in Rare Combination		Moments (kNm/m)		
Load Cases	Load Factors	Cantilever Portion	Intermediate Support	Intermediate Span
DL+SIDL excluding Wearing Coat	1.0	4.91	1.04	1.18
Wearing Coat	1.2	0.22	1.04	0.83
Class A	1.0	18.81	0.00	0.00
Class 70R Wheel	1.0	0.00	1.21	41.00
SV Loading	1.0	0.00	0.00	0.00
			3.29	43.01

Maximum Support moment (Mxx)	=	43.012	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 Igxx	=	8.87E+08	mm ⁴
Gross Moment of Inertia Igyy	=	1.83E+10	mm ⁴

Calculation of Modular Ratio:

Perimeter of section (u)	=	2440	mm
2 Ac/u	=	180.328	mm
For Dry atmospheric condition			
Creep coeff.	=	1.48	
Eceff	=	1.30E+04	
	=	$\frac{E_{cm}}{(1 + \phi)}$	

Modular Ratio (m)	=	Ec/ Eceff	=	15.35

Details of Reinforcements:

at Support	16 mm dia @ 150 mm c/c			
	0 mm dia @ 100 mm c/c			
Total Ast provided	=	1340.41	mm ² /m	
pt = Ast/(b*D)	=	0.0078		
effective cover	=	48	mm	
C.g. Of Tension steel from N.A. (y)	=	62.00	mm	
Area of concrete, Ac = (Ag- Ast)	=	218659.59	mm ²	
Transformed Area of Section AstT	=	239235.9473	mm ²	
Transformed M.O.I (Ixx)	=	8.87E+08		73942981.59
	=	9.61E+08	mm ⁴	
Section modulus (Zxx)	=	Ixx / (Dy/2)		
	=	8738875.59	mm ³	
Bending stress (M/ Zxx)	=	4.922	N/mm ²	
Tensile strength of concrete , fctm	=	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)	=	172.00	mm
Depth of neutral axis (yb)	=	k * dy	
where , k	=	$\sqrt{2 pt * m + (pt*m)^2} - pt* m$	
	=	0.3839	
Depth of Neutral axis from Extreme compression face(yb)	=	66.0357	mm
Depth of Neutral axis from Extreme Tension face(yt)	=	153.9643	mm

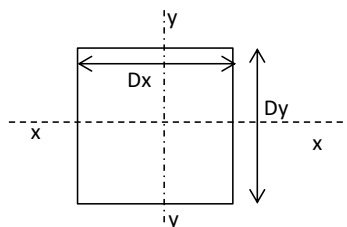
Depth of Neutral axis from c.g. Of tension steel (ys)	=	105.9643 mm		
Cracked Moment of Inertia (Ixxcr)	=	$Dx * (k * dy)^3 / 3 + m A_{st} * (dy - k * dy)^2$		
	=	3.2703E+08 mm ⁴		
Maximum compressive stress in concrete=	$M_{xx} / I_{xxcr} * y_b$	=	8.69 N/mm ²	< 16.8
Maximum Tensile stress in concrete=	$M_{xx} / I_{xxcr} * y_t$	=	20.25 N/mm ²	> 2.77
Maximum Tensile stress in steel=	$m * M_{xx} / I_{xxcr} * y_s$	=	213.94 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	4.912	1.04	1.18
Wearing Coat	1.20	0.223	1.04	0.832
Class A	0	0	0	0
Class 70R Wheel	0	0	0	0
SV Loading	0	0	0	0
			2.08	2.01

Maximum Support moment (M_{xx}) = 2.08 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	16 mm dia @ 150 mm c/c	
	0 mm dia @ 100 mm c/c	

Total Ast provided = 1340.41 mm²/m
 $p_t = A_{st}/(b \cdot D)$ = 0.0078
 effective cover = 48 mm
 C.g. Of Tension steel from N.A. (y) = 62.00 mm
 Area of concrete, A_c = (A_g - A_{st}) = 218659.59 mm²
 Transformed Area of Section A_{st}T Ac + m Ast = 239235.947 mm²
 Transformed M.O.I (I_{xx}) $I_{gxx} + (m-1) \cdot A_{st} \cdot y^2$ = 8.87E+08 + 73942981.59
 = 9.61E+08 mm⁴
 Section modulus (Z_{xx}) = $I_{xx} / (D_y/2)$
 = 8738875.59 mm³
 Bending stress (M/ Z_{xx}) = 0.238 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

STAAD ANALYSIS FOR DECK SLAB DESIGN

STAAD PLANE ANALYSIS FOR DESIGN OF SLAB (DL+SIDL+CWLL)

PAGE NO. 1

```

*****
*                               *
*   STAAD.Pro CONNECT Edition   *
*   Version  21.00.01.12        *
*   Proprietary Program of      *
*   Bentley Systems, Inc.       *
*   Date=    SEP 20, 2019       *
*   Time=    17:54:15           *
*                               *
* Licensed to: Rodic Consultants Pvt. LtdNew *
*****

```

1. STAAD SPACE

INPUT FILE: D:\Ranchi\Lalpura Chowk Flyover\with 8.5m deck width\Deck slab\staad\Deck Slab.STD

2. START JOB INFORMATION

3. ENGINEER DATE 8-MAY-17

4. END JOB INFORMATION

5. INPUT WIDTH 79

6. UNIT METER KN

7. JOINT COORDINATES

8. 1 0 0 0; 2 1.45 0 0; 3 4.25 0 0; 4 7.05 0 0; 5 8.5 0 0

9. MEMBER INCIDENCES

10. 1 1 2; 2 2 3; 3 3 4; 4 4 5

11. DEFINE MATERIAL START

12. ISOTROPIC CONCRETE

13. E 3.2E+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 4 PRIS YD 0.22 ZD 1

21. CONSTANTS

22. MATERIAL CONCRETE ALL

23. SUPPORTS

24. 2 TO 4 PINNED

25. *****

26. LOAD 1 SELF WEIGHT OF SLAB + CRASH BARRIER

27. SELFWEIGHT Y -1

28. MEMBER LOAD

29. 1 UNI GY -8 0.05 0.5

30. 4 UNI GY -8 0.95 1.45

31. *****

32. LOAD 2 WEARING COAT

33. MEMBER LOAD

34. 2 3 UNI GY -2.2

35. 1 UNI GY -2.2 0.5 1.45

36. 4 UNI GY -2.2 0 0.95

37. *****

38. LOAD 3 1 LANE CLASS-A
STAAD SPACE

-- PAGE NO. 2

39. MEMBER LOAD

40. 1 UNI GY -86.34 0.411 1.389

41. 2 UNI GY -36.33 0.761 1.739

43. *****

44. LOAD 4 CLASS-70R SAGGING

45. MEMBER LOAD

46. 2 UNI GY -68.59 2.58925 2.8

47. 3 UNI GY -68.59 0 0.63075

48. 3 UNI GY -38.75 0.60875 1.40125

49. 3 UNI GY -38.75 1.39875 2.19125

50. 3 UNI GY -68.59 2.16925 2.8

51. 4 UNI GY -68.59 0 0.21075

53. *****

54. LOAD 5 CLASS-70R HOGGING

55. MEMBER LOAD

56. 2 UNI GY -35.36 1.18925 2.03075

57. 2 UNI GY -51.22 2.00875 2.80125

58. 3 UNI GY -51.22 0.001 0.79125

59. 3 UNI GY -35.36 0.769 1.61075

61. *****

62. PERFORM ANALYSIS

PROBLEM STATISTICS

NUMBER OF JOINTS	5	NUMBER OF MEMBERS	4
NUMBER OF PLATES	0	NUMBER OF SOLIDS	0
NUMBER OF SURFACES	0	NUMBER OF SUPPORTS	3

Using 64-bit analysis engine.

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH= 1/ 1/ 9 DOF
TOTAL PRIMARY LOAD CASES = 5, TOTAL DEGREES OF FREEDOM = 21
TOTAL LOAD COMBINATION CASES = 0 SO FAR.
SIZE OF STIFFNESS MATRIX = 1 DOUBLE KILO-WORDS
REQRD/AVAIL. DISK SPACE = 12.0/ 390594.9 MB

***WARNING - INSTABILITY AT JOINT 5 DIRECTION = MX
PROBABLE CAUSE SINGULAR-ADDING WEAK SPRING
K-MATRIX DIAG= 2.5521797E+05 L-MATRIX DIAG= 2.9103830E-11 EQN NO 19
***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.

63. PRINT MEMBER FORCES LIST 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	11.57	0.00	0.00	0.00	-10.01
	2	1	0.00	-0.00	0.00	0.00	0.00	-0.00
		2	0.00	2.09	0.00	0.00	0.00	-0.99
	3	1	0.00	-0.00	0.00	0.00	0.00	-0.00
		2	0.00	84.44	0.00	0.00	0.00	-46.44
	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
2	1	2	0.00	11.17	0.00	0.00	0.00	10.01
		3	0.00	4.23	0.00	0.00	0.00	-0.28
	2	2	0.00	2.84	0.00	0.00	0.00	0.99
		3	0.00	3.32	0.00	0.00	0.00	-1.66
	3	2	0.00	37.31	0.00	0.00	0.00	46.44
		3	0.00	-1.78	0.00	0.00	0.00	2.95
	4	2	0.00	-7.79	0.00	0.00	0.00	0.00
		3	0.00	22.25	0.00	0.00	0.00	-23.34
	5	2	0.00	8.64	0.00	0.00	0.00	-0.00
		3	0.00	61.64	0.00	0.00	0.00	-27.25
3	1	3	0.00	4.02	0.00	0.00	0.00	0.28
		4	0.00	11.38	0.00	0.00	0.00	-10.58
	2	3	0.00	3.32	0.00	0.00	0.00	1.66
		4	0.00	2.84	0.00	0.00	0.00	-0.99
	3	3	0.00	-1.05	0.00	0.00	0.00	-2.95
		4	0.00	1.05	0.00	0.00	0.00	0.00
	4	3	0.00	81.76	0.00	0.00	0.00	23.34
		4	0.00	66.18	0.00	0.00	0.00	-1.52
	5	3	0.00	61.60	0.00	0.00	0.00	27.25
		4	0.00	8.64	0.00	0.00	0.00	0.00
4	1	4	0.00	11.97	0.00	0.00	0.00	10.58
		5	0.00	-0.00	0.00	0.00	0.00	-0.00
	2	4	0.00	2.09	0.00	0.00	0.00	0.99
		5	0.00	0.00	0.00	0.00	0.00	-0.00
	3	4	0.00	-0.00	0.00	0.00	0.00	-0.00
		5	0.00	0.00	0.00	0.00	0.00	0.00
	4	4	0.00	14.46	0.00	0.00	0.00	1.52

5	0.00	0.00	0.00	0.00	0.00	0.00
5 4	0.00	-0.00	0.00	0.00	0.00	-0.00
5	0.00	0.00	0.00	0.00	0.00	-0.00

STAAD SPACE -- PAGE NO. 5

***** END OF LATEST ANALYSIS RESULT *****

64. FINISH

Design of Projecting Deck Slab Beyond End Cross Girder As a Cantilever Slab

(This will be designed as cantilever slab)

Data Given

Thickness of Deck slab at edge	=	0.370 m
Thickness of profile correction	=	0.000 m
Length of Cantilever	=	0.425 m
Thickness of W.C.	=	0.065 m

Clear Cover	=	40 mm
Maximum dia of bar in slab	=	12 mm
Effective depth, d	=	0.318 m
Effective depth, d' = 0.9 d	=	0.286 m
a/d	=	1.336

Cantilever action

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 @ cant. face (kN-m/m)
1	Cantilever Slab seg.(1) + profile correction	1.350	0.425	0.370	25.000	3.931	0.213	1.128
2	Wt. of the Wearing Course (Wt. of W.C. 2.00 kN/m ²)	1.750	0.425	x	2.000	0.850	0.213	0.316
	Total					4.781		1.444

Moment in Cantilever Slab due to CWLL

Class-70R Bogie Axle placed symmetrically(M Type)

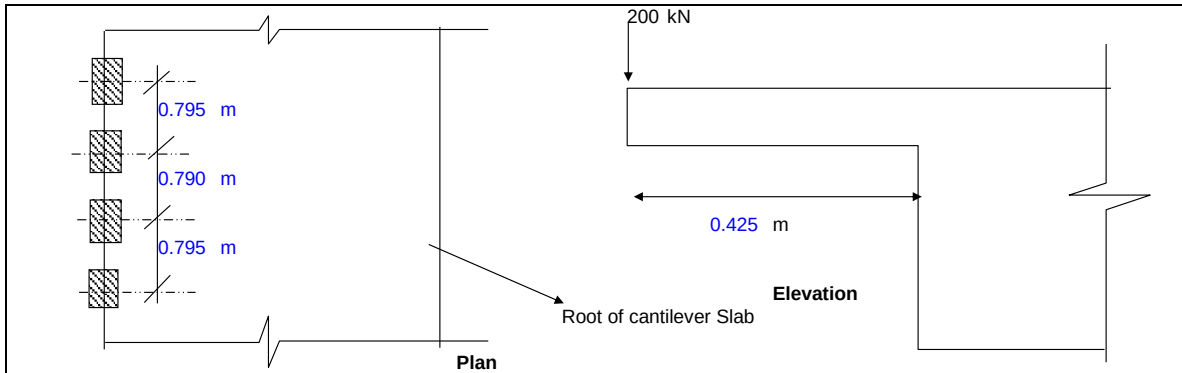
Considering 200.00 kN Axle passing through the section
Dis. of Nearest Axle for Cl.70R Bogie Axle = 1.220 m

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 $b_e = 1.2 a + b_1$

$b_1 = 0.360 + 2 \times 0.065 = 0.490$ m
Th. of W.C.

$a = 0.425$ m

$b_e = 1.200 \times 0.425 + 0.490 = 1$ m

Hence, effective width = $\frac{1}{2} + 0.795 + 0.790 + 0.795 + \frac{1}{2} = 3.38$ m

Impact Factor for Cl. 70R Wheeled Loading = 1.250
(for cantilever slab portion having length < 9m)

Considering no dispersion of Load along the Span

Intensity of the Load Live Load = $\frac{200.000}{3.380} = 59.172$ kN/m

Live load Factor = 1.500 -Refer Table 3.2 of IRC:6-2017

B.M. at the root of Cantilever due to Live Load = $1.250 \times 59.172 \times 0.425 \times 1.500 = 47.152$ kN-m/m

S.F. at the root of Cantilever due to Live Load = $1.250 \times 59.172 \times 1.500 = 110.947$ kN per m

$$\text{Total Ultimate B.M. at the root of Cantile} = 1.444 + 47.152 = \boxed{48.596 \text{ kN-m/m}}$$

Material Properties:

Grade of Concrete	=	M 35	
f _{ck}	=	35 N/mm ²	
f _{ctm}	=	2.77 N/mm ²	
E _c	=	29580.40 N/mm ²	(5000 x sqrt (f _{ck})
Grade of Steel	=	Fe 500	(HYSD Bars)
f _y or f _{yk}	=	500.00 N/mm ²	
f _{yd}	=	434.783 N/mm ²	(f _y /1.15)
E _s	=	200000 N/mm ²	
Clear cover to reinforcement	=	50 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)

$$\text{Mulim} = 0.165 \times f_{ck} \times b \times d^2 = 48.596 \text{ kNm/m}$$

$$\text{Effective Depth of Cap Required (dreq)} = \frac{\text{SQRT} \left(\frac{48.596}{0.165} \times \frac{1000000}{35 \times 1000} \right)}{1000}$$

$$\text{Effective Depth of Cap Required (dreq)} = 91.733 \text{ mm}$$

$$\text{Total Depth Required (Dreq)} = 159.733 \text{ mm}$$

$$\text{Total Depth Provided (Dprov)} = 370 \text{ mm}$$

$$\text{Deff provided} = 302 \text{ mm}$$

OK

$$R = \frac{M_u}{(b \times d^2)} = 0.533$$

Minimum Longitudinal Reinforcement in Pier Cap :

$$\text{As. Min} = 0.26 \times \frac{f_{ctm}}{f_{yk}} \times b \cdot d \text{ - Refer Eq. 16.5.1.1 \& 16.6.1.1 of IRC: 112-2011}$$

$$\text{OR} = 0.0015 \times b \cdot d \text{ -Refer Clause 16.9 of IRC:112-2011'}$$

Whichever is higher

$$b = 1000 \text{ mm}$$

$$d = 302 \text{ mm}$$

$$\text{Ast min} = 453 \text{ mm}^2$$

Area of Steel Required:

$$\text{Design Moment of Resistance } M_{RD} = 0.87 f_{yk} A_{st} \cdot (d - (\lambda x_u / 2))$$

$$\eta = 1$$

$$f_{cd} = 15.61 \text{ N/mm}^2$$

$$f_y = 500.00 \text{ N/mm}^2$$

$$\lambda = 0.8$$

$$M_{RD} = 0.87 f_{yk} A_{st} \cdot (d - (0.87 f_y A_{st} / (b \cdot \eta \cdot f_{cd} \cdot 2)))$$

$$\text{Area of Steel Required, Ast}_{red} = \boxed{437.14} \text{ mm}^2/\text{m}$$

$$\text{Ast}_{req} = 437.138 \text{ mm}^2/\text{m}$$

$$\text{Ast required} = \max(\text{Astmin}, \text{Astreq}) = 453.0000 \text{ mm}^2/\text{m}$$

Provide	12 dia @ spacing	200 +	10 dia @ spaci	200	=	958.19 mm ² /m

LOAD AND FORCES FOR DESIGN OF SPHERICAL BEARING

Seismic Coefficient Calculation

(As Per IRC:6-2017 , Clause 219)

Horizontal Seismic Force For Zone

II

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	=	0.1	
I	=	Importance factor	=	1.2	
R	=	Response Reduction factor	=	1.0	in Longitudinal direction
			=	1.0	In Transverse direction

Time period(Sec)	=	2.500	in Longitudinal direction
	=	2.500	In Transverse direction

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

$$\frac{S_a}{g} = 0.400$$

$$\frac{S_a}{g} = 0.40$$

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

$$\frac{S_a}{g} = 0.544$$

$$\frac{S_a}{g} = 0.54$$

For Soft Soil Strata, with $N < 10$

$$\frac{S_a}{g} = 0.668$$

$$\frac{S_a}{g} = 0.668$$

Medium Soil Strata

S_a/g	=	0.544	in Longitudinal direction
S_a/g	=	0.544	In Transverse direction

Seismic Coeff.	=	0.033	in Longitudinal direction
	=	0.033	In Transverse direction

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

A_h	=	0.033	In Longitudinal direction
A_h	=	0.033	In Transverse direction
A_v	=	0.022	In Vertical direction

Load Factors (As per IRC:6-2017)

	SLS	ULS	
Loads	Rare Combination	Basic Combination	Seismic Combination
Dead Load+SIDL except wearing course	1.000	1.350	1.35
Wearing Course only	1.200	1.750	1.75
CWLL Leading	1.000	1.500	0.00
CWLL accompanied	0.750	1.150	0.20
Seismic Effect (During Service)	0.000	0.000	1.50
Wind Leading	1.000	1.500	
Wind accompanied	0.600	0.900	
Thermal Leading	1.000		
Thermal accompanied	0.600		

DESIGN INPUTS

c/c Bearing (span)	=	23.750	m	
Skew Angle	=	0	deg	0.000 rad
Total No. of Pin Bearing	=	0		
Total No. of Metallic Guided bearings	=	0		
Total No. of Fix bearing	B2 & B3	=	1	
Total No. of Long. Guided bearings	B7 & B6	=	1	
Total No. of Trans. Guided bearings	B4 & B1	=	2	
Total No. of Free Bearings at free end	B5 & B8	=	2	
Total No. of bearings at Each Side	n	=	3	
Total No. of bearings for one Span		=	6	
C/C distance of bearings (Trans)		=	2.80	m
Total Width of Superstructure		=	8.50	m
Maximum Free Length For Expansion in Transverse Direction		=	4.25	m
Design Speed	V	=	60.00	kmph
Radius of Curvature	R	=	0.00	m
Friction Coeff.	μ	=	0.05	

Effective Bridge Temperature

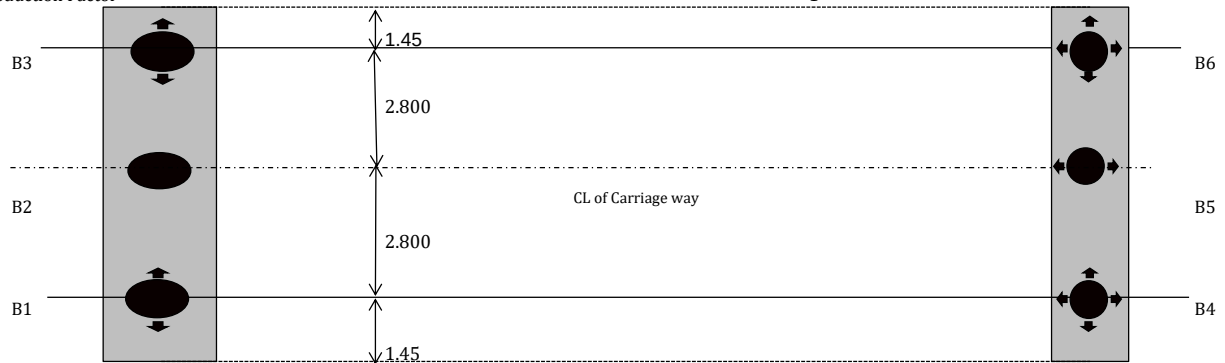
Maximum Air Shade temperature	=	45	/°C (as per Fig 15 of IRC:6-2017)
Minimum Air Shade temperature	=	2.5	/°C (as per Fig 16 of IRC:6-2017)
Mean of max and min temperature	=	23.75	/°C (as per clause 215.2 of IRC:6-2017)
Bridge temperature to be assumed	=	33.75	
Temperature Rise	=	33.75	
Temperature Fall	=	13.75	
Coefficient of Linear Expansion	=	1.20E-05	/°C (Refer Cl:215.4 of IRC:6 - 2017)

SEISMIC COEFFICIENTS		
Horizontal-Long	A_h	0.033
Horizontal-Trans	A_h	0.033
Vertical	A_v	0.022

IMPACT FACTOR for LIVE LOAD	
Class A	1.151
Class 70R W	1.151

Lane Reduction Factor

= 1



FIXED END

FREE END

Symbol:

B2	-	Fixed Bearing
B5	-	Longitudinal Guided Bearing
B1&B3	-	Transverse Guided Bearing
B4&B6	-	Free Bearing

Unfactored Vertical Load (refer design of superstructure)

Load Case	Vertical Load (KN) on Bearing								Max
	BRG 1	BRG 2	BRG 3	BRG 4	BRG 5	BRG 6	BRG 7	BRG 8	
Dead Load	532.11	532.11	532.11	532.11	532.11	532.11			
SIDL1	130.16	130.16	130.16	130.16	130.16	130.16			
SIDL2	72.31	72.31	72.31	72.31	72.31	72.31			
FPLL	0.00	0.00	0.00	0.00	0.00	0.00			
Live Load(Max)	328.65	472.67	-2.32	98.81	84.91	172.81			472.67
Live Load (Min)	-1.14	-0.77	-10.56	0.63	-0.42	-11.35			0.63
Live Load(SV)	0.00	0.00	0.00	0.00	0.00	0.00			0.00
Wind-Max	0.00	0.00	0.00	0.00	0.00	0.00			
Wind-Min	0.00	0.00	0.00	0.00	0.00	0.00			
Seismic-Max	5.22	5.41	4.79	4.92	4.91	5.02			
Seismic-Min	4.79	4.79	4.78	4.80	4.79	4.78			

Wind Force (refer design of superstructure)

				Longitudinal Force							
				BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
WIND - DOWNWARD	=			0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
WIND - UPWARD	=			0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	Max	=		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

			Transverse Force								
			BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8	
WIND -DOWNWARD	=		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
WIND - UPWARD	=		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Max	=		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	

SLS Load Combination

CB1	DL+SIDL1+SIDL2+LL Max
CB2	DL+SIDL1+SIDL2+LL leading + WL acc
CB3	DL+SIDL1+SIDL2+LL acc + WL leading

SLS -Rare Combination**BEARING 1**

		Vertical Force			Horizontal Force	
Description	Factor	Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	532.11	532.11	532.11		
SIDL1	1.00	130.16	130.16	130.16		
SIDL2	1.20	86.77	86.77	86.77		
LL-Leading	1.00	472.67	-1.14			
LL-Accompany	0.75	354.50	-0.86			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1	1221.71	747.90	749.04	83.38	0
Wind Case	CB2	1221.71	747.90	749.04	87.22	0.00
	CB3	1103.54	748.18			

SLS -Rare Combination**BEARING 2**

		Vertical Force			Horizontal Force	
Description	Factor	Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	532.11	532.11	532.11		
SIDL1	1.00	130.16	130.16	130.16		
SIDL2	1.20	86.77	86.77	86.77		
LL-Leading	1.00	472.67	-0.77			
LL-Accompany	0.75	354.50	-0.57			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1	1221.71	748.27	749.04	83.38	0
Wind Case	CB2	1221.71	748.27	749.04	94.42	0.00
	CB3	1103.54	748.47			

SLS -Rare Combination**BEARING 3**

		Vertical Force			Horizontal Force	
Description	Factor	Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	532.11	532.11	532.11		
SIDL1	1.00	130.16	130.16	130.16		
SIDL2	1.20	86.77	86.77	86.77		
LL-Leading	1.00	472.67	-10.56			
LL-Accompany	0.75	354.50	-7.92			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1	1221.71	738.48	749.04	83.38	0
Wind Case	CB2	1221.71	738.48	749.04	70.67	0.00
	CB3	1103.54	741.12			

SLS -Rare Combination**BEARING 4**

		Vertical Force			Horizontal Force	
Description	Factor	Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	532.11	532.11	532.11		
SIDL1	1.00	130.16	130.16	130.16		
SIDL2	1.20	86.77	86.77	86.77		
LL-Leading	1.00	472.67	0.63			
LL-Accompany	0.75	354.50	0.47			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1	1221.71	749.67	749.04	50.05	0
Wind Case	CB2	1221.71	749.67	749.04	75.73	0.00
	CB3	1103.54	749.51			

SLS -Rare Combination**BEARING 5**

		Vertical Force			Horizontal Force	
Description	Factor	Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	532.11	532.11	532.11		
SIDL1	1.00	130.16	130.16	130.16		
SIDL2	1.20	86.77	86.77	86.77		

	LL-Leading	1.00	472.67	-0.42			
	LL-Accompany	0.75	354.50	-0.32			
	WL-Leading	1.00	0.00	0.00			
	WL-Accompan	0.60	0.00	0.00			
Normal	CB1		1221.71	748.62	749.04	50.05	0
Wind Case	CB2		1221.71	748.62	749.04	41.70	0.00
	CB3		1103.54	748.72			

SLS -Rare Combination**BEARING 6**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.00	532.11	532.11	532.11		
	SIDL1	1.00	130.16	130.16	130.16		
	SIDL2	1.20	86.77	86.77	86.77		
	LL-Leading	1.00	472.67	-11.35			
	LL-Accompany	0.75	354.50	-8.52			
	WL-Leading	1.00	0.00	0.00			
	WL-Accompan	0.60	0.00	0.00			
Normal	CB1		1221.71	737.69	749.04	50.05	0.00
Wind Case	CB2		1221.71	737.69	749.04	46.09	0.00
	CB3		1103.54	740.52			

SLS -Rare Combination**BEARING 7**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	0.00	0.00	0.00		
SIDL1	1.00	0.00	0.00	0.00		
SIDL2	1.20	0.00	0.00	0.00		
LL-Leading	1.00	0.00	0.00			
LL-Accompany	0.75	0.00	0.00			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1		0.00	0.00	0.00	0
Wind Case	CB2		0.00	0.00	0.00	0.00
	CB3		0.00	0.00		

SLS -Rare Combination**BEARING 8**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.00	0.00	0.00	0.00		
SIDL1	1.00	0.00	0.00	0.00		
SIDL2	1.20	0.00	0.00	0.00		
LL-Leading	1.00	0.00	0.00			
LL-Accompany	0.75	0.00	0.00			
WL-Leading	1.00	0.00	0.00			
WL-Accompan	0.60	0.00	0.00			
Normal	CB1		0.00	0.00	0.00	0.00
Wind Case	CB2		0.00	0.00	0.00	0.00
	CB3		0.00	0.00		

ULS Load Combination

CB1 DL+SIDL1+SIDL2+LL Max
 CB2 DL+SIDL1+SIDL2+LL leading + WL acc
 CB3 DL+SIDL1+SIDL2+LL acc + WL leading
 CB4 DL+SIDL1+SIDL2+LL leading + EQ

ULS - Basic Combination**BEARING 1**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	-1.71			
	LL-Accompany	1.15	543.57	-1.31			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
	Normal	CB1		1729.61	1018.89	1020.60	121.01
Wind Case	CB2		1729.61	1018.89	1020.60	125.68	0.00
	CB3		1564.18	1019.29			

ULS - Basic Combination**BEARING 2**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	-1.15			
	LL-Accompany	1.15	543.57	-0.88			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
	Normal	CB1		1729.61	1019.46	1020.60	121.01
Wind Case	CB2		1729.61	1019.46	1020.60	136.48	0.00
	CB3		1564.18	1019.72			

ULS - Basic Combination**BEARING 3**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	-15.84			
	LL-Accompany	1.15	543.57	-12.15			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		1729.61	1004.76	1020.60	121.01	0
Wind Case	CB2		1729.61	1004.76	1020.60	100.86	0.00
	CB3		1564.18	1008.46			

ULS - Basic Combination **BEARING 4**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	0.94			
	LL-Accompany	1.15	543.57	0.72			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		1729.61	1021.55	1020.60	71.01	0
Wind Case	CB2		1729.61	1021.55	1020.60	58.44	0.00
	CB3		1564.18	1021.33			

ULS - Basic Combination **BEARING 5**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	-0.63			
	LL-Accompany	1.15	543.57	-0.48			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		1729.61	1019.97	1020.60	71.01	0
Wind Case	CB2		1729.61	1019.97	1020.60	57.40	0.00
	CB3		1564.18	1020.12			

ULS - Basic Combination **BEARING 6**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	718.35	718.35	718.35		
	SIDL1	1.35	175.71	175.71	175.71		
	SIDL2	1.75	126.54	126.54	126.54		
	LL-Leading	1.50	709.01	-17.03			
	LL-Accompany	1.15	543.57	-13.06			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		1729.61	1003.57	1020.60	71.01	0.00
Wind Case	CB2		1729.61	1003.57	1020.60	63.99	0.00
	CB3		1564.18	1007.55			

ULS - Basic Combination **BEARING 7**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	0.00	0.00	0.00		
	SIDL1	1.35	0.00	0.00	0.00		
	SIDL2	1.75	0.00	0.00	0.00		
	LL-Leading	1.50	0.00	0.00			
	LL-Accompany	1.15	0.00	0.00			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		0.00	0.00	0.00	0.00	0
Wind Case	CB2		0.00	0.00	0.00	0.00	0.00
	CB3		0.00	0.00			

ULS - Basic Combination**BEARING 8**

	Description	Factor	Vertical Force			Horizontal Force	
			Max	Min	Permanent	Longitudinal	Transverse
	DL	1.35	0.00	0.00	0.00		
	SIDL1	1.35	0.00	0.00	0.00		
	SIDL2	1.75	0.00	0.00	0.00		
	LL-Leading	1.50	0.00	0.00			
	LL-Accompany	1.15	0.00	0.00			
	WL-Leading	1.50	0.00	0.00			
	WL-Accompan	0.90	0.00	0.00			
Normal	CB1		0.00	0.00	0.00	0.00	0.00
Wind Case	CB2		0.00	0.00	0.00	0.00	0.00
	CB3		0.00	0.00			

ULS - Seismic Combination**BEARING 1**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	-0.23			
Seismic	1.50	7.84	7.19	7.84		
Total		1122.97	1027.57	1028.44	91.93	0.00

ULS - Seismic Combination**BEARING 2**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	-0.15			
Seismic	1.50	8.12	7.19	8.12		
Total		1123.26	1027.64	1028.72	91.93	113.55

ULS - Seismic Combination**BEARING 3**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	-2.11			
Seismic	1.50	7.19	7.17	7.19		
Total		1122.33	1025.66	1027.79	91.93	0.00

ULS - Seismic Combination**BEARING 4**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	0.13			
Seismic	1.50	7.39	7.19	7.39		
Total		1122.52	1027.92	1027.99	50.97	0.00

ULS - Seismic Combination**BEARING 5**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	-0.08			
Seismic	1.50	7.36	7.36	7.36		
Total		1122.50	1027.88	1027.96	50.97	113.55

ULS - Seismic Combination**BEARING 6**

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	718.35	718.35	718.35		
SIDL1	1.35	175.71	175.71	175.71		
SIDL2	1.75	126.54	126.54	126.54		
LL	0.20	94.53	-2.27			
Seismic	1.50	7.53	7.17	7.53		
Total		1122.67	1025.50	1028.14	50.97	0.00

U/S - Seismic Combination BEARING 7

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	0.00	0.00	0.00		
SIDL1	1.35	0.00	0.00	0.00		
SIDL2	1.75	0.00	0.00	0.00		
LL	0.20	0.00	0.00	0.00		
Seismic	1.50	0.00	0.00	0.00		
Total		0.00	0.00	0.00	0.00	0.00

U/S - Seismic Combination BEARING 8

Description	Factor	Vertical Force			Horizontal Force	
		Max	Min	Permanent	Longitudinal	Transverse
DL	1.35	0.00	0.00	0.00		
SIDL1	1.35	0.00	0.00	0.00		
SIDL2	1.75	0.00	0.00	0.00		
LL	0.20	0.00	0.00	0.00		
Seismic	1.50	0.00	0.00	0.00		
Total		0.00	0.00	0.00	0.00	0.00

Calculation of Translation of Superstructure due to Temp., Shrinkage and Creep Reversible Movement due to Temperature rise and fall

(At Free End)

Movement in Longitudinal Direction	=	1.20E-05	x	45	x	23750	=	+	-	12.825 mm
Movement in transverse direction	=	1.20E-05	x	45	x	4250.00	=	+	-	2.295 mm

Note: Increase the movement by ±20mm in both direction of movement with a min. movement of ±50mm in longitudinal direction and ±25mm in transverse direction

										Max
Hence Movement in Longitudinal Direction	=	12.825	+		15	=	27.825	mm		50.000
					subjected to min		50	mm		
Movement in transverse direction	=	2.295	+		15	=	5.000	mm		25.000
					subjected to min		25			

Irreversible Movement due to Shrinkage and Creep

										shrinkage strain
2	Due to shrinkage	=	-2.0E-04	x	23750	=	-4.750	mm		
										creep strain
3	Due to creep	=	$\frac{-4.0E-04}{10}$	x	23750	=	-0.950	mm		

Total movement due to Shrinkage and Creep shall be	=	-5.700	mm
	say	6	mm

Calculation of RotationGrade of concrete = **M45**

For simply supported span it may be computed as

$$\alpha_d = \frac{400 M_{max} L \times 10^{-3}}{(E I)}$$

Es = 2.0E+11

Where Ec = 3.43E+10 Pa

E = Ec / 2 = 1.72E+10 Pa For permanent load

S. No.	Load case	Mmax. (Nm)	Es (Pa)	I (m ⁴)	L (m)	α_d
1	DL	2923000	3.43E+10	0.2218	23.750	3.65E-03
2	SIDL	895000	3.43E+10	0.4910	23.750	5.05E-04
3	F.P.L.L	0.00	3.43E+10	0.4910	23.750	0.00E+00
4	wind	0	3.43E+10	0.4910	23.750	0.00E+00
5	CWLL	2289000	3.43E+10	0.4910	23.750	1.29E-03
					Total	5.44E-03

Note: Increase the rotation by ±0.005 rad subjected to min ± 10/r rad

										Max
Final design rotation	=	0.0054	+	0.005	=		0.0104			0.0104
						Min	0.010			

HORIZONTAL FORCE CALCULATION IN SERVICEABILITY LIMIT STATE**SLS Bearing Forces**

Load Case	Load Factor	Vertical Load (kN) on Bearing							
		BRG 1	BRG 2	BRG 3	BRG 4	BRG 5	BRG 6	BRG 7	BRG 8
Dead Load	1	532.11	532.11	532.11	532.11	532.11	532.11	0.00	0.00
SIDL1	1	130.16	130.16	130.16	130.16	130.16	130.16	0.00	0.00
SIDL2	1.2	86.77	86.77	86.77	86.77	86.77	86.77	0.00	0.00
FPLL	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(Max)	1	328.65	472.67	-2.32	98.81	84.91	172.81	0.00	0.00
Live Load (Min)	1	-1.14	-0.77	-10.56	0.63	-0.42	-11.35	0.00	0.00
Live Load(SV)	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Wind-Max	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Wind-Min	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

SLS Vertical Reaction

		Fix end	Free End	Max
Dead Load Reaction on one End	=	1596.33 kN	1596.33	1596.33
SIDL1 Reaction on one End	=	390.47 kN	390.47	390.47
SIDL2 Reaction on one End	=	216.93 kN	216.93	216.93
Max Live Load reaction on one End	=	799.00 kN	356.53	799.00
Min Live Load reaction	=	-12.47 kN	-11.15	-11.15
Max Live Load on one End (SV)	=	0.00	0.00	0.00
Wind-Max acting with LL on one end	=	0.00 kN	0.00	0.00
Wind-Min acting with LL on one end	=	0.00 kN	0.00	0.00

Calculation of Horizontal Force**Normal Case**

Bearing Force

1-70R Wheel +1-Class A

Load on span	First train	=	1000.00 kN	1-70R Wheel
	Second train	=	0.00 kN	1-70R Wheel
	Third Train	=	0.0 kN	
Bearing Force (Fh)	=	200.00 kN	(20% of First Train and 5% of Second and Third)	

Horizontal Force due to Secondary effect (Temp, Shrinkage etc)

Friction Coeff. μ = 0.05

$\mu (Rg+Rq)$	=	150.14 kN	109.63 kN	110.19 kN
		(LL Max)	(LL Min)	(No LL)
Fh- $\mu(Rg+Rq)$		49.86 kN	90.37 kN	89.81 kN
Fh/2+ $\mu(Rg+Rq)$		250.14 kN	209.63 kN	210.186635 kN
Max=		250.14 kN	209.63 kN	210.19 kN

Centrifugal Force Calculation

As per clause 212 of IRC:6-2017

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

Normal Case

Design Speed	V	=	60.00 kmph
Live Load	W	=	982.72 kN
Radius of Curvature	R	=	0.00 m
CENTRIFUGAL FORCE	C	=	0.00 kN

Longitudinal Force

BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
83.38	83.38	83.38	50.05	50.05	50.05	0.00	0.00

Transverse Force

BRG1	BRG2	BRG3	BRG4	BRG5	BRG6
0	0	0	0	0	0

Wind Case

Load Factor = 1

Factored Longitudinal Force

BRG1	=	0.00
BRG2	=	0.00
BRG3	=	0.00
BRG4	=	0.00
BRG5	=	0.00
BRG6	=	0.00
BRG7	=	0.00
BRG8	=	0.00

Factored Transverse Force

BRG1	=	0.00
BRG2	=	0.00
BRG3	=	0.00
BRG4	=	0.00
BRG5	=	0.00
BRG6	=	0.00
BRG5	=	0.00
BRG6	=	0.00

Braking Force (Fh) = 200.00 kN (20% of First Train and 5% of Second and Third)

Friction Coeff. μ = 0.05**BRG1**

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	53.884	66.667	12.782	87.218	87.218
LL Min	37.395	66.667	29.272	70.728	70.728
No LL	37.452	0.000	-37.452	37.452	37.452

BRG2

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	61.086	66.667	5.581	94.419	94.419
LL Min	37.414	66.667	29.253	70.747	70.747
No LL	37.452	0.000	-37.452	37.452	37.452

BRG3

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	37.336	66.667	29.331	70.669	70.669
LL Min	36.924	66.667	29.743	70.257	70.257
No LL	37.452	0.000	-37.452	37.452	37.452

BRG4

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	42.393	66.667	24.274	75.726	75.726
LL Min	37.483	66.667	29.183	70.817	70.817
No LL	37.452	0.000	-37.452	37.452	37.452

BRG5

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	41.697	0.000	-41.697	41.697	41.697
LL Min	37.431	0.000	-37.431	37.431	37.431
No LL	37.452	0.000	-37.452	37.452	37.452

BRG6

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	46.092	0.000	-46.092	46.092	46.092
LL Min	36.884	0.000	-36.884	36.884	36.884
No LL	37.452	0.000	-37.452	37.452	37.452

BRG7

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	0.000	0.000	0.000	0.000	0.000
LL Min	0.000	0.000	0.000	0.000	0.000
No LL	0.000	0.000	0.000	0.000	0.000

BRG8

Case	$\mu (Rg+Rq)$	Fh	Fh- $\mu(Rg+Rq)$	Fh/2+ $\mu(Rg+Rq)$	Max
LL Max	0.000	0.000	0.000	0.000	0.000
LL Min	0.000	0.000	0.000	0.000	0.000
No LL	0.000	0.000	0.000	0.000	0.000

Longitudinal Wind Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
87.22	94.42	70.669	75.73	41.70	46.092	0.00	0.000

Transverse Wind Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000

HORIZONTAL FORCE CALCULATION IN ULTIMATE LIMIT STATE**ULS-Basic Load Combination**

Load Case	Load Factor	Vertical Load (kN) on Bearing							
		BRG 1	BRG 2	BRG 3	BRG 4	BRG 5	BRG 6	BRG 7	BRG 8
Dead Load	1.35	718.35	718.35	718.35	718.35	718.35	718.35	0.00	0.00
SIDL1	1.35	175.71	175.71	175.71	175.71	175.71	175.71	0.00	0.00
SIDL2	1.75	126.54	126.54	126.54	126.54	126.54	126.54	0.00	0.00
FPLL	1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Live Load(Max)	1.5	492.97	709.01	-3.48	148.22	127.36	259.22	0.00	0.00
Live Load (Min)	1.5	-1.71	-1.15	-15.84	0.94	-0.63	-17.03	0.00	0.00
Live Load(SV)	1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Wind-Max	1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Wind-Min	1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Unfactored Vertical Reaction

		Fix end	Free End	Max
Dead Load Reaction on one End	=	2155.04 kN	2155.04	2155.04
SIDL1 Reaction on one End	=	527.14 kN	527.14	527.14
SIDL2 Reaction on one End	=	379.63 kN	379.63	379.63
Max Live Load reaction on one End	=	1198 kN	535	1198.50
Min Live Load reaction	=	-19 kN	-17	-16.72
Max Live Load on one End (SV)	=	0	0	0.00
Wind-Max acting with LL on one end	=	0 kN	0	0.00
Wind-Min acting with LL on one end	=	0 kN	0	0.00

Calculation of Horizontal Force**Normal Case**

Bearing Force

1-70R Wheel +1-Class A

Load on span	First train	=	1500.00 kN	1-70R Wheel
	Second train	=	0.00 kN	1-70R Wheel
	Third Train	=	0 kN	
Bearing Force (Fh)	=	300.00 kN	(20% of First Train and 5% of Second and Third)	

Horizontal Force due to Secondary effect (Temp, Shrinkage etc)

Friction Coeff. μ = 0.05

$\mu (Rg+Rq)$	=	213.02 kN	152.25 kN	153.09 kN
		(LL Max)	(LL Min)	(No LL)
Fh- $\mu(Rg+Rq)$		86.98 kN	147.75 kN	146.91 kN
Fh/2+ $\mu(Rg+Rq)$		363.02 kN	302.25 kN	303.090617 kN
Max=		363.02 kN	302.25 kN	303.09 kN

Centrifugal Force Calculation

As per clause 212 of IRC:6-2017

CENTRIFUGAL FORCE	C	=	$\frac{W V^2}{127 R}$	
			Normal Case	Seismic Case
Design Speed	V	=	60.00 kmph	60.00
Live Load	W	=	1474.08 kN	294.82
Radius of Curvature	R	=	0.00 m	0.00
CENTRIFUGAL FORCE	C	=	0.00 kN	0.00

Longitudinal Force

BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
121.01	121.01	121.01	71.01	71.01	71.01	0.00	0.00

Transverse Force

BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
0	0	0	0	0	0	0	0

Wind Case

Load Factor = 1.5

Factored Longitudinal Force

BRG1	=	0.00
BRG2	=	0.00
BRG3	=	0.00
BRG4	=	0.00
BRG5	=	0.00
BRG6	=	0.00
BRG7	=	0.00
BRG8	=	0.00

Factored Transverse Force

BRG1	=	0.00
BRG2	=	0.00
BRG3	=	0.00
BRG4	=	0.00
BRG5	=	0.00
BRG6	=	0.00
BRG7	=	0.00
BRG8	=	0.00

Braking Force (Fh) = 300.00 kN (20% of First Train and 5% of Second and Third)

Friction Coeff. μ = 0.05**BRG1**

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	75.679	100.000	24.321	125.679	125.679
LL Min	50.944	100.000	49.056	100.944	100.944
No LL	51.030	0.000	-51.030	51.030	51.030

BRG2

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	86.481	100.000	13.519	136.481	136.481
LL Min	50.973	100.000	49.027	100.973	100.973
No LL	51.030	0.000	-51.030	51.030	51.030

BRG3

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	50.856	100.000	49.144	100.856	100.856
LL Min	50.238	100.000	49.762	100.238	100.238
No LL	51.030	0.000	-51.030	51.030	51.030

BRG4

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	58.441	0.000	-58.441	58.441	58.441
LL Min	51.077	0.000	-51.077	51.077	51.077
No LL	51.030	0.000	-51.030	51.030	51.030

BRG5

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	57.398	0.000	-57.398	57.398	57.398
LL Min	50.999	0.000	-50.999	50.999	50.999
No LL	51.030	0.000	-51.030	51.030	51.030

BRG6

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	63.991	0.000	-63.991	63.991	63.991
LL Min	50.179	0.000	-50.179	50.179	50.179
No LL	51.030	0.000	-51.030	51.030	51.030

BRG7

Case	$\mu (R_g+R_q)$	Fh	Fh- $\mu(R_g+R_q)$	Fh/2+ $\mu(R_g+R_q)$	Max
LL Max	0.000	0.000	0.000	0.000	0.000
LL Min	0.000	0.000	0.000	0.000	0.000
No LL	0.000	0.000	0.000	0.000	0.000

BRG8

Case	$\mu (R_g+R_q)$	F_h	$F_h - \mu(R_g+R_q)$	$F_h/2 + \mu(R_g+R_q)$	Max
LL Max	0.000	0.000	0.000	0.000	0.000
LL Min	0.000	0.000	0.000	0.000	0.000
No LL	0.000	0.000	0.000	0.000	0.000

Longitudinal Wind Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
125.68	136.48	100.856	58.44	57.40	63.991	0.00	0.000

Transverse Wind Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Seismic (Long) Case

Seismic Load Factor	=	1.50
Longitudinal Seismic Force		
Seismic Force (Fs)	=	215.79 kN
Braking Force	=	60.00 kN
$\mu (R_g+R_q)$	=	152.92 kN
$F_h - \mu(R_g+R_q)$	=	122.87 kN
$F_h/2 + \mu(R_g+R_q)$	=	290.82 kN
$F_h + F_s$	=	275.79 kN

Seismic (Trans) Case

Transverse Seismic Force		
F_t	=	$A_h \times (DL+SIDL+LL_{max}/5)$
	=	227.10 kN

Longitudinal Seismic Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
91.93	91.93	91.93	50.97	50.97	50.97	0.00	0.00

Transverse Seismic Force							
BRG1	BRG2	BRG3	BRG4	BRG5	BRG6	BRG7	BRG8
0.00	113.55	0.00	0.00	113.55	0.00	0.00	0

SPHERICAL BEARING DESIGN FORCES

Bearing Type				Fix Bearing	Trasverse Guided Bearing	Longitudinal Guided Bearing	Free Bearing
Bearing Identification Mark/Number				B2	B1&B3	B5	B4&B6
Seating Material	Upper Surface			Concrete (M45)	Concrete (M45)	Concrete (M45)	Concrete (M45)
	Lower Surface			Concrete (M45)	Concrete (M45)	Concrete (M45)	Concrete (M45)
Design Load (kN)	Serviceability Limit State (Normal)	Vertical Load	Max	1222	1222	1222	1222
			Permenant	749	749	749	749
			Min	748	748	749	750
		Horizontal Load	Longitudinal	83	83	50	50
			Transverse	0	0	0	0
	Serviceability Limit State (Wind)	Vertical Load	Max	1222	1222	1222	1222
			Permenant	749	749	749	749
			Min	748	748	749	750
		Horizontal Load	Longitudinal	94	87	42	76
			Transverse	0	0	0	0
	Ultimate Limit State (Normal)	Vertical Load	Max	1730	1730	1730	1730
			Permenant	1021	1021	1021	1021
			Min	1019	1019	1020	1022
		Horizontal Load	Longitudinal	121	121	71	71
			Transverse	0	0	0	0
	Ultimate Limit State (Wind)	Vertical Load	Max	1730	1730	1730	1730
			Permenant	1021	1021	1021	1021
			Min	1020	1019	1020	1022
		Horizontal Load	Longitudinal	136	126	57	64
			Transverse	0	0	0	0
	Ultimate Limit State (Seismic)	Vertical Load	Max	1123	1123	1122	1123
			Permenant	1029	1028	1028	1028
			Min	1028	1028	1028	1028
		Horizontal Load	Longitudinal	92	92	51	51
			Transverse	114	0	114	0
Translation (mm)	Serviceability Limit State (Normal)	Irreversible	Longitudinal	-	-	-	-
			Transverse	-	-	-	-
		Reversible	Longitudinal	-	-	± 50	± 50
			Transverse	-	± 25	± -	± 25
	Ultimate Limit State (Normal)	Irreversible	Longitudinal	-	-	-	-
			Transverse	-	-	-	-
		Reversible	Longitudinal	-	-	-	-
			Transverse	-	-	-	-
Rotation (Radian)	Serviceability Limit State (Normal)	Irreversible	Longitudinal	0.010	0.010	0.010	0.010
			Transverse	-	-	-	-
		Reversible	Longitudinal	0.010	0.010	0.010	0.010
			Transverse	-	-	-	-

ANNEXURE-I : STAAD INPUT FOR ANALYSIS OF SUPERSTRUCTURE

STAAD FLOOR ANALYSIS OF COMPOSITE GIRDER & DECK FOR SIDL (23.75M)

START JOB INFORMATION

ENGINEER DATE 23-MAY-13

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 0.35 0 0; 3 2 0 0; 4 3.5 0 0; 5 6.288 0 0; 6 9.256 0 0;
 7 12.225 0 0; 8 15.194 0 0; 9 18.163 0 0; 10 20.95 0 0; 11 22.45 0 0;
 12 24.1 0 0; 13 24.45 0 0; 14 0 0 0.25; 15 0.35 0 0.25; 16 2 0 0.25;
 17 3.5 0 0.25; 18 6.288 0 0.25; 19 9.256 0 0.25; 20 12.225 0 0.25;
 21 15.194 0 0.25; 22 18.163 0 0.25; 23 20.95 0 0.25; 24 22.45 0 0.25;
 25 24.1 0 0.25; 26 24.45 0 0.25; 27 0 0 1.45; 28 0.35 0 1.45; 29 2 0 1.45;
 30 3.5 0 1.45; 31 6.288 0 1.45; 32 9.256 0 1.45; 33 12.225 0 1.45;
 34 15.194 0 1.45; 35 18.163 0 1.45; 36 20.95 0 1.45; 37 22.45 0 1.45;
 38 24.1 0 1.45; 39 24.45 0 1.45; 40 0 0 2.85; 41 0.35 0 2.85; 42 2 0 2.85;
 43 3.5 0 2.85; 44 6.288 0 2.85; 45 9.256 0 2.85; 46 12.225 0 2.85;
 47 15.194 0 2.85; 48 18.163 0 2.85; 49 20.95 0 2.85; 50 22.45 0 2.85;
 51 24.1 0 2.85; 52 24.45 0 2.85; 53 0 0 4.25; 54 0.35 0 4.25; 55 2 0 4.25;
 56 3.5 0 4.25; 57 6.288 0 4.25; 58 9.256 0 4.25; 59 12.225 0 4.25;
 60 15.194 0 4.25; 61 18.163 0 4.25; 62 20.95 0 4.25; 63 22.45 0 4.25;
 64 24.1 0 4.25; 65 24.45 0 4.25; 66 0 0 5.65; 67 0.35 0 5.65; 68 2 0 5.65;
 69 3.5 0 5.65; 70 6.288 0 5.65; 71 9.256 0 5.65; 72 12.225 0 5.65;
 73 15.194 0 5.65; 74 18.163 0 5.65; 75 20.95 0 5.65; 76 22.45 0 5.65;
 77 24.1 0 5.65; 78 24.45 0 5.65; 79 0 0 7.05; 80 0.35 0 7.05; 81 2 0 7.05;
 82 3.5 0 7.05; 83 6.288 0 7.05; 84 9.256 0 7.05; 85 12.225 0 7.05;
 86 15.194 0 7.05; 87 18.163 0 7.05; 88 20.95 0 7.05; 89 22.45 0 7.05;
 90 24.1 0 7.05; 91 24.45 0 7.05; 92 0 0 8.25; 93 0.35 0 8.25; 94 2 0 8.25;
 95 3.5 0 8.25; 96 6.288 0 8.25; 97 9.256 0 8.25; 98 12.225 0 8.25;
 99 15.194 0 8.25; 100 18.163 0 8.25; 101 20.95 0 8.25; 102 22.45 0 8.25;
 103 24.1 0 8.25; 104 24.45 0 8.25; 131 0 0 8.5; 132 0.35 0 8.5; 133 2 0 8.5;
 134 3.5 0 8.5; 135 6.288 0 8.5; 136 9.256 0 8.5; 137 12.225 0 8.5;
 138 15.194 0 8.5; 139 18.163 0 8.5; 140 20.95 0 8.5; 141 22.45 0 8.5;
 142 24.1 0 8.5; 143 24.45 0 8.5;

MEMBER INCIDENCES

1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
 11 11 12; 12 12 13; 13 14 15; 14 15 16; 15 16 17; 16 17 18; 17 18 19; 18 19 20;
 19 20 21; 20 21 22; 21 22 23; 22 23 24; 23 24 25; 24 25 26; 25 27 28; 26 28 29;
 27 29 30; 28 30 31; 29 31 32; 30 32 33; 31 33 34; 32 34 35; 33 35 36; 34 36 37;
 35 37 38; 36 38 39; 37 40 41; 38 41 42; 39 42 43; 40 43 44; 41 44 45; 42 45 46;
 43 46 47; 44 47 48; 45 48 49; 46 49 50; 47 50 51; 48 51 52; 49 53 54; 50 54 55;
 51 55 56; 52 56 57; 53 57 58; 54 58 59; 55 59 60; 56 60 61; 57 61 62; 58 62 63;
 59 63 64; 60 64 65; 61 66 67; 62 67 68; 63 68 69; 64 69 70; 65 70 71; 66 71 72;
 67 72 73; 68 73 74; 69 74 75; 70 75 76; 71 76 77; 72 77 78; 73 79 80; 74 80 81;
 75 81 82; 76 82 83; 77 83 84; 78 84 85; 79 85 86; 80 86 87; 81 87 88; 82 88 89;
 83 89 90; 84 90 91; 109 92 93; 110 93 94; 111 94 95; 112 95 96; 113 96 97;
 114 97 98; 115 98 99; 116 99 100; 117 100 101; 118 101 102; 119 102 103;
 120 103 104; 121 131 132; 122 132 133; 123 133 134; 124 134 135; 125 135 136;
 126 136 137; 127 137 138; 128 138 139; 129 139 140; 130 140 141; 131 141 142;
 132 142 143; 133 1 14; 134 14 27; 135 27 40; 136 40 53; 137 53 66; 138 66 79;
 139 79 92; 142 92 131; 143 2 15; 144 15 28; 145 28 41; 146 41 54; 147 54 67;
 148 67 80; 149 80 93; 152 93 132; 153 3 16; 154 16 29; 155 29 42; 156 42 55;
 157 55 68; 158 68 81; 159 81 94; 162 94 133; 163 4 17; 164 17 30; 165 30 43;
 166 43 56; 167 56 69; 168 69 82; 169 82 95; 172 95 134; 173 5 18; 174 18 31;
 175 31 44; 176 44 57; 177 57 70; 178 70 83; 179 83 96; 182 96 135; 183 6 19;
 184 19 32; 185 32 45; 186 45 58; 187 58 71; 188 71 84; 189 84 97; 192 97 136;
 193 7 20; 194 20 33; 195 33 46; 196 46 59; 197 59 72; 198 72 85; 199 85 98;
 202 98 137; 203 8 21; 204 21 34; 205 34 47; 206 47 60; 207 60 73; 208 73 86;
 209 86 99; 212 99 138; 213 9 22; 214 22 35; 215 35 48; 216 48 61; 217 61 74;
 218 74 87; 219 87 100; 222 100 139; 223 10 23; 224 23 36; 225 36 49; 226 49 62;
 227 62 75; 228 75 88; 229 88 101; 232 101 140; 233 11 24; 234 24 37; 235 37 50;
 236 50 63; 237 63 76; 238 76 89; 239 89 102; 242 102 141; 243 12 25; 244 25 38;
 245 38 51; 246 51 64; 247 64 77; 248 77 90; 249 90 103; 252 103 142; 253 13 26;
 254 26 39; 255 39 52; 256 52 65; 257 65 78; 258 78 91; 259 91 104; 262 104 143;

MEMBER PROPERTY INDIAN

*DUMMY MEMBERS

1 TO 24 PRIS AX 0.001 IX 0.0001 IZ 0.0001
 109 TO 132 PRIS AX 0.001 IX 0.0001 IZ 0.0001
 37 TO 48 PRIS AX 0.001 IX 0.0001 IZ 0.0001
 61 TO 72 PRIS AX 0.001 IX 0.0001 IZ 0.0001
 133 TO 139 142 PRIS AX 0.001 IX 0.0001 IZ 0.0001

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253 TO 259 262 PRIS AX 0.001 IX 0.0001 IZ 0.0001
*OUTER L-GIRDER
25 26 35 36 1.7982 IX 0.0979 0.4797 IZ 0.5819
27 34 75 82 1.5742 IX 0.0596 0.4652 IZ 0.5365
28 TO 33 76 0.0213 IY 0.4507 0.491
*
*INNER L-GIRDER
49 50 59 60 IY 0.4574 IZ 0.5783
51 58 PRIS 0.0595 IY 0.4429 0.5334
52 TO 57 PF IX 0.0212 IY 0.4283 IZ 0.4885
*
*END X-GIRDER
145 TO 149 0.0283 IY 0.0084 0.129

*INTERMEDIATE X-GIRDER
*TRANS. SLAB MEMBER
143 144 152 0.2960 IX 0.0068 0.0158 IZ 0.0034
153 TO 159 0.0043 IY 0.2534 0.0021
163 TO 169 0.0038 IY 0.1807 0.0019
173 TO 179 0.0051 IY 0.4376 0.0026
183 TO 189 192 TO 199 202 TO 209 -
212 PRIS AX 0.653 IX 0.0053 IY 0.4793 IZ 0.0026
*
SUPPORTS
28 54 80 FIXED BUT FX FZ MX MY MZ
38 64 90 PINNED
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 3.4e+007
POISSON 0.15
DENSITY 25
END DEFINE MATERIAL
CONSTANTS
MATERIAL MATERIAL1 ALL

*****DEFINE MOVING LOAD
DEFINE MOVING LOAD
**1-70RW
TYPE 1 LOAD 85 85 85 85 60 60 40
DIST 1.37 3.05 1.37 2.13 1.52 3.96
**2-CLASS A
TYPE 2 LOAD 34 34 34 34 57 57 13.5 13.5
DIST 3 3 3 4.3 1.2 3.2 1.1
*****
*LOADING
LOAD 1 SIDL-1
MEMBER LOAD
*DUE TO CRASH BARRIER
13 TO 24 UNI GY -8
109 TO 120 UNI GY -8
*****
LOAD 2 SIDL-2
*DUE TO W/C
FLOOR LOAD
YRANGE 0 0 FLOAD -2.2 X RANGE 0 24.45 Z RANGE 0.25 8.25 GY
***** 1- 70R
LOAD GENERATION 440
TYPE 1 -13.4 0 2.13 XINC 0.1
TYPE 1 -13.4 0 4.06 XINC 0.1
*****CWLL ( 2-Class A )
LOAD GENERATION 440
TYPE 2 -18.8 0 0.9 XINC 0.1
TYPE 2 -18.8 0 2.7 XINC 0.1
TYPE 2 -18.8 0 4.4 XINC 0.1
TYPE 2 -18.8 0 6.2 XINC 0.1
PERFORM ANALYSIS
FINISH
*****

```