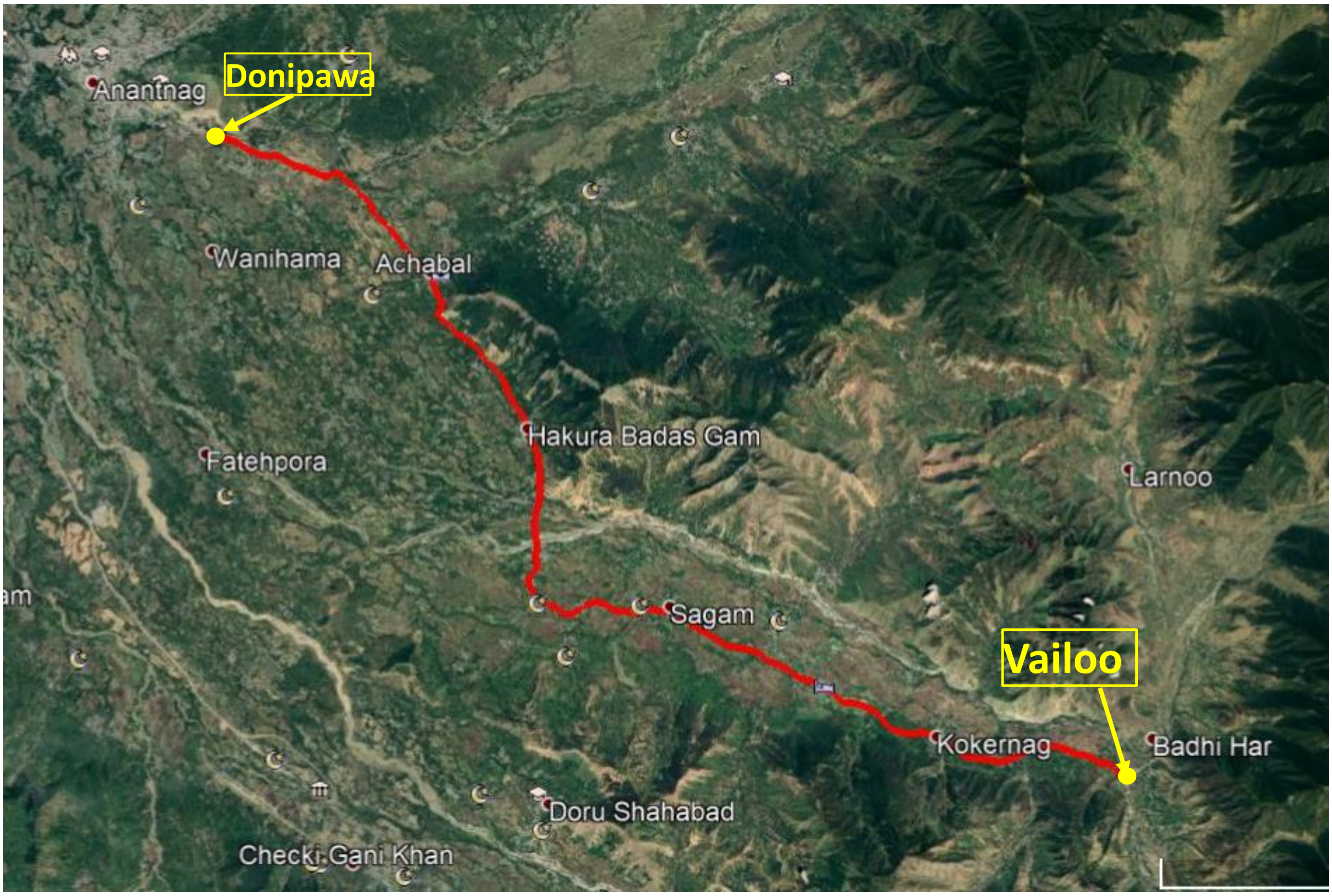
RC-1564-HO-DGN-RD -DPR -001 - D0

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 upgradation to 2 lane with paved shoulder from (i) Km 44.500 to Km 142.000 of Chattroo Village & (ii) Km 235.00 (Vailoo Village) to Km 269.00 (Khanabal) of Khellani- Kishtwar-Chattroo- Khanabal Section of NH 244 in the state of Jammu & Kashmir



FINAL DETAILED PROJECTREPORT

VAILOO TO DONIPAWA SECTION

VOLUME-II B: DESIGN REPORT STRUCTURE

NOVEMBER 2020

RODIC CONSULTANTS PVT. LTD.

IN JV WITH

MONARCH SURVEYORS AND ENGINEERING CONSULTANTS PVT. LTD.

DESIGN INDEX

Consultancy Services for feasibility study, Preparation of Detailed Project Report and Providing Preconstruction Services for Upgradation to 2 Lane with Paved Shoulder from i) Km. 44+500 to 142+000 of Chatroo Village & ii) Km. 235+00 (Vaillo Village) to Km 269+000 (Khanabal) of Khellani-Kishtawar-Chatroo-Khanabal Section of NH-244 in the state of Jammu and Kashmir

Volume II (A) - Design Calculations for Bridges / Structures

DESIGN OF ABUTMENT(A1) WITH OPEN FOUNDATION

FOR MINOR BRIDGE (WANGON BRIDGE) AT CH:-151+096)

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.	=	28.700m	28.700m
Thickness of Expansion Joint	=	0.050m	0.050m
Slab projection Beyond C/L of Bearing (Back Side)	=	0.625m	0.625m
Slab projection Beyond C/L of Bearing (Span Side)	=	0.625m	0.625m
Span -c/c of E.J.	=	30.000m	30.00m
Type of Superstructure	=	Composite Steel Girder	
Width of Crash barrier (Both Side)	=	0.500m	
Width of Carriageway	=	11.000m	
Projection beyond crash barrier	=	0.000m	
Thickness of Wearing coat	=	0.065m	
Length of Approach Slab (Right)	=	3.500m	3.500m
Width of Footpath on one side	=	0.000m	
Railing/kerb on footpath edge	=	0.250m	
Total Width of Superstructure	=	12.500m	
Median Width minus 20mm gap	=	0.000m	

Bearings

No of Bearings	=	5(Excluding pin bearing)	
Type of Bearing	=	POT -PTFE	
Coeff. Of Friction for POT-PTFE Bearing	=	0.05	
C/C Distance of Bearing in Transverse Direction	=	2.500m (Right)	2.500 (Skew)

Type of Soil	=	1	Hard or Rocky Strata
NBC of soil -Normal Case	=	350	kN/m^2 Assumed
SBC of soil-Normal Case	=	400	kN/m^2
SBC of soil-Seismic Case	=	500	kN/m^2
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	=	2.140m	
Height of Trapering portion	=	0.097m	
Length of Dirt Wall at top (Uniform portion)	=	12.500m	12.500m
Length of Dirt Wall at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment Cap

Width of Abutment cap of Uniform portion	=	1.775m	1.775m
Width of Abutmentcap at bottom of Tapering Portion	=	1.000m	1.000m
Projection of Abutment Cap (Span Side)	=	0.300m	0.300m

Projection of Abutment Cap Back Side	=	0.475m	0.475m
Abutmentcap thickness (Uniform portion)	=	0.300m	
Abutmentcap thickness (Tapering Portion)	=	0.300m	
Length of Abutment Cap at top (Uniform portion)	=	12.500m	12.500m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.500m	12.500m

Abutment- Wall Type

Thickness of Abutment	=	1.000m	
Width of abutment shaft	=	12.500m	12.500m
Thickness of Abutment shaft at Top	=	1.000m	1.000m
Thickness of Abutment shaft at HFL	=	1.129m	1.129m
Thickness of Abutment shaft at Bottom	=	1.300m	1.300m

Solid Return Wall

Length of Return wall	=	5.500m
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	=	10.000m
abutment pedestal width	=	1.300m
abutment pedestal Height	=	1.250m
Width of Toe Slab	=	3.200m
Width of Heel Slab	=	5.500m
Thickness of Toe slab at tip	=	0.500m
Thickness of Toe slab near shaft	=	1.250m
Thickness of heel slab at tip	=	0.500m
Thickness of heel slab near shaft	=	1.250m
Width of backfill on heel slab	=	5.500m
Thickness of heel slab at back fill edge	=	1.250m
Height of back fill at bottom edge of heel slab	=	10.445m
Height of back fill at back fill edge of heel slab	=	9.695m

Across Traffic Direction:

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

Levels

Deck Level at Median Edge=	**1980.517m**	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	1980.373m	Height of Superstructure	=	1.575m
Deck level at center line =	1980.517m	Min. Height of Footpath Side Pedestal (1)	=	0.500m
Soffit Level at center of bridge =	1978.877m	Height of Pedestal (2)	=	0.563m
Abutment cap top level =	1978.208m	Height of Pedestal (3)	=	0.625m
Abutment cap bottom lvl (uniform portion ends	1977.908m	Height of Pedestal (4)	=	0.688m
Abutment cap bottom lvl (corbel portion ends)	1977.608m	Height of Pedestal (5)	=	0.625m
Abutment shaft top level =	1977.608m	Height of Pedestal (6)	=	0.000m
		Height of Pedestal (7)	=	0.000m
Ground level/LBL =	1972.670m	Distance of nearest girder to c.l. of deck	=	1.250m
Abutment shaft bottom level =	1970.750m	Height (Avg.) of Dirt Wall	=	2.237m
Foundation level =	1969.500m	Abutment shaft Above G.L	=	4.938m
HFL =	1974.670m	Abutment Shaft below G.L	=	1.920m
		Height of abutment shaft	=	6.858m
		MSL	=	1971.500m
		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 Characteistic Strength, $\varepsilon c2$	=	0.02	
Ultimate Strain, $\varepsilon cu2$	=	0.035	
Ecm	=	32308.250	N/mm^2
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa (1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m^3
Dry unit weight of soil	=	20	kN/m^3
Permissible Crack Width	=	0.3	mm - For Moderate Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48 fck	
	=	16.8	N/mm^2
Maximum tensile stress in steel under rare combination	=	300	N/mm^2

Creep Coefficient

For Abutment Shaft	=	2	for 90 days
For Footing	=	2	for 90 days

Clear Cover to Reinforcement

Earth Face	=	75	mm
Non-Earth Face	=	50	mm

Seismic Data:

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

Properties of backfill material :

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	=	342.45	5.00
DL Reaction on Bearing No.-2	=	319.01	2.50
DL Reaction on Bearing No.-3	=	319.01	0.00
DL Reaction on Bearing No.-4	=	319.01	-2.50
DL Reaction on Bearing No.-5	=	342.45	-5.00
DL Reaction on Bearing No.-6	=	0.00	0.00
DL Reaction on Bearing No.-7	=	0.00	0.00
		1641.91	0.000

Super Imposed Dead Load Reactions (Excluding Wearing Course)

SIDL Reaction on Bearing No.-1	=	149.346	5.00
SIDL Reaction on Bearing No.-2	=	0.452	2.50
SIDL Reaction on Bearing No.-3	=	0.002	0.00
SIDL Reaction on Bearing No.-4	=	0.452	-2.50
SIDL Reaction on Bearing No.-5	=	149.346	-5.00
SIDL Reaction on Bearing No.-6	=	0.000	0.00
SIDL Reaction on Bearing No.-7	=	0.000	0.00
		299.60	0.000

Reaction Due to Wearing Course only

SIDL Reaction on Bearing No.-1	=	82.500	5.00
SIDL Reaction on Bearing No.-2	=	75.000	2.50
SIDL Reaction on Bearing No.-3	=	75.000	0.00
SIDL Reaction on Bearing No.-4	=	75.000	-2.50
SIDL Reaction on Bearing No.-5	=	82.500	-5.00
SIDL Reaction on Bearing No.-6	=	0.000	0.00
SIDL Reaction on Bearing No.-7	=	0.000	0.00
		390.00	0.000

Carriageway Live Load Reactions

MAXIMUM REACTION CASE:

(1-70RW + 1-CLASS A) Reduction Factor = **0.9** (for 3 Lane)

max	=	1110.64	KN	Corr. Transv	=	1055.109	kNm	ecc.	0.95
min	=	239.36	KN	Corr. Transv	=	227.391	kNm	ecc.	0.95

SV Loading

max	=	1890.94	KN	Corr. Transv	=	567.282	kNm	ecc.	0.30
min	=	1709.06	KN	Corr. Transv	=	512.718	kNm	ecc.	0.30

MAXIMUM TRASVERSE MOMENT CASE:

(1-70RW) Reduction Factor = **1** (for 2 Lane)

max	=	843.25	KN	Corr. Transv	=	2660.446	kNm	ecc.	3.16
min	=	156.75	KN	Corr. Transv	=	494.554	kNm	ecc.	3.16

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.065
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for Cl A Wheeled loading

Impact Factor upto abut. cap	=	1.065
Impact Factor for Abut. Shaft Base	=	1.000

Bearing Pedestal

Right Span :

Pedestal 1	=	0.500 x	0.800 x	0.800 =	0.3200 m^3	5.000
Pedestal 2	=	0.563 x	0.800 x	0.800 =	0.3600 m^3	2.500
Pedestal 3	=	0.625 x	0.800 x	0.800 =	0.4000 m^3	0.000
Pedestal 4	=	0.688 x	0.800 x	0.800 =	0.4400 m^3	-2.500
Pedestal 5	=	0.625 x	0.800 x	0.800 =	0.4000 m^3	-5.000
Pedestal 6	=	0.000 x	0.800 x	0.800 =	0.0000 m^3	0.000
Pedestal 7	=	0.000 x	0.800 x	0.800 =	0.0000 m^4	0.000
					1.9200 m^3	-0.313

VOLUME CALCULATION

C.G. Of Footing = 5.000 m
C.G. Of shaft from toe tip = 3.850 m
Distance between c.g. of shaft and footing = 1.150 m

Description	No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
		m	m	m	m^3	m	m	m	
Dirt Wall -Uniform portion	1	12.50	0.300	2.140	8.025	-0.825	0.325	-4.675	0.000
-Trapering portion	1	12.50	0.300	0.097	0.363	-0.825	0.325	-4.675	0.000
Bracket (Rectangle)	1	12.50	0.300	0.300	1.125	-1.125	0.025	-4.975	0.000
(Corbel) 0.5	1	12.50	0.300	0.300	0.563	-1.075	0.075	-4.925	0.000
Cap (uniform portion)	1	12.50	1.775	0.300	6.656	-0.087	1.063	-3.938	0.000
Cap (Corbel Portion)	1	12.50 12.50	1.775 1.000	0.300	5.134	-0.087	1.063	-3.938	0.000
Shaft above HFL	1	12.50	1.064	2.938	39.088	0.117	1.267	-3.733	0.000
Shaft below HFL	1	12.50	1.214	3.420	51.910	0.042	1.192	-3.808	0.000
Solid Return Wall	1	5.50	0.500	10.142	27.891	-3.400	-2.250	-7.250	6.000
Cantilever Return wall(Rectangular portion)	1	0.00	0.000	0.000	0.000	-0.650	0.500	-4.500	6.000
Cantilever Return wall(Traingular portion)	1	0.00	0.000	0.000	0.000	-0.650	0.500	-4.500	6.000
Footing									
Heel Slab	1	12.50	5.500	0.875	60.156		-1.857	-6.857	0.000
Toe Slab	1	12.50	3.200	0.875	35.000		3.171	-1.829	0.000
Portion between Heel and Toe	1	12.50	1.300	1.250	20.313		1.150	-3.850	0.000
Back filling above HFL over Heel Slab	1	12.50	5.500	5.847	401.981		-2.250	-7.250	0.000
Back filling below HFL over Heel Slab	1	12.50	5.500	4.295	295.281		-2.330	-7.330	0.000
Backfill above Heel slab	1	12.50	5.500	10.070	692.321		-2.284	-7.284	0.000
Front Filling over Toe Slab	1	12.50	3.200	2.295	91.800		3.313	-1.687	0.000
Front Filling over Toe Slab in HFL Case	1	12.50	3.200	1.125	45.000		2.072	-2.928	0.000
Side filling between heel and toe	1	0.00	1.300	2.295	0.000		0.000	0.000	0.000
Aproach Slab	1	12.500	1.750	0.300	6.563	-1.125	0.025	-4.975	0.000
Back fill above HFL on flared portion of stem	1	12.50	0.129	5.847	9.394		0.621	-4.379	0.000
Back fill below HFL on flared portion of stem	1	12.50	0.171	4.295	9.206		0.557	-4.443	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barri	2	8 kN/m	1.750	28.00kN		-0.825	0.325	-4.675

SECTIONAL PROPERTIES

Width of Footing (B) = 10.000 m
Length of Footing (L) = 12.500 m

A = 10.000 x 12.500 = 125.000 m^2
ZL = 12.500 x 16.667 = 208.333 m^3
ZT = $\frac{IT1 + IT2}{\text{distance of extreme point from centre}}$

IT1 = 10.000 x 162.760 = 1627.60 m^4
IT2 (moment of inertia of triangle) = 10.000 x 0.000 + 0.500 x 10.000 x 0.000 x 39.063
from centre of footing = 0.000 m^4

Moment of inertia of two triangle = 0.000 m^4

Total moment of inertia = 1627.60 m^4

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

Total Section modulus (ZT) = 260.417 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 Laod,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 Laod+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 Servicibilty Limit State

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Laod+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 Laod+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

Feq = Seismic forces to be resisted
Feq = Ah x (Dead load + Appropriate Live load)
Ah = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \; \frac{Sa}{g}}{\frac{R}{I}}$$

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.

$$= 2.0 \left(\frac{D}{1000F}\right)^{1/2}$$

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

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

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction
= 9.208 m
C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction
= 10.747 m

Abutment Cap Top Level - Foundation Level
= 8.708 m

Dimensions of Abutment Shaft

Length	=	12.50	m
Width	=	1.15	m
Moment of Inertia , $I_{longitudinal}$	=	1.188	m^4
Moment of Inertia , $I_{transverse}$	=	187.174	m^4
Ecm	=	3.231E+07	kN/m^2

Longitudinal Direction				**Transverse Direction**			
Force	=	160.563	KN	Force	=	20330.768	KN
D	=	2331.51	KN	D	=	2601.512	KN
T	=	0.2410	sec	T	=	0.0226	sec

For Hard or Rocky Strata, with N>30

$\frac{Sa}{g}$ = 2.500 $\frac{Sa}{g}$ = 1.339

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

$\frac{Sa}{g}$ = 2.500 $\frac{Sa}{g}$ = 1.339

For Soft Soil Strata, with N<10

$\frac{Sa}{g}$ = 2.5 $\frac{Sa}{g}$ = 1.339

Hard or Rocky Strata

Sa/g = 2.5 Sa/g = 1.339

Seismic Coeff. In Longitudinal Direction = 0.18

Seismic Coeff. In Transverse Direction = 0.289

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.180	In Longitudinal direction
Ah	=	0.289	In Transverse direction
Av	=	0.193	In Vertical direction

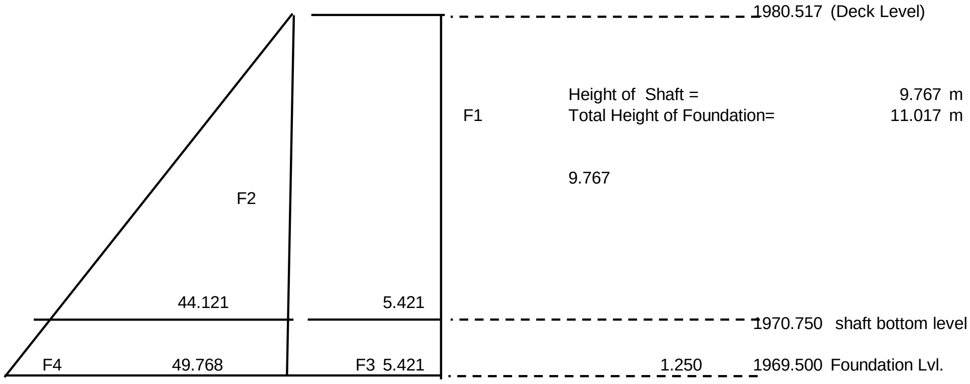
For Design of Foundation (35% increment in Seismic Coeff for Foundation as per IRC:6-2017,Clause No. 219.8)

Ah	=	0.243	In Longitudinal direction
Ah	=	0.289	In Transverse direction
Av	=	0.193	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
	θ1	=	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 **1970.750 m (at Shaft Base)**
@ RL **1969.500 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	9.767	x	12.500		=	**661.813 kN**
M1	=	661.81	x	4.88	=	**3231.963**	**kN.m**		**at Shaft Bottom**
F3	=	5.421	x	11.017	x	12.500		=	**746.513 kN**
M3	=	746.513	x	5.509	=	**4112.166**	**kN.m**		**at Foundation**

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)

	=	0.226	x	20	x	9.767	=	44.121 kN/m^2
F2	=	0.5	x	44.12	x	9.767	x	12.50
	=	**2693.302 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	=	2693.30	x	4.10	=	**11048.303 kN.m**	**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	0.226	x	20	x	11.017	=	49.768 kN/m^2
F4	=	0.5	x	49.77	x	11.017	x	12.50
	=	**3426.805 kN**						
M4	=	3426.81	x	4.63	=	**15856.307 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	9.767	=	46.882 kN/m^2		
F	=	0.5	x	46.882	x	9.767	x	12.500
	=	**2861.829 kN**						
M	=	2861.83	x	3.256	=	**9317.160 kN.m**		**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	4.800	x	11.017	=	52.88 kN/m^2		
F	=	0.5	x	52.882	x	11.02	x	12.500
	=	**3641.229 kN**						
M	=	3641.23	x	3.672	=	**13371.805 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 12.50
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m at Foundation**

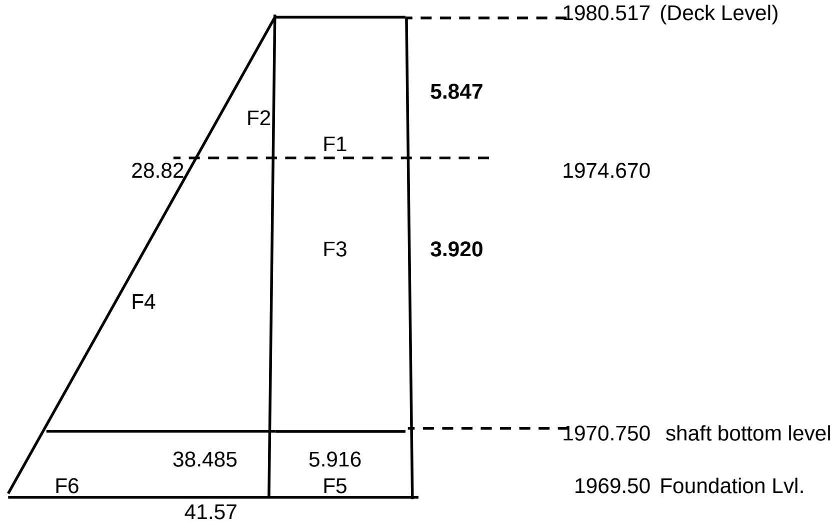
Summary of Moment and Horizontal Force

	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	11048.303	15856.307	2693.302	3426.805
Due to Minimum Fluid Pressure	9317.160	13371.805	2861.829	3641.229
Governing of Two	**11048.303**	**15856.307**	**2861.829**	**3641.229**
Due to Live Load Surcharge	**3231.963**	**4112.166**	**661.813**	**746.513**
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	
	γd	=	20 kN/m^3	
	γwater	=	10 kN/m^3	
Equivalent Live Load Surcharge height		=	1.2 m	

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1970.8 m (at Shaft Base)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.246	x	20	x	1.200	=	5.916 kN/m^2
F1	=	5.916	x	9.767	x	12.500	=	**722.212 kN**
M1	=	722.21	x	4.88	=	**3526.921 kN.m**		**at Shaft Bottom**
F3	=	5.916	x	11.017	x	12.500	=	**814.642 kN**
M3	=	814.64	x	5.51	=	**4487.455 kN.m**		**at Foundation Level**

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.246	x	20	x	5.847	=	28.823 kN/m^2
(At Shaft bottom level) Below HFL	=	0.246	x	10	x	3.920	=	9.662 kN/m^2
F2	=	0.5	x	28.82	x	5.847	x	12.50
	=	**1053.315 kN**						

$$F4 = \frac{(28.82 + 38.49)}{2} \times 3.92 \times 12.50$$

= **1649.066 kN**

Total Force = 2702.381 kN

M2	=	1053.32	x	6.38	=	**6715.664 kN.m**
M4	=	1649.07	x	1.87	=	**3077.513 kN.m**

Total Moment = **9793.18 kN.m** **at Shaft Bottom**

Intensity for triangular portion

Upto HFL = 0.246 x 20 x 5.847 = 28.823 kN/m^2

at Foundation level = 0.246 x 10 x 5.170 = 12.743 kN/m^2

F2 = 0.5 x 28.82 x 5.847 x 12.50
= **1053.315 kN**

F6 = $\frac{(28.82 + 41.57) \times}{2}$ 5.17 x 12.50

= **2274.471 kN**

Total Force = 3327.787 kN

M2 = 1053.32 x 7.63 = **8032.308 kN.m**

M6 = 2274.47 x 2.43 = **5524.710 kN.m**

Total Moment = 13557.02 kN.m Foundation Lvl.

Intensity of Passive pressure:
= 5.249 x 10 x 0.00 = 0.000 kN/m^2
Force due to passive @ Foundation, F
= 0.5 x 0.000 x 0.00 x 12.50
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m** **Foundation Lvl.**

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	9793.177	13557.017	2702.381	3327.787
Due to Minimum Fluid Pressure	9317.160	13371.805	2861.829	3641.229
Governing of Two	**9793.177**	**13557.017**	**2861.829**	**3641.229**
Due to Live Load Surcharge	**3526.921**	**4487.455**	**722.212**	**814.642**
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

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

α h	=	0.180	
α v	=	0.193	
φ	=	35.00	0.611
δ	=	22.50	0.393
α	=	0.00	0.000
β	=	0.00	0.000

α h	=	HORIZONTAL SEISMIC COFFICIENT
α v	=	VERTICAL SEISMIC COFFICIENT
φ	=	ANGLE OF INTERNAL FRICTION OF SOIL
δ	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL
α	=	ANGLE OF FRICTION BETWEEN THE WALL AND EARTH FILL,
β	=	SLOPE OF EARTH FILL

$\lambda = \tan^{-1} \dfrac{\alpha\ h}{(1 \pm \alpha\ v)}$ = 0.150
0.219

		1	2
C_a	=	0.378	0.301

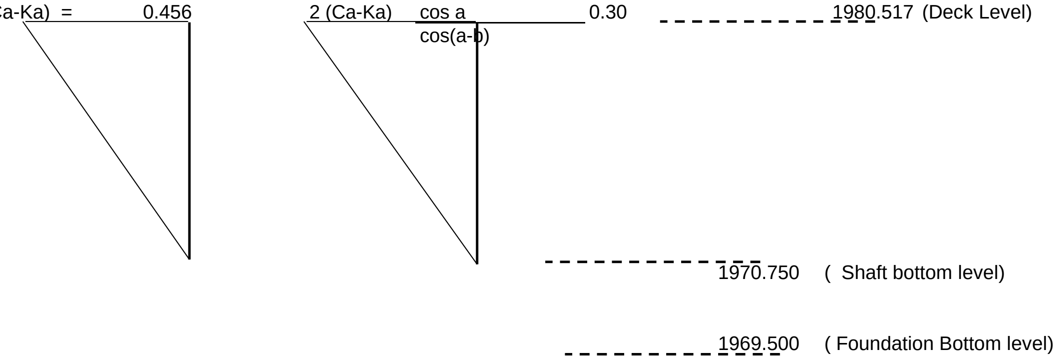
Ca	=	0.378		
Ka	=	**0.226**		
Dynamic Increment	=	0.378	-0.226	**0.152**

3 Earth Pressure : **DRY CASE (Seismic case)**

Equivalent Live Load Surcharge height		=	1.2 m
Assuming	γdry	=	20 kN/m^3
	γwater	=	10.00 kN/m^3

Active Earth Pressure **Surcharge Earth Pressure**

3 (Ca-Ka) = 0.456 2 (Ca-Ka) cos a / cos(a-b) 0.30 1980.517 (Deck Level)



1970.750 (Shaft bottom level)

1969.500 (Foundation Bottom level)

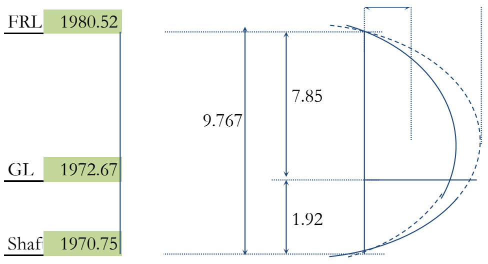
Earth Pressure Diagram for Dynamic Increment

Due to Dynamic Live Load Surcharge

	=	0.304	x	20.00	x	1.2	=	7.297	kN/m^2
at Shaft Bottom Level									
E1	=	0.50	x	7.297	x	9.767	x	12.500	
	=	**445.448**	kN						
M1	=	445.448	x	6.544			=	**2914.965**	kN.m
at Foundation Bottom Level									
E2	=	0.50	x	7.297	x	9.767	x	12.500	
	=	**445.448**	kN						
M2	=	445.448	x	7.794			=	**3471.776**	kN.m

Due to Dynamic Active Earth Pressure

22.27
22.27



Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level

p_mid_height = 22.27 t/m^2
h = 9.767 m
y = 2.9635 m
L = 12.500 m

Parabola below Water Level

p_mid_height = 22.27 t/m^2
h = 9.767 m
y = -2.9635 m
L = 12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	1630.2	5.3
Parabola below Water Level	182.6	1.3
Total Dynamic Earth Pressure	**1812.8**	**4.9**

E3 = **1812.789** kN

E4 = **1812.789** kN

M3 = 1812.79 x 4.88 = **8852.757** kN.m **(Shaft bottom level)**

M4 = 1812.79 x 6.13 = **11118.744** 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)	11048.303	15856.307	2693.302	3426.805
Due to active Earth Pressure (dynamic Increment)	8852.757	11118.744	1812.789	1812.789
Total Earth Pressure	19901.060	26975.051	4506.092	5239.595
Due to Minimum Fluid Pressure	9317.160	13371.805	2861.829	3641.229
Governing of Two	**11048.303**	**26975.051**	**4506.092**	**5239.595**
Due to Live Load Surcharge (Static)	**3231.963**	**4112.166**	**661.813**	**746.513**
Due to Live Load Surcharge(Dynamic)	**2914.965**	**3471.776**	**445.448**	**445.448**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

α h	=	0.180	
α v	=	0.193	
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

$$\lambda = \tan^{-1} \frac{Ws}{Ws-1} \cdot \frac{\alpha\ h}{(1 \pm \alpha\ v)} = \begin{matrix} 0.293 \\ 0.420 \end{matrix}$$

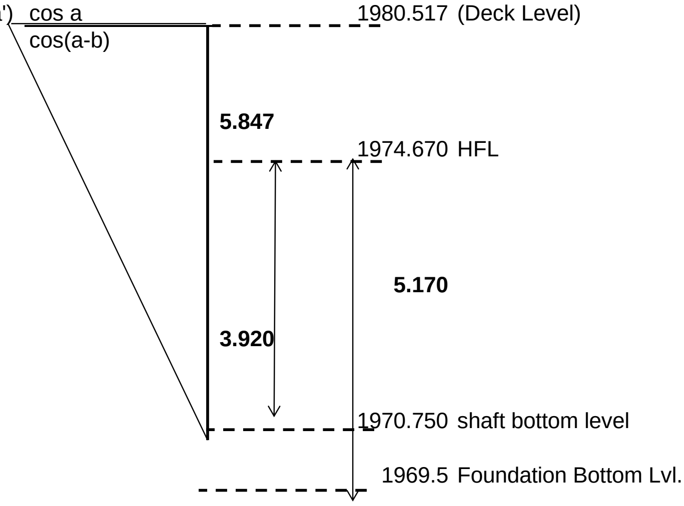
		1	2
C_a'	=	0.551	0.515

Ca'	=	0.551		
Ka'	=	**0.246**		
Dynamic Increment	=	0.551	-0.246	**0.305**

Dynamic Increment	=	0.305
γd	=	20 kN/m^3
γsat	=	10 kN/m^3
γwater	=	10 kN/m^3
Equivalent Live Load Surcharge height	=	1.2 m
Foundation	=	10.00 m

Dynamic Active earth pressure Variation Dynamic Surcharge earth pressure Variation

3 (ca'-ka') 2 (Ca'-Ka') $\frac{\cos a}{\cos(a-b)}$



Earth Pressure Diagram

h'	=	3.920
h	=	9.767

Dynamic active earth pressure

3 (ca-ka)	=	0.456 at deck level
3 (ca'-ka')	=	0.914 at deck level
3 (ca'-ka')h'/h	=	0.367 at HFL level

Dynamic Surcharge earth pressure

2 (Ca'-Ka') $\frac{\cos a}{\cos(a-b)}$	=	0.609 at deck level	Pressure	=	14.62 kN/m^2
	=	0.245 at HFL level	Pressure	=	5.87 kN/m^2

Due to Live Load Surcharge

Dynamic surcharge pressure in dry condition at deck level	=	7.297 kN/m^2
Dynamic surcharge pressure in Submerge condition at HFL level	=	5.87 kN/m^2

at Shaft Bottom Level

F1	=	6.583	x	5.847	x	12.500	=	**481.143 kN**
M1	=	481.14	x	6.84	=	**3292.703 kN.m**		

F2	=	0.50 x	5.869	x	3.920	x	12.500	
	=	**143.791 kN**						
M2	=	143.79	x	2.63	=	**377.653 kN.m**		

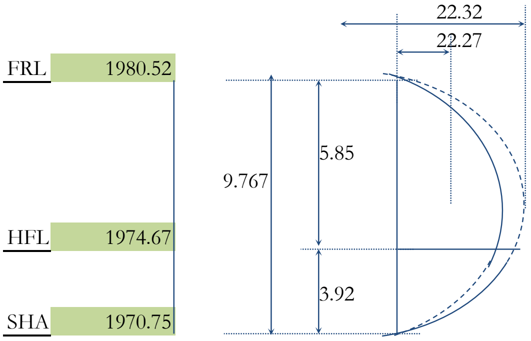
Total Force at Abutment shaft level = 624.934 **kN**
Total Moment at Abutment shaft level = 3670.356 **kN.m**

at Foundation Bottom Level

F3	=	6.583	x	5.847	x	12.500	=	**481.143 kN**	
M3	=	481.14	x	8.09	=	**3894.132 kN.m**			
F4	=	0.500	x	5.869	x	3.920	x	12.500	
	=	**143.791 kN**							
M4	=	143.79	x	3.88	=	**557.392 kN.m**			

Total Force at Founding level = 624.934 **kN**
Total Moment at Founding level = 4451.523 **kN.m**

Due to Dynamic Active Earth Pressure



Dynamic Earth Pressure

Dyanmic Earth Pressure Calculation

Parabola above Water Level			Parabola below Water Level		
p_mid_height	=	22.27 t/m^2	p_mid_height	=	22.32 t/m^2
h	=	9.767 m	h	=	9.767 m
y	=	0.9635 m	y	=	-0.9635 m
L	=	12.500 m	L	=	12.500 m

Dyanmic Earth Pressure	Pa	ey
	T	m
Parabola above Water Level	1171.2	6.2
Parabola below Water Level	642.9	2.5
Total Dynamic Earth Pressure	**1814.1**	**4.9**

Total Force (F1 + F2)	**=**	**1814.053 kN**	**at Shaft Bottom Level**
Total Force (F1 + F2)	**=**	**1814.053 kN**	**at Foundation Bottom Level**

M1	=	1814.05	x	4.88	=	**8855.909 kN.m**
Total Mome	**=**	**8855.909 kN.m**	**at Shaft Bottom**			
M1	=	1814.05	x	6.13	=	**11123.476 kN.m**
Total Mome	**=**	**11123.476 kN.m**	**at Foundation Bottom Level**			

Summary of Moment and Horizontal Force

MOMENTS **HORIZONTAL FORCE**

	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom Lvl kN	At Foundation Bottom Lvl kN
Due to active Earth Pressure(Static	9793.177	13557.017	2702.381	3327.787
Due to active Earth Pressure (Dynamic Increment)	8855.909	11123.476	1814.053	1814.053
Total Earth Pressure	18649.086	24680.493	4516.435	5141.840
Due to Minimum Fluid Pressure	9317.160	13371.805	2861.829	3641.229
Governing of Two	**18649.086**	**24680.493**	**4516.435**	**5141.840**
Due to Live Load Surcharge(Stat	**3526.921**	**4487.455**	**722.212**	**814.642**
Due to Live Load Surcharge (Dynamic Increment)	**3670.356**	**4451.523**	**624.934**	**624.934**
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 - POT -PTFE

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1641.91	1.35	1.35	2216.58	2216.58	
SIDL except wc	=	299.60	1.35	1.35	404.46	404.46	
WC	=	390.00	1.75	1.75	682.50	682.50	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	1110.64	1.5	0.20	1665.96	222.13	(1-70RW + 1-CLASS A)
CWLLmax-Reaction case	=	1890.94	1	0.20	1890.94	378.19	SV Loading
CWLLmin	=	239.36	1.5	0.20	359.04	47.87	(1-70RW + 1-CLASS A)
CWLLmin	=	1709.06	1	0.20	1709.06	341.81	SV Loading
CWLLmax-Transv. Moment Case		843.25	1.5	0.20	1264.87	168.65	(1-70RW)

Braking Force	=	0.2	x	1000	+	0.05	x	554
	=	227.7	KN					

Normal Case:

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)		
DL+SIDL	3303.54	0	165.177	165.177	-165.177	165.177		
DL+SIDL+LL-Max Reaction case	3662.58	341.55	183.129	353.904	158.421	353.904	(1-70RW + 1-CLASS A)	Dry Case
	5371.64	0	268.582	268.582	-268.582	268.582	SV Loading	
DL+SIDL+LL-Min Reaction case	4969.50	341.55	248.475	419.250	93.075	419.250	(1-70RW + 1-CLASS A)	HFL Case
	5194.48	0	259.724	259.724	-259.724	259.724	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	4568.41	341.55	228.421	399.196	113.129	399.196		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	3303.54	1259.02	165.177	794.685	1093.839	1259.017	
DL+SIDL+LL-Max Reaction case		3351.41	1304.56	167.571	819.849	1136.986	1304.557	Dry Case
DL+SIDL+LL-Min Reaction case		3525.67	1304.56	176.283	828.562	1128.273	1304.557	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3472.19	1304.56	173.610	825.888	1130.947	1304.557	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	3303.54	377.705	165.177	354.030	212.528	377.705	
DL+SIDL+LL-Max Reaction case		3351.41	423.245	167.571	379.193	255.674	423.245	Dry Case

DL+SIDL+LL-Min Reaction case		3525.67	423.245	176.283	387.906	246.961	423.245	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3472.19	423.245	173.610	385.232	249.635	423.245	

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	=	1641.91	1.35	1.35	2216.58	2216.58
SIDL except wc	=	299.60	1.35	1.35	404.46	404.46
WC	=	390.00	1.75	1.75	682.50	682.50
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	1110.64	1.5	0.75	1665.96	832.98
CWLLmax-Transv. Moment Case	=	843.25	1.5	0.75	1264.87	632.44
CWLLmin	=	239.36	1.5	0.75	359.04	179.52

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)		
DL+SIDL	3303.54	0.000	165.177	165.177	-165.177	165.177		
DL+SIDL+LL-Max Reaction case	3662.58	341.550	183.129	353.904	158.421	353.904	(1-70RW + 1-CLASS A)	Dry Case
	5371.64	0.000	268.582	268.582	-268.582	268.582	SV Loading	
DL+SIDL+LL-Min Reaction case	4969.50	341.550	248.475	419.250	93.075	419.250	(1-70RW + 1-CLASS A)	HFL Case
	5194.48	0.000	259.724	259.724	-259.724	259.724	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	4568.41	341.550	228.421	399.196	113.129	399.196		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	3303.54	1699.67	165.177	1015.013	1534.495	1699.672	
DL+SIDL+LL-Max Reaction case		3483.06	1745.21	174.153	1046.759	1571.059	1745.212	Dry Case
DL+SIDL+LL-Min Reaction case		4136.52	1745.21	206.826	1079.432	1538.386	1745.212	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3935.98	1745.21	196.799	1069.405	1548.413	1745.212	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	3303.54	509.902	165.177	420.128	344.725	509.902	
DL+SIDL+LL-Max Reaction case		3483.06	555.442	174.153	451.874	381.289	555.442	Dry Case

DL+SIDL+LL-Min Reaction case		4136.52	555.442	206.826	484.547	348.616	555.442	HFL Case
DL+SIDL+LL-Max Transv. Moment case		3935.98	555.442	196.799	474.520	358.643	555.442	

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	=	1641.91	1	1.00	1641.91	1641.91
SIDL except wc	=	299.60	1	1.00	299.60	299.60
WC	=	390.00	1	1.00	390.00	390.00
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax-Reaction case	=	1110.64	1	0.20	1110.64	222.13
CWLLmax-Transv. Moment Case		843.25	1	0.20	843.25	168.65
CWLLmin	=	239.36	1	0.20	239.36	47.87

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)		
DL+SIDL	2331.51	0.000	116.576	116.576	-116.576	116.576		
DL+SIDL+LL-Max Reaction case	2570.87	227.700	128.544	242.394	99.156	242.394	(1-70RW + 1-CLASS A)	Dry Case
	4040.57	0.000	202.029	202.029	-202.029	202.029	SV Loading	
DL+SIDL+LL-Min Reaction case	3442.15	227.700	172.108	285.958	55.592	285.958	(1-70RW + 1-CLASS A)	HFL Case
	4222.45	0.000	211.123	211.123	-211.123	211.123	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	3174.76	227.700	158.738	272.588	68.962	272.588		

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	1133.115	116.576	683.133	1016.539	1133.115	
DL+SIDL+LL-Max Reaction case	2379.38	1178.655	118.969	708.297	1059.686	1178.655	Dry Case
DL+SIDL+LL-Min Reaction case	2553.64	1178.655	127.682	717.009	1050.973	1178.655	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2500.16	1178.655	125.008	714.336	1053.647	1178.655	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	339.934	116.576	286.543	223.359	339.934	
DL+SIDL+LL-Max Reaction case	2379.38	385.474	118.969	311.706	266.505	385.474	Dry Case

DL+SIDL+LL-Min Reaction case	2553.64	385.474	127.682	320.419	257.792	385.474	HFL Case
DL+SIDL+LL-Max Transv. Moment case	2500.16	385.474	125.008	317.745	260.466	385.474	

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	=	1641.91	1.1	1.10	1806.11	1806.11
SIDL except wc	=	299.60	1.1	1.10	329.56	329.56
WC	=	390.00	1.35	1.35	526.50	526.50
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	1110.64	1.5	0.00	1665.96	0.00
CWLLmax-Transv. Moment Case		843.25	1.5	0.00	1264.87	0.00
CWLLmin	=	239.36	1.5	0.00	359.04	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	µ R	Fh/2 +µ R	Fh-µ R	Governing Long. Force (kN)		
DL+SIDL	2662.16	0.000	133.108	133.108	-133.108	133.108		
DL+SIDL+LL-Max Reaction case	3021.20	341.550	151.060	321.835	190.490	321.835	(1-70RW + 1-CLASS A)	Dry Case
	4371.22	0.000	218.561	218.561	-218.561	218.561	SV Loading	
DL+SIDL+LL-Min Reaction case	4328.12	341.550	216.406	387.181	125.144	387.181	(1-70RW + 1-CLASS A)	HFL Case
	4553.10	0.000	227.655	227.655	-227.655	227.655	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	3927.03	341.550	196.352	367.127	145.198	367.127		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	µ R	Fh/2 +µ R	Fh-µ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	2662.16	1699.67	133.108	982.944	1566.564	1699.672	
DL+SIDL+LL-Max Reaction case		2662.16	1699.67	133.108	982.944	1566.564	1699.672	Dry Case
DL+SIDL+LL-Min Reaction case		2662.16	1699.67	133.108	982.944	1566.564	1699.672	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2662.16	1699.67	133.108	982.944	1566.564	1699.672	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	µ R	Fh/2 +µ R	Fh-µ R	Governing Long. Force (kN)	
DL+SIDL	2331.51	2662.16	509.902	133.108	388.059	376.794	509.902	
DL+SIDL+LL-Max Reaction case		2662.16	509.902	133.108	388.059	376.794	509.902	Dry Case

DL+SIDL+LL-Min Reaction case		2662.16	509.902	133.108	388.059	376.794	509.902	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2662.16	509.902	133.108	388.059	376.794	509.902	

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	=	1641.91	1	1	1	1641.91	1641.91	1641.91
SIDL except wc	=	299.60	1	1	1	299.60	299.60	299.60
WC	=	390.00	1.20	1.20	1.20	468.00	468.00	468.00
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	1110.64	1	0.75	0	1110.64	832.98	0.00
CWLLmax-Transv. Moment Case	=	843.25	1	0.75	0	843.25	632.44	0.00
CWLLmin	=	239.36	1	0.75	0	239.36	179.52	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)		
DL+SIDL	2409.51	0.000	120.476	120.476	-120.476	120.476		
DL+SIDL+LL-Max Reaction case	2648.87	227.700	132.444	246.294	95.256	246.294	(1-70RW + 1-CLASS A)	Dry Case
	4118.57	0.000	205.929	205.929	-205.929	205.929	SV Loading	
DL+SIDL+LL-Min Reaction case	3520.15	227.700	176.008	289.858	51.692	289.858	(1-70RW + 1-CLASS A)	HFL Case
	4300.45	0.000	215.023	215.023	-215.023	215.023	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	3252.76	227.700	162.638	276.488	65.062	276.488		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2409.51	0.000	120.476	120.476	-120.476	120.476	
DL+SIDL+LL-Max Reaction case	2589.03	170.775	129.452	214.839	41.323	214.839	Dry Case
DL+SIDL+LL-Min Reaction case	3242.49	170.775	162.125	247.512	8.650	247.512	HFL Case
DL+SIDL+LL-Max Transv. Moment case	3041.95	170.775	152.097	237.485	18.678	237.485	

Normal Case: Quasi Permenant Combination

	Vertical Force (R)	Fh	μ R	Fh/2 +μ R	Fh-μ R	Governing Long. Force (kN)	
DL+SIDL	2409.51	0.000	120.476	120.476	-120.476	120.476	
DL+SIDL+LL-Max Reaction case	2409.51	0.000	120.476	120.476	-120.476	120.476	Dry Case
DL+SIDL+LL-Min Reaction case	2409.51	0.000	120.476	120.476	-120.476	120.476	HFL Case

DL+SIDL+LL-Max Transv. Moment case	2409.51	0.000	120.476	120.476	-120.476	120.476

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

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

			Normal Case		Seismic Case	
Design Speed	V	=	40.00	kmph	40.00	kmph
Live Load	W	=	1110.64	kN	1110.64	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

Possible Load Combination

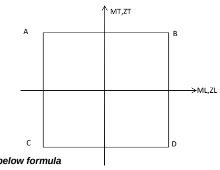
Normal Dry Case	Case 1 : DL+SIDL-Normal Dry Case Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case
Normal HFLCase	Case 3 : DL+SIDL-Normal HFL Case Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case
Longitudinal Seismic Dry Case	Case 5 : DL+SIDL-Long. Seismic Dry Case Case 6 : DL+SIDL+LL-(Maximum 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 = 1969.500 m

Properties of Footing Base:

A	=	125.000	m^2
ZL	=	208.333	m^3
ZT	=	260.417	m^3



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

Moment along longitudinal axis	ML = ML Cos θ + MT Sin θ
Moment along transverse axis	MT = MT Cos θ - ML Sin θ

Case 1 : DL+SIDL-Normal Dry Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure												
Dead Load	1			1641.914	1.150	1888.201	0.000	0.000	0.900	1477.723	-3.850	-5689.232
SIDL except Wearing Course	1			299.598	1.150	344.538	0.000	0.000	0.900	269.638	-3.850	-1038.107
Wearing Course	1			390.000	1.150	448.500	0.000	0.000	1.000	390.000	-3.850	-1501.500
Bearing Pedestal	1	25	1.920	48.000	1.150	55.200	-0.313	-15.000	0.900	43.200	-3.850	-166.320
				2379.512		**2736.439**		**-15.000**		**2180.561**		**-8395.159**
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	1	25	8.025	200.625	0.325	65.203	0.000	0.000	0.900	180.563	-4.675	-844.130
Dirt Wall-Tapered portion	1	25	0.363	9.082	0.325	2.952	0.000	0.000	0.900	8.174	-4.675	-38.213
Bracket - Uniform portion	1	25	1.125	28.125	0.025	0.703	0.000	0.000	0.900	25.313	-4.975	-125.930
Bracket - Tapered portion	1	25	0.563	14.063	0.075	1.055	0.000	0.000	0.900	12.656	-4.925	-62.332
Cap - (uniform portion)	1	25	6.656	166.406	1.063	176.807	0.000	0.000	0.900	149.766	-3.938	-589.702
Cap - (corbel portion)	1	25	5.134	128.353	1.063	136.375	0.000	0.000	0.900	115.518	-3.938	-454.850
Cantilever Return Wall-Rectangle po	1	25	0.000	0.000	0.500	0.000	6.000	0.000	0.900	0.000	-4.500	0.000
Cantilever Return Wall-Trainqle port	1	25	0.000	0.000	0.500	0.000	6.000	0.000	0.900	0.000	-4.500	0.000
RCC Railing or Crash Barrier	1			28.000	0.325	9.100	0.000	0.000	0.900	25.200	-4.675	-117.810
Aproach Slab	1	25	6.563	164.063	0.025	4.102	0.000	0.000	0.900	147.656	-4.975	-734.590
				738.716		**396.296**		**0.000**		**664.845**		**-2967.556**
Substructure & Foundation -Portion 2												
Solid Return wall	1	25	27.891	697.263	-2.250	-1568.841	6.000	4183.575	0.900	627.536	-7.250	-4549.638
Abutment Shaft	1	25	90.998	2274.955	1.192	2711.426	0.000	0.000	0.900	2047.459	-3.808	-7797.014
Back filling over heel slab	1	20	692.321	13846.422	-2.284	-31627.105	0.000	0.000	0.900	12461.780	-7.284	-90773.293
Front Filling over toe slab	1	20	0.000	0.000	3.313	0.000	0.000	0.000	0.900	0.000	-1.687	0.000
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000	0.900	0.000	0.000	0.000
Heel slab	1	25	60.156	1503.906	-1.857	-2792.969	0.000	0.000	0.900	1353.516	-6.857	-9281.250
Toe slab	1	25	35.000	875.000	3.171	2775.000	0.000	0.000	0.900	787.500	-1.829	-1440.000
portion between heel & toe	1	25	20.313	507.813	1.150	583.984	0.000	0.000	0.900	457.031	-3.850	-1759.570
Vertical Components of active earth pressure	1			1508.246	-5.000	-7541.231	0.000	0.000	0.900	1357.422	-10.000	-13574.217
				21585.598		**-37240.642**		**4183.575**		**19427.038**		**-130641.117**
Total				**24703.826**		**-34107.907**		**4168.575**		**22272.444**		**-142003.833**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	116.576	1978.877	1093.129	116.58	1093.13	0.00	0.00
due to Earth pressure	1	3641.229		15856.307	3641.23	15856.31	0.00	0.00
					3757.804	16949.437	0.000	0.000

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		133.108	1978.877	1248.155	133.11	1248.16	0.00	0.00
due to Earth pressure	1.5	5461.843		23784.461	5461.84	23784.46	0.00	0.00
		5594.951		25032.616	5594.951	25032.616	0.000	0.000

Summary of Forces For SBC		
P	24703.826	KN
ML	-17158.471	kNm
MT	4168.575	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Forces from Case :DL+SIDL		24703.826		-34107.907		4168.575		22272.44361		-142003.83
CWLL-Max. Reaction case	1	1110.641	1.150	1277.237	0.950	1055.109	0.000		-3.850	0.00
Vertical Components of LL Surcharge	1	309.216	-5.000	-1546.079	0.000	0.000	0.900		-10.000	-2782.94
Total		**26123.683**		**-34376.749**		**5223.684**		**22272.444**		**-144786.78**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	242.394	1978.877	2272.924	242.39	2272.92	0.00	0.00
due to Earth pressure	1	3641.229		15856.307	3641.23	15856.31	0.00	0.00
due to Live load surcharge	1	746.513		4112.166	746.513	4112.166	0.000	0.000
					4630.14	**22241.40**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		321.835	1978.877	3017.848	321.84	3017.85	0.00	0.00
due to Earth pressure	1.5	5461.843		23784.461	5461.84	23784.46	0.00	0.00
due to Live load surcharge	1.2	895.815		4934.599	895.82	4934.60	0.00	0.00
		6679.494		**31736.908**	**6679.494**	**31736.908**	**0.000**	**0.000**

Summary of Forces For SBC		
P	26123.683	KN
ML	-12135.352	kNm
MT	5223.684	kNm

Case 3 : DL+SIDL-Normal HFL Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure				**2379.512**		**2736.439**		**-15.000**		**2180.561**		**-8395.159**
Substructure & Foundation -Portion 1				**738.716**		**396.296**		**0.000**		**664.845**		**-2967.556**
Substructure & Foundation -Portion 2												
Solid Return wall	1	25	27.891	697.263	-2.250	-1568.841	6.000	4183.575	0.900	627.536	-7.250	-4549.638
Shaft above HFL	1	25	39.088	977.210	1.267	1238.342	0.000	0.000	0.900	879.489	-3.733	-3282.939
Shaft below HFL	1	15	51.910	778.647	1.192	928.037	0.000	0.000	0.900	700.782	-3.808	-2668.677
Back filling above HFL over heel slal	1	20	401.981	8039.625	-2.250	-18089.156	0.000	0.000	0.900	7235.662	-7.250	-52458.553
Back filling below HFL over heel slab	1	10	295.281	2952.813	-2.330	-6880.156	0.000	0.000	0.900	2657.531	-7.330	-19479.797
Front Filling over toe slab	1	10	0.000	0.000	2.072	0.000	0.000	0.000	0.900	0.000	-2.928	0.000
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000	0.900	0.000	0.000	0.000
Heel slab	1	15	60.156	902.344	-1.857	-1675.781	0.000	0.000	0.900	812.109	-6.857	-5568.750
Toe slab	1	15	35.000	525.000	3.171	1665.000	0.000	0.000	0.900	472.500	-1.829	-864.000
Portion between Heel & Toe	1	15	20.313	304.688	1.150	350.391	0.000	0.000	0.900	274.219	-3.850	-1055.742
Vertical Components of active earth pressure	1			1508.246	-5.000	-7541.231	0.000	0.000	0.900	1357.422	-10.000	-13574.217
				16965.769		**-31405.474**		**4183.575**		**15269.192**		**-104610.888**
Total				**20083.997**		**-28272.739**		**4168.575**		**18114.598**		**-115973.604**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	116.576	1978.877	1093.129	116.58	1093.13	0.00	0.00
due to Earth pressure	1	3641.229		13557.017	3641.23	13557.02	0.00	0.00
					3757.80	**14650.15**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		133.108	1978.877	1248.155	133.11	1248.16	0.00	0.00
due to Earth pressure	1.5	5461.843		20335.526	5461.84	20335.53	0.00	0.00
		5594.951		**21583.681**	**5594.95**	**21583.68**	**0.00**	**0.00**

Summary of Forces For SBC		
P	20083.997	KN
ML	-13622.593	kNm
MT	4168.575	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		20083.997		-28272.739		4168.575
CWLL-Min. Reaction case	1	239.359	1.150	275.263	0.950	227.391
Vertical Components of LL Surcharge	1	337.436	-5.000	-1687.179	0.000	0.000
Total		**20660.792**		**-29684.655**		**4395.966**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	18114.598		-115973.604
0.000	0.000	-3.850	0
0.900	303.692	-10.000	-3036.922
	18418.290		**-119010.525**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	285.958	1978.877	2681.425
due to Earth pressure	1	3641.229		13557.017
due to Live load surcharge	1	814.642		4487.455

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
285.96	2681.42	0.00	0.00
3641.23	13557.02	0.00	0.00
814.642	4487.455	0.000	0.000
4741.83	**20725.90**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		387.181	1978.877	3630.598
due to Earth pressure	1.5	5461.843		20335.526
due to live load surcharge	1.2	977.570		5384.946
		6826.595		**29351.070**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
387.18	3630.60	0.00	0.00
5461.84	20335.53	0.00	0.00
977.570	5384.946	0.000	0.000
6826.59	**29351.07**	**0.00**	**0.00**

Summary of Forces For SBC		
P	20660.792	KN
ML	-8958.758	kNm
MT	4395.966	kNm

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

Seismic Effect Factor =	1	αh=	0.243	In Longitudinal direction	Weight of shaft below Ground level = 686.968 KN
		αh=	0.289	In Transverse direction	Weight of back fill below Ground level = 2640.000 KN
		αv=	0.193	In Vertical direction	

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1641.914		142.502	95.002	1.150	1888.201	109.252	1979.927		0.000	0.000	1485.872
SIDL except Wearing Course	1			299.598		26.002	17.335	1.150	344.538	19.935	1980.966		0.000	0.000	298.139
Wearing Course	1			390.000		33.848	22.565	1.150	448.500	25.950	1980.517		0.000	0.000	372.906
Bearing Pedestal	1	25	1.920	48.000		4.166	2.777	1.150	55.200	3.194			-0.313	-15.000	
				2379.512		**206.519**	**137.679**		**2736.439**	**158.331**				**-15.000**	**2156.917**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	8.025	200.625	48.752	17.412	11.608	0.325	65.203	3.773	1979.447	484.935	0.000	0.000	173.200
Dirt Wall-Tapered portion	1	25	0.363	9.082	2.207	0.788	0.525	0.325	2.952	0.171	1978.329	19.484	0.000	0.000	6.959
Bracket - Uniform portion	1	25	1.125	28.125				0.025	0.703						
Bracket - Tapered portion	1	25	0.563	14.063				0.075	1.055						
Cap - (uniform portion)	1	25	6.656	166.406	40.437	14.442	9.628	1.063	176.807	10.230	1978.058	346.068	0.000	0.000	123.602
Cap - (corbel portion)	1	25	5.134	128.353	31.190	11.140	7.427	1.063	136.375	7.891	1977.758	257.573	0.000	0.000	91.995
Cantilever Return Wall-Rectangle pc	1	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1980.517	0.000	6.000	0.000	0.000
Cantilever Return Wall-Traingle port	1	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1980.517	0.000	6.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.325	9.100				0.000	0.000	
Aproach Slab	1	25	6.563	164.063				0.025	4.102				0.000	0.000	
				738.716	**122.585**	**43.783**	**29.189**		**396.296**	**22.064**		**1108.059**		**0.000**	**395.757**
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	27.891	697.263	169.435	60.516	40.344	-2.250	-1568.841	-90.773	1975.482	1013.548	6.000	4183.575	362.001
Abutment Shaft	1	25	90.998	2274.955	385.881	137.822	91.881	1.192	2711.426	109.510	1975.139	2176.030	0.000	0.000	777.195
Back filling over heel slab	1	20	692.321	13846.422	0.000	0.000	0.000	-2.284	-31627.105	0.000	1975.482	0.000	0.000	0.000	0.000
Front Filling over toe	1	20	0.000	0.000				3.313	0.000				0.000	0.000	
Side filling between heel and toe	1	20	0.000	0.000				0.000	0.000				0.000	0.000	
Heel slab	1	25	60.156	1503.906				-1.857	-2792.969				0.000	0.000	
Toe slab	1	25	35.000	875.000				3.171	2775.000				0.000	0.000	
portion between heel & toe	1	25	20.313	507.813				1.150	583.984				0.000	0.000	
Vertical component of active earth pressure	1			1419.429				-5.000	-7097.146				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			750.882				-5.000	-3754.410				0.000	0.000	
				22247.663	**555.316**	**198.338**	**132.225**		**-40550.966**	**18.736**		**3189.579**		**4183.575**	**1139.196**
Total =				**25365.891**	**677.901**	**448.639**	**299.093**		**-37418.232**	**199.131**		**4297.638**		**4168.575**	**3691.870**
							-299.093			-199.131					

Vertical Forces For Restoring or Resisting Effect / **For Overturning or Sliding Effect**

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv	Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure												
Dead Load	0.9			1477.723	85.501	-3.850	-5689.232	-329.180				
SIDL except Wearing Course	0.9			269.638	15.601	-3.850	-1038.107	-60.065				
Wearing Course	1.00			390.000	22.565	-3.850	-1501.500	-86.877				
Bearing Pedestal	0.9	25	1.92	43.200		-3.850	-166.320					
				2180.561	**123.668**		**-8395.159**	**-476.123**				
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	0.9	25	8.025	180.563	10.447	-4.675	-844.130	-48.842	1.0	48.752	1979.447	484.935
Dirt Wall-Tapered portion	0.9	25	0.363	8.174	0.473	-4.675	-38.213	-2.211	1.0	2.207	1978.329	19.484
Bracket - Uniform portion	0.9	25	1.125	25.313								
Bracket - Tapered portion	0.9	25	0.563	12.656								
Cap - (uniform portion)	0.9	25	6.656	149.766	8.665	-3.938	-589.702	-34.120	1.0	40.437	1978.058	346.068
Cap - (corbel portion)	0.9	25	5.134	115.518	6.684	-3.938	-454.850	-26.318	1.0	31.190	1977.758	257.573
Cantilever Return Wall-Rectangle pc	0.9	25	0.000	0.000	0.000	-4.500	0.000	0.000	1.0	0.000	1980.517	0.000
Cantilever Return Wall-Traingle port	0.9	25	0.000	0.000	0.000	-4.500	0.000	0.000	1.0	0.000	1980.517	0.000
RCC Railing or Crash Barrier	0.9			25.200		-4.675	-117.810					
Aproach Slab	0.9	25	6.563	147.656		-4.975	-734.590					
				664.845	**26.270**		**-2779.295**	**-111.491**		**122.585**		**1108.059**
Substructure & Foundation -Portion 2												
Abutment Shaft	0.9	25	90.998	2047.459	78.718	-3.808	-7797.014	-299.771	1.0	367.333	1975.139	2071.435
Solid Return wall	0.9	25	27.891	627.536	36.309	-7.250	-4549.638	-263.243	1.0	169.435	1975.482	1013.548
Back filling over heel slab	0.9	20	692.321	12461.780	0.000	-7.284	-90773.3	0.000	1.0	0.000	1975.482	0.000
Front Filling over toe	0.9	20	0.000	0.000		-1.687	0.000					
Side filling between heel and toe	0.9	20	0.000	0.000		0.000	0.000					
Heel slab	0.9	25	60.156	1353.516		-6.857	-9281.250					
Toe slab	0.9	25	35.000	787.500		-1.829	-1440.000					
portion between heel & toe	0.9	25	20.313	457.031		-3.850	-1759.570					
Vertical component of active earth pressure	0.9			1357.422		-10.000	-13574.217					
Vertical component of dynamic increment of earth pressure	0.9			675.794		-10.000	-6757.938					
				20102.832	**115.028**		**-137399.05**	**-563.014**		**536.768**		**3084.983**
Total =				**22948.237**	**264.966**		**-148573.51**	**-1150.627**		**659.353**		**4193.043**
					-264.966			1150.627				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	1133.115	206.519	1978.877	10625.218	2156.917
due to Substructure	1	677.901	242.120		4297.638	1534.953
due to Earth pressure	1	5239.595			26975.051	

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
1133.11	10625.22	0.00	0.00	0.00	0.00	206.52	2156.92
677.90	4297.64	0.00	0.00	0.00	0.00	242.12	1534.95
5239.59	26975.05						
7050.61	**41897.91**	**0.00**	**0.00**	**0.00**	**0.00**	**448.64**	**3691.87**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1699.672	1978.877	15937.83
due to Substructure	1.5	989.029		6289.56
due to Active Earth pressure	1	3426.805		15856.31
due to dynamic Earth pressure	1.5	2719.184		16678.12
		8834.691		**54761.814**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1699.67	15937.83	0.00	0.00
989.03	6289.56	0.00	0.00
3426.805	15856.307	0.000	0.000
2719.184	16678.116	0.000	0.000
8834.69	**54761.81**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	25664.984	25066.798	KN
ML	4678.806	4280.544	kNm
MT	7860.445	7860.445	kNm

Summary of Restoring Forces

Vertical Load	22683.272	kN
Moment	-149724.136	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 = 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 Superstucture				2379.512		206.519	137.679		2736.439	158.331				-15.000	2156.917
Forces from Substructure				22986.379	677.901	242.120	161.414		-40154.670	40.800		4297.638		4183.575	1534.953
CWLL-Max. Reaction case	0.20			222.13		19.279	12.852	1.150	255.447	14.780	1981.717		0.950	211.022	235.526
Vertical component of LL Surcharge	0.20			61.84				-5.000	-309.216				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			36.90				-5.000	-184.511				0.000	0.000	
Total =				**25686.764**	**677.901**	**467.918**	**311.945**		**-37656.511**	**213.912**		**4297.638**		**4379.597**	**3927.396**
							-311.945			-213.912					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstucture				2180.561	123.668	0.000	-8395.16	-476.123
Forces from Substructure				20767.677	141.297	0.000	-140178.35	-674.504
CWLL-Max. Reaction case	0.00			0.00	0.000	-3.850	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-10.000	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-10.000	0.00	
Total =				**22948.237**	**264.966**		**-148573.51**	**-1150.627**
					-264.966			**1150.627**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	1178.655	225.797	1978.877	11052.247	2392.443
due to Substructure	1	677.901	242.120		4297.638	1534.953
due to Earth pressure	1	5239.595			26975.051	
due to Live load surcharge	0.20	238.392			1516.788	

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
1178.65	11052.25	0.00	0.00	0.00	0.00	225.80	2392.44
677.90	4297.64	0.00	0.00	0.00	0.00	242.12	1534.95
5239.59	26975.05						
238.39	1516.79						
7334.54	**43841.72**	**0.00**	**0.00**	**0.00**	**0.00**	**467.92**	**3927.40**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1699.672	1978.877	15937.827
due to Substructure	1.5	989.029		6289.564
due to Active Earth pressure	1	3426.805		15856.307
due to dynamic Earth pressure	1.5	2719.184		16678.116
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		8834.691		**54761.814**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1699.67	15937.83	0.00	0.00
989.03	6289.56	0.00	0.00
3426.805	15856.307		
2719.184	16678.116		
0.000	0.000		
0.000	0.000		
8834.69	**54761.81**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	25998.710	25374.819	KN
ML	6399.125	5971.301	kNm
MT	8306.993	8306.993	kNm

Summary of Restoring Forces

Vertical Load	22683.272	kN
Moment	-149724.136	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 αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**2379.512**		**206.519**	**137.679**		**2736.439**	**158.331**				**-15.000**	**2156.917**
Substructure & Foundation -Portion 1				**738.716**	**122.585**	**43.783**	**29.189**		**396.296**	**22.064**		**1108.059**		**0.000**	**395.757**
Substructure & Foundation -Portion 2															
Solid Return wall	1		27.89	697.26	169.435	60.516	40.344	-2.25	-1568.841	-90.773	1975.48	1013.548	6.00	4183.575	362.001
Shaft above HFL	1	25	39.088	977.210	237.462	84.812	56.542	1.267	1238.342	71.651	1976.139	1576.541	0.000	0.000	563.080
Shaft below HFL	1	15	51.910	778.647	148.419	53.010	35.340	1.192	928.037	42.120	1973.670	618.906	0.000	0.000	221.050
Back filling above HFL over heel slal	1	20	401.981	8039.625	0.000	0.000	0.000	-2.250	-18089.156	0.000	1977.594	0.00	0.000	0.000	0.00
Back filling below HFL over heel slal	1	10	295.281	2952.813	0.000	0.000	0.000	-2.330	-6880.156	0.000	1972.398	0.00	0.000	0.000	0.00
Front Filling over toe	1	10	0.000	0.000				2.072	0.000				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	60.156	902.344				-1.857	-1675.781				0.000	0.000	
Toe slab	1	15	35.000	525.000				3.171	1665.000				0.000	0.000	
portion between heel & toe	1	15	20.313	304.688				1.150	350.391				0.000	0.000	
Vertical component of active earth pressure	1			1378.414				-5.000	-6892.072				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			751.406				-5.000	-3757.028				0.000	0.000	
				17587.343	**623.340**	**222.633**	**148.422**		**-34681.264**	**22.997**		**3576.914**		**4183.575**	**1277.538**
Total =				**20705.571**	**745.925**	**472.935**	**315.290**		**-31548.529**	**203.392**		**4684.973**		**4168.575**	**3830.211**
							-315.290			-203.392					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**2180.561**	**123.668**		**-8395.159**	**-476.123**
Substructure & Foundation -Portion 1				**664.845**	**26.270**		**-2779.295**	**-111.491**
Substructure & Foundation -Portion 2								
Solid Return wall	0.9	25	27.8905	627.536	36.309	-7.25	-4549.638	-263.243
Shaft above HFL	0.9	25	39.088	879.489	50.887	-3.733	-3282.939	-189.952
Shaft below HFL	0.9	15	51.910	700.782	27.831	-3.808	-2668.677	-105.984
Back filling above HFL over heel slal	0.9	20	401.981	7235.662	0.000	-7.250	-52458.553	0.000
Back filling below HFL over heel slal	0.9	10	295.281	2657.531	0.000	-7.330	-19479.797	0.000
Front Filling overtoe	0.9	10	0.000	0.000		-2.928	0.000	
Side filling between heel and toe	0.9	10	0.000	0.000		0.000	0.000	
Heel slab	0.9	15	60.156	812.109		-6.857	-5568.750	
Toe slab	0.9	15	35.000	472.500		-1.829	-864.000	
portion between heel & toe	0.9	15	20.313	274.219		-3.850	-1055.742	
Vertical component of active earth pressure	0.9			1240.573		-10.000	-12405.729	
Vertical component of dynamic increment of earth pressure	0.9			676.265		-10.000	-6762.650	
				15828.608	**129.605**		**-110205.05**	**-623.321**
Total =				**18674.014**	**279.543**		**-121379.50**	**-1210.934**
					-279.543			1210.934

For Overturning or Sliding Effect

Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
	122.585		**1108.059**
1.0	169.435	1975.482	1013.548
1.0	237.462	1976.139	1576.541
1.0	129.871	1973.670	541.560
1.0	0.000	1977.594	0.000
1.0	0.000	1972.398	0.000
	536.768		**3131.649**
	659.353		**4239.709**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	1133.115	206.519	1978.877	10625.218	2156.917
due to Substructure	1	745.925	266.416		4684.973	1673.294
due to Earth pressure	1	5141.840			24680.493	

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
1133.11	10625.22	0.00	0.00	0.00	0.00	206.52	2156.92
745.92	4684.97	0.00	0.00	0.00	0.00	266.42	1673.29
5141.84	24680.49						
7020.88	**39990.68**	**0.00**	**0.00**	**0.00**	**0.00**	**472.93**	**3830.21**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1699.672	1978.877	15937.827
due to Substructure	1.5	989.029		6359.563
due to Active Earth pressure	1	3327.787		13557.017
due to dynamic Earth pressure	1.5	2721.080		16685.214
		8737.568		**52539.621**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1699.67	15937.83	0.00	0.00
989.03	6359.56	0.00	0.00
3327.787	13557.017		
2721.080	16685.214		
8737.57	**52539.62**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	21020.861	20390.281	KN
ML	8645.547	8238.763	kNm
MT	7998.786	7998.786	kNm

Summary of Restoring Forces

Vertical Load	18394.471	kN
Moment	-122590.438	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				2379.512		206.519	137.679		2736.439	158.331				-15.000	2156.917

Forces from Substructure				18326.059	745.925	266.416	177.611		-34284.968	45.061		4684.973		4183.575	1673.294
CWLL-Max. Reaction case	0.20			47.87		4.155	2.770	1.150	55.053	3.185	1981.717		0.950	45.478	50.759
Vertical component of LL Surcharge	0.20			67.49				-5.000	-337.436				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			51.77				-5.000	-258.856				0.000	0.000	
Total =				**20872.701**	**745.925**	**477.090**	**318.060**		**-32089.769**	**206.578**		**4684.973**		**4214.053**	**3880.970**
							-318.060			-206.578					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				2180.561	123.668		-8395.159	-476.123
Forces from Substructure				16493.453	155.875		-112984.34	-734.812
CWLL-Max. Reaction case	0.00			0.00	0.00		-3.85	0.00
Vertical component of LL Surcharge	0.00			0.00		-3.850	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-10.000	0.000	0.000
Total =				**18674.014**	**279.543**		**-121383.35**	**-1210.934**
					-279.543			**1210.934**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	1178.655	210.674	1978.877	11052.247	2207.676
due to Substructure	1	745.925	266.416		4684.973	1673.294
due to Earth pressure	1	5141.840			24680.493	
due to Live load surcharge	0.20	287.915			1787.796	

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
1178.65	11052.25	0.00	0.00	0.00	0.00	210.67	2207.68
745.92	4684.97	0.00	0.00	0.00	0.00	266.42	1673.29
5141.84	24680.49						
287.92	1787.80						
7354.34	**42205.51**	**0.00**	**0.00**	**0.00**	**0.00**	**477.09**	**3880.97**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1699.672	1978.877	15937.827
due to Substructure	1.5	989.029		6359.5629
due to Active Earth pressure	1	3327.787		13557.017
due to dynamic Earth pressure	1.5	2721.080		16685.214
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		8737.568		**52539.621**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1699.67	15937.83	0.00	0.00
989.03	6359.56	0.00	0.00
3327.787	13557.017		
2721.080	16685.214		
0.000	0.000		
0.000	0.000		
8737.57	**52539.62**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	21190.761	20554.641	KN
ML	10322.318	9909.162	kNm
MT	8095.024	8095.024	kNm

Summary of Restoring Forces

Vertical Load	18394.471	kN
Moment	-122594.288	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = 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				**2379.512**		**688.396**	**137.679**		**2736.439**	**158.331**				**-15.000**	**7189.723**
Substructure & Foundation -Portion 1				**738.716**	**36.776**	**145.943**	**29.189**		**396.296**	**22.064**		**332.418**		**0.000**	**1319.189**
Substructure & Foundation -Portion 2				**22247.663**	**166.595**	**661.125**	**132.225**		**-40550.966**	**18.736**		**956.874**		**4183.575**	**3797.321**
Total =				**25365.891**	**203.370**	**1495.464**	**299.093**		**-37418.232**	**199.131**		**1289.291**		**4168.575**	**12306.232**
							-299.093			-199.131					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**2180.561**	**123.668**		**-8395.16**	**-476.123**
Substructure & Foundation -Portion 1				**664.845**	**26.270**		**-2779.29**	**-111.491**
Substructure & Foundation -Portion 2				**20102.832**	**115.028**		**-137399.05**	**-563.014**
Total =				**22948.237**	**264.966**		**-148573.51**	**-1150.627**
					-264.966			1150.627

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x αh x P (kN)	C.g. of Force (m)	MLs due to FL
	122.585		**1108.059**
	536.768		**3084.983**
	659.353		**4193.043**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	339.934	688.396	1978.877	3187.565	7189.723
due to Substructure	1	203.370	807.068		1289.291	5116.509
due to Earth pressure	1	5239.595			26975.051	

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
339.93	3187.57	0.00	0.00	0.00	0.00	688.40	7189.72
203.37	1289.29	0.00	0.00	0.00	0.00	807.07	5116.51
5239.59	26975.05						
5782.90	**31451.91**	**0.00**	**0.00**	**0.00**	**0.00**	**1495.46**	**12306.23**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		509.902	1978.877	4781.348
due to Substructure	1.5	989.029		6289.5639
due to Active Earth pressure	1	3426.805		15856.307
due to dynamic Earth pressure	1.5	815.755		5003.4347
		5741.491		**31930.654**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
509.90	4781.35	0.00	0.00
989.03	6289.56	0.00	0.00
3426.805	15856.307		
815.755	5003.435		
5741.49	**31930.65**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	25664.984	25066.798	KN
ML	-5767.193	-6165.455	kNm
MT	16474.807	16474.807	kNm

Summary of Restoring Forces

Vertical Load	22683.272	kN
Moment	-149724.136	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**25686.764**	**311.945**	**-37656.51**	**213.912**	**4379.597**
		-311.945		-213.912	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL2	MLs due to Fv
Total =	**22948.237**	**264.966**	**-148573.51**	**-1150.627**
		-264.966		**1150.627**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	385.474	752.658	1978.877	3614.594	7974.811
due to Substructure	1	203.370	807.068		1289.291	5116.509
due to Earth pressure	1	5239.595			26975.051	
due to Live load surcharge	0.2	238.392			1516.788	

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
385.47	3614.59	0.00	0.00	0.00	0.00	752.66	7974.81
203.37	1289.29	0.00	0.00	0.00	0.00	807.07	5116.51
5239.59	26975.05						
238.39	1516.79						
6066.83	**33395.72**	**0.00**	**0.00**	**0.00**	**0.00**	**1559.73**	**13091.32**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		509.90	1978.877	4781.348
due to Substructure	1.5	296.71		1886.869
due to Active Earth pressure	1	3426.81		15856.307
due to dynamic Earth pressure	1.5	815.76		5003.435
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		5049.17		**27527.959**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
509.90	4781.35	0.00	0.00
296.71	1886.87	0.00	0.00
3426.805	15856.307		
815.755	5003.435		
0.000	0.000		
0.000	0.000		
5049.17	**27527.96**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	25998.710	25374.819	KN
ML	-4046.875	-4474.698	kNm
MT	17470.917	17470.917	kNm

Summary of Restoring Forces

Vertical Load	22683.272	kN
Moment	-149724.136	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT

Vertical Forces For Restoring or Resisting Effect

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv

Superstructure		**2379.512**	**137.679**	**2736.439**	**158.331**	**-15.000**
Substructure & Foundation -Portion 1		**738.716**	**29.189**	**396.296**	**22.064**	**0.000**
Substructure & Foundation -Portion 2		**17587.343**	**148.422**	**-34681.264**	**22.997**	**4183.575**
Total =		**20705.571**	**315.290**	**-31548.529**	**203.392**	**4168.575**
			-315.290		-203.392	

Superstructure	**2180.561**	**123.668**		**-8395.159**	**-476.123**
Substructure & Foundatio	**664.845**	**26.270**		**-2779.295**	**-111.491**
Substructure & Foundatio	**15828.608**	**129.605**		**-110205.050**	**-623.321**
Total =	**18674.014**	**279.543**		**-121379.504**	**-1210.934**
		-279.543			1210.934

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	339.93	688.40	1978.88	3187.57	7189.72
due to Substructure	1	223.78	888.05		1405.49	5577.65
due to Earth pressure	1	5141.84			24680.49	

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
339.93	3187.57	0.00	0.00	0.00	0.00	688.40	7189.72
223.78	1405.49	0.00	0.00	0.00	0.00	888.05	5577.65
5141.84	24680.49						
5705.55	**29273.55**	**0.00**	**0.00**	**0.00**	**0.00**	**1576.45**	**12767.37**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		509.902	1978.877	4781.35
due to Substructure	1.5	296.709		1907.87
due to Active Earth pressure	1	3327.787		13557.02
due to dynamic Earth pressure	1.5	816.324		5005.56
		4950.721		**25251.80**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
509.90	4781.35	0.00	0.00
296.71	1907.87	0.00	0.00
3327.787	13557.017		
816.324	5005.564		
4950.72	**25251.80**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	21020.861	20390.281	KN
ML	-2071.586	-2478.371	kNm
MT	16935.945	16935.945	kNm

Summary of Restoring Forces

Vertical Load	18394.471	kN
Moment	-122590.438	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**20872.701**	**318.060**	**-32089.77**	**206.578**	**4214.053**
		-318.060		-206.578	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Total =	**18674.014**	**279.543**		**-121383.35**	**-1210.934**
		-279.543			**1210.934**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	385.474	702.24506	1978.877	3614.594	7358.9208
due to Substructure	1	223.777	888.05346		1405.492	5577.6474
due to Earth pressure	1	5141.840			24680.493	
due to Live load surcharge	0.2	287.915			1787.796	

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
385.47	3614.59	0.00	0.00	0.00	0.00	702.25	7358.92
223.78	1405.49	0.00	0.00	0.00	0.00	888.05	5577.65
5141.84	24680.49						
287.92	1787.80						
6039.01	**31488.37**	**0.00**	**0.00**	**0.00**	**0.00**	**1590.30**	**12936.57**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		509.90	1978.877	4781.35
due to Substructure	1.5	296.71		1907.87
due to Active Earth pressure	1	3327.79		13557.02
due to dynamic Earth pressure	1.5	816.32		5005.56
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		4950.72		**25251.80**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
509.90	4781.35	0.00	0.00
296.71	1907.87	0.00	0.00
3327.787	13557.017		
816.324	5005.564		
0.000	0.000		
0.000	0.000		
4950.72	**25251.80**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	21190.761	20554.641	KN
ML	-394.816	-807.972	kNm
MT	17150.621	17150.621	kNm

Summary of Restoring Forces

Vertical Load	18394.471	kN
Moment	-122594.288	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 (1981.717 - 1969.500) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.20 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1981.717 - 1969.500) = 0.000 kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= μP + c.A + Fp	FOS	Overturning moment	Restoring Moment = ΣP.eToe+Mp	FOS
Normal Dry Case	kN	kNm	kNm	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	24703.826	-17158.471	4168.575	295.999	131.277	263.984	99.263	295.999	99.263	5594.951	15590.711	2.79	25032.616	142003.83	5.67
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	26123.683	-12135.352	5223.684	287.298	170.799	247.180	130.681	287.298	130.681	6679.494	15590.711	2.33	31736.908	144786.78	4.56
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	20083.997	-13622.593	4168.575	242.068	111.291	210.053	79.276	242.068	79.276	5594.951	12680.218	2.27	21583.681	115973.60	5.37
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	20660.792	-8958.758	4395.966	225.169	139.165	191.408	105.404	225.169	105.404	6826.595	12892.803	1.89	29351.070	119010.53	4.05
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	25664.984	4678.806	7860.445	213.046	257.962	152.677	197.594	257.962	152.677	8834.691	15878.290	1.80	54761.814	149724.14	2.73
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	25998.710	6399.125	8306.993	209.173	270.604	145.375	206.807	270.604	145.375	8834.691	15878.290	1.80	54761.814	149724.14	2.73
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	21020.861	8645.547	7998.786	157.384	240.381	95.953	178.950	240.381	95.953	8737.568	12876.130	1.47	52539.621	122590.44	2.33
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	21190.761	10322.318	8095.024	151.064	250.158	88.894	187.988	250.158	88.894	8737.568	12876.130	1.47	52539.621	122594.29	2.33
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	25664.984	-5767.193	16474.807	296.266	240.901	169.739	114.374	296.266	114.374	5741.491	15878.290	2.77	31930.654	149724.14	4.69
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	25998.710	-4046.875	17470.917	294.503	255.653	160.326	121.476	294.503	121.476	5049.171	15878.290	3.14	27527.959	149724.14	5.44
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	21020.861	-2071.586	16935.945	243.145	223.257	113.076	93.189	243.145	93.189	4950.721	12876.130	2.60	25251.798	122590.44	4.85

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	21190.761	-394.816	17150.621	237.280	233.489	105.563	101.773	237.280	101.773	4950.721	12876.130	2.60	25251.798	122594.29	4.85
								SAFE	SAFE			SAFE			SAFE

DESIGN OF FOUNDATION

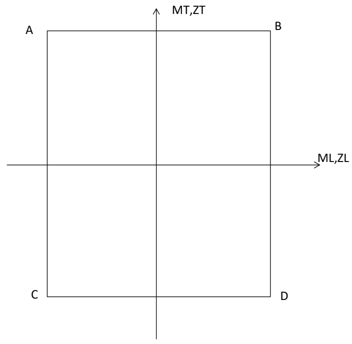
Foundation Lvl = 1969.500 m

Properties of Footing Base:

A	=	125.000	m^2
ZL	=	208.333	m^3
ZT	=	260.417	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			2216.584	1.150	2549.072	0.000	0.000
SIDL except Wearing Course	1.35			404.457	1.150	465.126	0.000	0.000
Wearing Course	1.75			682.500	1.150	784.875	0.000	0.000
Bearing Pedestal	1.35	25	1.920	64.800	1.15	74.520	-0.313	-20.250
				3368.341		**3873.592**		**-20.250**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	8.025	270.844	0.325	88.024	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.363	12.261	0.325	3.985	0.000	0.000
Bracket - Uniform portion	1.35	25	1.125	37.969	0.025	0.949	0.000	0.000
Bracket - Tapered portion	1.35	25	0.563	18.984	0.075	1.424	0.000	0.000
Cap - (uniform portion)	1.35	25	6.656	224.648	1.063	238.689	0.000	0.000
Cap - (corbel portion)	1.35	25	5.134	173.276	1.063	184.106	0.000	0.000
Cantilever Return Wall-Rectangle po	1.35	25	0.000	0.000	0.500	0.000	6.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	0.000	0.000	0.500	0.000	6.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.35			37.800	0.325	12.285	0.000	0.000
Aproach Slab	1.35	25	6.563	221.484	0.025	5.537	0.000	0.000
				997.267		**534.999**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	27.891	941.304	-2.250	-2117.935	6.000	5647.826
Abutment Shaft	1.35	25	90.998	3071.189	1.192	3660.425	0.000	0.000
Back filling over heel slab	1.35	20	692.321	18692.670	-2.284	-42696.592	0.000	0.000
Front Filling over toe slab	1.35	20	91.800	2478.600	3.313	8211.240	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	60.156	2030.273	-1.857	-3770.508	0.000	0.000
Toe slab	1.35	25	35.000	1181.250	3.171	3746.250	0.000	0.000
portion between heel & toe	1.35	25	20.313	685.547	1.150	788.379	0.000	0.000
Vertical Components of active earth pressure	1.5			2262.369	-5.000	-11311.847	0.000	0.000
				31845.394		**-43194.812**		**5647.826**
Total				**36211.002**		**-38786.220**		**5627.576**

Summary of Forces About C.G. OF Footing

P	36211.002	KN
ML	-13452.894	kNm
MT	5627.576	kNm

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		165.177	1978.877	1548.865
due to Earth pressure	1.5	5461.843		23784.461

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
165.18	1548.87	0.00	0.00
5461.84	23784.46	0.00	0.00
5627.020	**25333.326**	**0.000**	**0.000**

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		36211.002		-38786.220		5627.576
CWLL-Max. Reaction case	1.5	1665.961	1.150	1915.855	0.950	1582.663
Vertical Components of LL Surcharge	1.2	371.059	-5.000	-1855.295	0.000	0.000
Total		**38248.022**		**-38725.659**		**7210.239**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		353.904	1978.877	3318.558
due to Earth pressure	1.5	5461.843		23784.461
due to Live load surcharge	1.2	895.815		4934.599
		6711.562		**32037.618**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
353.90	3318.56	0.00	0.00
5461.84	23784.46		
895.82	4934.60		
6711.562	**32037.618**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	38248.022	KN
ML	-6688.041	kNm
MT	7210.239	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				**3368.341**		**3873.592**		**-20.250**
Substructure & Foundation -Portion 1				**997.267**		**534.999**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	27.891	941.304	-2.250	-2117.935	6.000	5647.826
Shaft above HFL	1.35	25	39.088	1319.234	1.267	1671.761	0.000	0.000
Shaft below HFL	1.35	15	51.910	1051.173	1.192	1252.850	0.000	0.000
Back filling above HFL over heel slab	1.35	20	401.981	10853.494	-2.250	-24420.361	0.000	0.000
Back filling below HFL over heel slab	1.35	10	295.281	3986.297	-2.330	-9288.211	0.000	0.000
Front Filling over toe slab	1.35	10	0.000	0.000	2.072	0.000	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	60.156	1218.164	-1.857	-2262.305	0.000	0.000
Toe slab	1.35	15	35.000	708.750	3.171	2247.750	0.000	0.000
Portion between Heel & Toe	1.35	15	20.313	411.328	1.150	473.027	0.000	0.000
Vertical Components of active earth pressure	1.5			2262.369	-5.000	-11311.847	0.000	0.000
				23130.025		**-43528.575**		**5647.826**
Total				**27495.633**		**-39119.983**		**5627.576**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		165.177	1978.877	1548.865
due to Earth pressure	1.5	5461.843		20335.526

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
165.18	1548.87	0.00	0.00
5461.84	20335.53		
5627.020	**21884.391**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	27495.633	KN
ML	-17235.592	kNm
MT	5627.576	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		27495.633		-39119.983		5627.576
CWLL-Min. Reaction case	1.5	359.039	1.150	412.895	0.950	341.087
Vertical Components of LL Surcharge	1.2	404.923	-5.000	-2024.614	0.000	0.000
Total		**28259.595**		**-40731.703**		**5968.663**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		419.250	1978.877	3931.308
due to Earth pressure	1.5	5461.843		20335.526
due to Live load surcharge	1.2	977.570		5384.946

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
419.25	3931.31	0.00	0.00
5461.84	20335.53		
977.57	5384.95		
6858.663	**29651.780**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	28259.595	KN
ML	-11079.923	kNm
MT	5968.663	kNm

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

Seismic Effect Factor = 1.50 αh= 0.243 In Longitudinal direction Weight of shaft below Ground level = 686.968 KN
αh= 0.289 In Transverse direction Weight of back fill below Ground level = 2640.000 KN
αv= 0.193 In Vertical direction

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 = 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			2216.584		213.753	142.502	1.150	2549.072	163.878	1979.927		0.000	0.000	2228.807
SIDL except Wearing Course	1.35			404.457		39.003	26.002	1.150	465.126	29.903	1980.966		0.000	0.000	447.209
Wearing Course	1.75			682.500		50.772	33.848	1.150	784.875	38.925	1980.517		0.000	0.000	559.359
Bearing Pedestal	1.35	25	1.92	64.800		6.249	4.166	1.150	74.520	4.791			-0.313	-20.250	
				3368.341		**309.778**	**206.519**		**3873.592**	**237.497**				**-20.250**	**3235.375**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	8.025	270.844	73.128	26.118	17.412	0.325	88.024	5.659	1979.447	727.402	0.000	0.000	259.800
Dirt Wall-Tapered portion	1.35	25	0.363	12.261	3.310	1.182	0.788	0.325	3.985	0.256	1978.329	29.226	0.000	0.000	10.438
Bracket - Uniform portion	1.35	25	1.125	37.969				0.025	0.949						
Bracket - Tapered portion	1.35	25	0.563	18.984				0.075	1.424						
Cap - (uniform portion)	1.35	25	6.656	224.648	60.655	21.664	14.442	1.063	238.689	15.345	1978.058	519.101	0.000	0.000	185.403
Cap - (corbel portion)	1.35	25	5.134	173.276	46.785	16.710	11.140	1.063	184.106	11.836	1977.758	386.359	0.000	0.000	137.993
Cantilever Return Wall-Rectangle po	1.35	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1980.517	0.000	6.000	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1980.517	0.000	6.000	0.000	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.325	12.285				0.000	0.000	
Aproach Slab	1.35	25	6.563	221.484				0.025	5.537				0.000	0.000	
				997.267	**183.878**	**65.674**	**43.783**		**534.999**	**33.096**		**1662.089**		**0.000**	**593.635**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	27.891	941.304	254.152	90.773	60.516	-2.250	-2117.935	-136.160	1975.482	1520.322	6.000	5647.826	543.001
Abutment Shaft	1.35	25	90.998	3071.189	643.740	229.919	153.280	1.192	3660.425	182.688	1975.139	3630.129	0.000	0.000	1296.544
Back filling over heel slab	1.35	20	692.321	18692.670	0.000	0.000	0.000	-2.284	-42696.592	0.000	1975.482	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	91.800	2478.600				3.313	8211.240				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	60.156	2030.273				-1.857	-3770.508				0.000	0.000	
Toe slab	1.35	25	35.000	1181.250				3.171	3746.250				0.000	0.000	
portion between heel & toe	1.35	25	20.313	685.547				1.150	788.379				0.000	0.000	
Vertical component of active earth pressure	1.00			1419.429				-5.000	-7097.146						
Vertical component of dynamic increment of earth pressure	1.50			1126.323				-5.000	-5631.615						
				32128.777	**897.892**	**320.693**	**213.795**		**-44611.725**	**46.527**		**5150.451**		**5647.826**	**1839.545**
Total =				**36494.385**	**1081.770**	**696.145**	**464.097**		**-40203.133**	**317.120**		**6812.540**		**5627.576**	**5668.556**
							-464.097			-317.120					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1699.672	309.778	1978.877	15937.827	3235.375
due to Substructure		1081.770	386.367		6812.540	2433.180
due to Active Earth pressure	1.00	3426.805			15856.307	
due to dynamic increment of EP	1.50	2719.184			16678.116	

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
1699.67	15937.83	0.00	0.00	0.00	0.00	309.78	3235.38
1081.77	6812.54	0.00	0.00	0.00	0.00	386.37	2433.18
3426.81	15856.31						
2719.18	16678.12						
8927.43	**55284.79**	**0.00**	**0.00**	**0.00**	**0.00**	**696.15**	**5668.56**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	36958.482	36030.288	KN
ML	15398.777	14764.537	kNm
MT	11296.132	11296.132	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				3368.341		309.778	206.519		3873.592	237.497				-20.250	3235.375
Forces from Substructure				33126.044	1081.770	386.367	257.578		-44076.725	79.624		6812.540		5647.826	2433.180
CWLL-Max. Reaction case	0.75			832.98		108.442	72.295	1.150	957.928	83.139	1981.717		0.950	791.332	1324.836
Vertical component of LL Surcharge	0.20			61.843				-5.000	-309.216						
Vertical component of dynamic increment LL Surcharge	1.50			276.766				-5.000	-1383.831						
Total =				**37665.975**	**1081.770**	**804.587**	**536.391**		**-40938.252**	**400.259**		**6812.540**		**6418.908**	**6993.392**
							-536.391			-400.259					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1745.212	418.220	1978.877	16364.856	4560.212
due to Substructure		1081.770	386.367		6812.540	2433.180
due to Active Earth pressure	1	3426.805			15856.307	
due to dynamic increment of EP	1.50	2719.184			16678.116	
due to Live load surcharge	0.20	149.303			822.433	
due to dynamic increment of Surcharge	1.50	668.173			5207.664	

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
1745.21	16364.86	0.00	0.00	0.00	0.00	418.22	4560.21
1081.77	6812.54	0.00	0.00	0.00	0.00	386.37	2433.18
3426.81	15856.31						
2719.18	16678.12						
149.30	822.43						
668.17	5207.66						
9790.45	**61741.92**	**0.00**	**0.00**	**0.00**	**0.00**	**804.59**	**6993.39**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	38202.366	37129.583	KN
ML	21203.923	20403.404	kNm
MT	13412.299	13412.299	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**3368.341**		**309.778**	**206.519**		**3873.592**	**237.497**				**-20.250**	**3235.375**

Substructure & Foundation -Portion 1				**997.267**	**183.878**	**65.674**	**43.783**		**534.999**	**33.096**		**1662.089**		**0.000**	**593.635**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	27.891	941.304	254.152	90.773	60.516	-2.250	-2117.935	-136.160	1975.482	1520.322	6.000	5647.826	543.001
Shaft above HFL	1.35	25	39.088	1319.234	356.193	127.219	84.812	1.267	1671.761	107.476	1976.139	2364.811	0.000	0.000	844.621
Shaft below HFL	1.35	15	51.9097793	1051.173	98.335	35.122	23.414	1.192	1252.850	27.907	1973.670	410.059	0.000	0.000	146.457
Back filling above HFL over heel slab	1.35	20	401.98125	10853.494	0.000	0.000	0.000	-2.250	-24420.361	0.000	1977.594	0.000	0.000	0.000	0.000
Back filling below HFL over heel slab	1.35	10	295.28125	3986.297	0.000	0.000	0.000	-2.330	-9288.211	0.000	1972.398	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	10	0.000	0.000				2.072	0.000				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	60.156	1218.164				-1.857	-2262.305				0.000	0.000	
Toe slab	1.35	15	35.000	708.750				3.171	2247.750				0.000	0.000	
portion between heel & toe	1.35	15	20.313	411.328				1.150	473.027				0.000	0.000	
Vertical component of active earth pressure	1.00			1378.414				-5.000	-6892.072						
Vertical component of dynamic increment of earth pressure	1.50			1127.108				-5.000	-5635.541						
				23373.178	**708.681**	**253.114**	**168.743**		**-44744.340**	**-0.777**		**4295.192**		**5647.826**	**1534.079**
Total =				**27738.787**	**892.559**	**628.566**	**419.044**		**-40335.749**	**269.815**		**5957.281**		**5627.576**	**5363.089**
							-419.044			-269.815					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1699.672	309.778	1978.877	15937.827	3235.375
due to Substructure		892.559	318.788		5957.281	2127.714
due to Active Earth pressure	1.00	3327.787			13557.017	
due to dynamic increment of EP	1.50	2721.080			16685.214	

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
1699.67	15937.83	0.00	0.00	0.00	0.00	309.78	3235.38
892.56	5957.28	0.00	0.00	0.00	0.00	318.79	2127.71
3327.79	13557.02						
2721.08	16685.21						
8641.10	**52137.34**	**0.00**	**0.00**	**0.00**	**0.00**	**628.57**	**5363.09**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	28157.83	27319.74	KN
ML	12071.41	11531.78	kNm
MT	10990.67	10990.67	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 = 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				3368.341		309.778	206.519		3873.592	237.497				-20.250	3235.375
Forces from Substructure				24370.445	892.559	318.788	212.525		-44209.341	32.319		5957.281		5647.826	2127.714
CWLL-Min. Reaction case	0.75			179.52		23.371	15.581	1.150	206.447	17.918	1981.717		0.950	170.543	285.521
Vertical component of LL Surcharge	0.20			67.487				-5.000	-337.436						
Vertical component of dynamic increment LL Surcharge	1.50			388.284				-5.000	-1941.422						
Total =				**28374.077**	**892.559**	**651.937**	**434.625**		**-42408.159**	**287.733**		**5957.281**		**5798.120**	**5648.611**
							-434.625			-287.733					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1745.212	333.149	1978.877	16364.856	3520.897
due to Substructure		892.559	318.788		5957.281	2127.714
due to Active Earth pressure	1	3327.787			13557.017	
due to dynamic increment of EP	1.50	2721.080			16685.214	
due to Live load surcharge	0.20	162.928			897.491	
due to dynamic increment of Surcharge	1.50	937.401			6677.285	

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
1745.21	16364.86	0.00	0.00	0.00	0.00	333.15	3520.90
892.56	5957.28	0.00	0.00	0.00	0.00	318.79	2127.71
3327.79	13557.02						
2721.08	16685.21						
162.93	897.49						
937.40	6677.29						
9786.97	**60139.14**	**0.00**	**0.00**	**0.00**	**0.00**	**651.94**	**5648.61**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	28808.702	27939.453	KN
ML	18018.718	17443.252	kNm
MT	11446.731	11446.731	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 αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**3368.341**		**1032.594**	**206.519**		**3873.592**	**237.497**				**-20.250**	**10784.584**
Substructure & Foundation -Portion 1				**997.267**	**55.163**	**218.914**	**43.783**		**534.999**	**33.096**		**498.627**		**0.000**	**1978.783**
Substructure & Foundation -Portion 2				**32128.777**	**269.368**	**1068.976**	**213.795**		**-44611.725**	**46.527**		**1545.135**		**5647.826**	**6131.818**
Total =				**36494.385**	**324.531**	**2320.484**	**464.097**		**-40203.133**	**317.120**		**2043.762**		**5627.576**	**18895.185**
							-464.097			-317.120					

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		509.902	1032.594	1978.877	4781.348	10784.584
due to Substructure		324.531	1287.890		2043.762	8110.601
due to Active Earth pressure		3426.805			15856.307	
due to dynamic increment of EP		815.755			5003.435	

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
509.90	4781.35	0.00	0.00	0.00	0.00	1032.59	10784.58
324.53	2043.76	0.00	0.00	0.00	0.00	1287.89	8110.60
3426.81	15856.31						
815.76	5003.43						
5076.99	**27684.85**	**0.00**	**0.00**	**0.00**	**0.00**	**2320.48**	**18895.19**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	36958.482	36030.288	KN
ML	-12201.161	-12835.401	kNm
MT	24522.761	24522.761	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**37665.975**	**536.391**	**-40938.252**	**400.259**	**6418.908**
					-536.391		-400.259	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		555.442	1394.067	1978.877	5208.377	15200.705
due to Substructure		324.531	1287.890		2043.762	8110.60064
due to Earth pressure		3426.805			15856.307	

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
555.44	5208.38	0.00	0.00	0.00	0.00	1394.07	15200.71
324.53	2043.76	0.00	0.00	0.00	0.00	1287.89	8110.60
3426.81	15856.31						

due to dynamic increment of EP		815.755			5003.435	
due to Live load surcharge		149.303			822.433	
due to dynamic increment of Surcharge		200.452			1562.299	

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
815.76	5003.43						
149.30	822.43						
200.45	1562.30						
5472.29	**30496.61**	**0.00**	**0.00**	**0.00**	**0.00**	**2681.96**	**23311.31**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	38202.366	37129.583	KN
ML	-10041.380	-10841.898	kNm
MT	29730.213	29730.213	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure				**3368.341**	**206.519**	**3873.592**	**237.497**	**-20.250**
Substructure & Foundation -Portion 1				**997.267**	**43.783**	**534.999**	**33.096**	**0.000**
Substructure & Foundation -Portion 2				**23373.178**	**168.743**	**-44744.340**	**-0.777**	**5647.826**
Total =				**27738.787**	**419.044**	**-40335.749**	**269.815**	**5627.576**
					-419.044		-269.815	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		509.902	1032.594	1978.877	4781.348	10784.5844
due to Substructure		267.768	1062.627		1787.184	7092.381
due to Active Earth pressure	1.00	3327.787			13557.017	
due to dynamic increment of EP	1.50	816.324			5005.564	

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
509.90	4781.35	0.00	0.00	0.00	0.00	1032.59	10784.58
267.77	1787.18	0.00	0.00	0.00	0.00	1062.63	7092.38
3327.79	13557.02						
816.32	5005.56						
4921.78	**25131.11**	**0.00**	**0.00**	**0.00**	**0.00**	**2095.22**	**17876.96**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	28157.831	27319.743	KN
ML	-14934.819	-15474.450	kNm
MT	23504.541	23504.541	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**28374.077**	**434.625**	**-42408.159**	**287.733**	**5798.120**
		-434.625		-287.733	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		555.442	1110.496	1978.877	5208.377	11736.3226
due to Substructure		267.768	1062.627		1787.184	7092.38056
due to Earth pressure		3327.787			13557.017	
due to dynamic increment of EP		816.324			5005.564	
due to Live load surcharge		162.928			897.491	
due to dynamic increment of Surcharge		281.220			2003.186	

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
555.44	5208.38	0.00	0.00	0.00	0.00	1110.50	11736.32
267.77	1787.18	0.00	0.00	0.00	0.00	1062.63	7092.38
3327.79	13557.02						
816.32	5005.56						
162.93	897.49						
281.22	2003.19						
5411.47	**28458.82**	**0.00**	**0.00**	**0.00**	**0.00**	**2173.12**	**18828.70**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	28808.702	27939.453	KN
ML	-13661.607	-14237.073	kNm
MT	24626.823	24626.823	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 (1981.717 - 1969.500) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.75 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1981.717 - 1969.500) = 0.000 kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

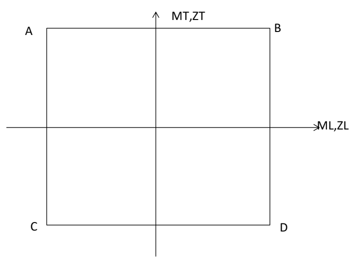
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

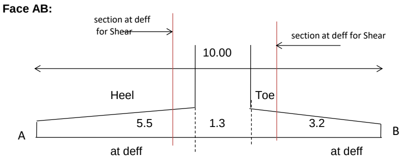
Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

Summary of Design Base Pressure

LOAD CASES	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	36211.002	-13452.894	5627.576	375.872	246.724	332.652	203.504
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	38248.022	-6688.041	7210.239	365.774	301.569	310.399	246.194
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	27495.633	-17235.592	5627.576	324.306	158.844	281.086	115.624
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	28259.595	-11079.923	5968.663	302.180	195.813	256.341	149.973
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	36958.482	15398.777	11296.132	265.131	412.959	178.377	326.205
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	38202.366	21203.923	13412.299	255.343	458.901	152.337	355.895
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	28157.831	12071.406	10990.666	209.524	325.410	125.116	241.001
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	28808.702	18018.718	11446.731	187.935	360.915	100.024	273.004
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	36958.482	-12201.161	24522.761	448.401	331.270	260.066	142.935
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	38202.366	-10041.380	29730.213	467.982	371.584	239.654	143.256
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	28157.831	-14934.819	23504.541	387.207	243.833	206.692	63.318
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	28808.702	-13661.607	24626.823	390.612	259.461	201.478	70.327



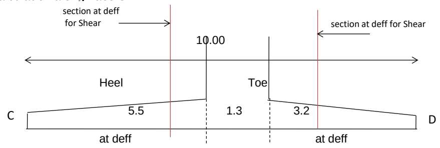
Pressure calculation along Face AB:



		at deff			at deff	
Case : 1	375.872	319.886	304.84	288.051	273.006	246.724
Case : 2	365.774	337.941	330.46	322.115	314.635	301.569
Case : 3	324.306	252.578	233.30	211.792	192.516	158.844
Case : 4	302.180	256.070	243.68	229.850	217.459	195.813
Case : 5	265.131	329.214	346.44	365.654	382.876	412.959
Case : 6	255.343	343.586	367.30	393.763	417.477	458.901
Case : 7	209.524	259.760	273.26	288.326	301.827	325.410
Case : 8	187.935	262.922	283.07	305.561	325.714	360.915
Case : 9	448.401	397.624	383.98	368.752	355.106	331.270
Case : 10	467.982	426.193	414.96	402.431	391.201	371.584
Case : 11	387.207	325.054	308.35	289.713	273.010	243.833
Case : 12	390.612	333.758	318.48	301.429	286.150	259.461

Average MAX Base Pressure for Design of Heel Slab-along Face AB	=	441.472	kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face AB	=	235.505	kN/m^2
Average MAX Base Pressure for Design of Toe Slab-along Face AB	=	426.332	kN/m^2
Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	417.477	kN/m^2
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	426.193	kN/m^2

Pressure calculation along Face CD:



		at deff			at deff	
Case : 1	332.652	276.666	261.62	244.832	229.786	203.504
Case : 2	310.399	282.567	275.09	266.740	259.260	246.194
Case : 3	281.086	209.358	190.08	168.572	149.296	115.624
Case : 4	256.341	210.231	197.84	184.011	171.619	149.973
Case : 5	178.377	242.460	259.68	278.900	296.122	326.205
Case : 6	152.337	240.579	264.29	290.756	314.471	355.895
Case : 7	125.116	175.352	188.85	203.918	217.419	241.001
Case : 8	100.024	175.011	195.16	217.651	237.803	273.004
Case : 9	260.066	209.290	195.64	180.417	166.771	142.935
Case : 10	239.654	197.865	186.64	174.103	162.873	143.256
Case : 11	206.692	144.540	127.84	109.198	92.495	63.318
Case : 12	201.478	144.624	129.35	112.295	97.016	70.327

Average MAX Base Pressure for Design of Heel Slab-along Face CD	=	297.136	kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face CD	=	147.594	kN/m^2
Average Base Pressure for Design of Toe Slab-along Face CD	=	323.325	kN/m^2
Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	314.471	kN/m^2
Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	282.567	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	=	441.472 kN/m^2								
Upward moment due to Base pressure	=	6677.269 kNm/m								
		factor								
Downward moment due to backfill	=	1.35 x		692.321	/	12.500	x	20	x	2.750
	=	4112.387 kNm/m								
Downward moment due to self weight of Heel slab	=	1.35 x		60.156	/	12.500	x	25	x	2.357
	=	382.852 kNm/m								
Net Moment at face of shaft	=	6677.269	-4112.387	-382.852 =		2182.030 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	=	147.594 kN/m^2								
Upward moment due to Base pressure	=	2232.355 kNm/m								
		factor								
Downward moment due to backfill	=	1.35 x		692.321	/	12.500	x	10	x	2.750
	=	2056.194 kNm/m								
Downward moment due to self weight of Heel slab	=	1.35 x		60.156	/	12.500	x	15	x	2.357
	=	229.711 kNm/m								
Net Moment at face of shaft	=	2232.355	-2056.194	-229.711 =		-53.549 kNm/m		Tension at Top of Heel Slab		

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	1.091 m					
effective depth at critical section	=	1.006 m					
Base pressure at deff from face of wall	=	426.193 kN/m^2					
Shear Force due to upward pressure at deff from face of wall	=	447.087 x	4.335 x	12.500	=	24226.552	KN
		factor					
Downward Force due to backfill	=	1.35 x	692.321 x	20	=	18692.670	KN
Downward Force due to self weight of Heel slab	=	1.35 x	60.156 x	25	=	2030.273	KN
Net Shear Force	=	24226.552	-18692.670	-2030.273	=	3503.609	KN
Net Shear Force / unit meter	=	3503.609 /	12.500		=	280.289	KN/m

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	426.332 kN/m^2								
Upward moment due to Base pressure	=	2182.819 kNm/m								
Downward moment due to self weight of Toe slab	=	1.35	x	35.000	/	12.500	x	25	x	1.371
	=	129.6000 kNm/m								
Net Moment at face of shaft	=	2182.819		-129.600 =		2053.219 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	=	0.977 m							
effective depth at critical section	=	0.889 m							
Base pressure at deff from face of wall	=	417.477 kN/m^2							
upward shear force due to base pressure	=	438.189 x		2.035 x		12.500		=	11146.433 KN
C.g. Of base pressure	=	0.908 m							
moment due to upward pressure at critical section	=	10120.657 kNm							
$\tan\beta$	=	0.234							
reduction in shear force (V_{ccd})	=	$\frac{M \tan\beta}{d}$ =		2427.987	KN				
Downward force due to self weight of toe slab	=	1.35	x	0.738	x	2.035	x	12.500	x 25
	=	633.994	KN						
Net Shear Force at deff	=	11146.433	-	633.994	-	2427.987		=	8084.452 KN
Net Shear Force / unit meter	=	8084.452	/	12.500				=	646.756 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
fctm	=	2.77 N/mm^2
Grade of Reinforcement Steel	=	Fe 500D (HYSD Bars)
fy or fyk	=	500.00 N/mm^2
fyd	=	434.78 N/mm^2 (fy/1.15)
Es	=	200000.00 N/mm^2

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	Toe Slab
Ultimate bending moment,Mu (kNm/m)	=	**53.55**	**2053.22**
Effective depth required (dreq) (mm)	=	95.72	592.69
Effective depth provided (dpro) (mm)	=	1165.00	1165.00
Check for provided depth	=	**SAFE**	**SAFE**
R = Mu/(b d2)	=	**0.04**	**1.51**
Total depth provided (mm)	=	1250.00	1250.00
Limiting depth of neutral axis (mm)	=	718.64	718.64
Actual depth of neutral axis (mm)	=	88.89	169.38
Check for Neutral axis depth	=	**OK**	**OK**
Lever arm (z) , mm	=	1128.02	1094.54
Moment of Resistance w.r.to steel	=	1263.44	2336.00
Check for Moment Capacity	=	**SAFE**	**SAFE**
Ast reqd (mm2 / m)	=	113.53	4588.00
cl. 16.6.1 (2) of IRC :112-2011			
$A_{S,min} = 0.26\, f_{ctm}\, b_t\, d / f_{yk} >= 0.0013\, b_t\, d$	=	1678.83	1678.83
Governing Ast (mm2 / m)	=	1678.83	4588.00

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 (mm2 / 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 (mm2/m)	=	515.22	981.75
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	100.00
Ast provided (mm2 /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})	=	**280.289**	**646.756**		kN/m
Ast provided	=	2576.106	4908.74		mm2/m
Depth of slab at critical section	=	**1091.136**	**976.953**		mm
Effective depth at critical section	=	1006.136	889.453		mm
Percentage of steel provided ($\rho 1$)	=	0.0024	0.0050		
cl. 10.3.1 of IRC :112-2011					
$\rho 1 = A_{sl}/(b_w\ d) <= 0.02$	=	OK	OK		
Actual shear stress=v_{ED} = (V_{Ed}/b*0.9d)	**=**	**0.310**	**0.808**		N/mm^2
Max shear capacity, 0.135 fck(1-fck/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) <= 2.0	=	1.446	1.474		
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010					
$v_{min} = 0.031\ K^{3/2}\ fck^{1/2}$	=	0.319	0.328		N/mm^2
$0.12\ K\ (80\ \rho 1\ f_{ck})^{0.33}$	=	0.324	0.423		N/mm^2
$\sigma_{cp} = N_{Ed} / A_c <= 0.2\ f_{cd}$	=	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	=	376.973	493.157		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	153.600		KN/m
b	=	12.500	12.500		m
Total Shear Force	=	0.000	1919.995		kN
θ = 0.5 x sin^-1 [vEd / (0.18 fck (1- fck/250))]		1.638	4.288		
cot θ = (< 1 cotθ <2.5)	=	2.500	2.500		
fywd = 0.8 x fy/1.15	=	347.826	347.826		N/mm^2
Provide Shear Reinforcement					
Legged	=	**16**	**30**		
Dia	=	**12**	**12**		mm
Area of Shear Reinf, Asw	=	1809.557	3392.920		mm^2
z = 0.9 * d	=	905.523	800.508		mm
Spacing of shear Reinforcement required					
S = Asw*z* fywd* cotθ / V_{Rd}	=	0.000	1230.103		mm
As per Clause 10.3.3.5 of IRC:112-2011					
Asw/ (b S) = $\rho_{w,min} = (0.072\ f_{ck}^{0.5}) / f_{yk}$	=	0.001	0.001		
Spacing of shear Reinforcement required	=	169.928	318.616		mm
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011					
S_{max} = 0.75 d	=	754.602	667.090		mm
Governing Spacing of Shear Reinf.	=	**0.000**	**318.616**		mm
Provided Spacing of Shear Reinf.	=	**200**	**200**		mm

SLS CHECK OF FOUNDATION

Foundation Lvl = 1969.500 m

Properties of Footing Base:

A	=	125.000	m^2
ZL	=	208.333	m^3
ZT	=	260.417	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			1641.914	1.150	1888.201	0.000	0.000
SIDL except Wearing Course	1.00			299.598	1.150	344.538	0.000	0.000
Wearing Course	1.20			468.000	1.150	538.200	0.000	0.000
Bearing Pedestal	1.00	25	1.920	48.000	1.150	55.200	-0.313	-15.000
				2457.512		**2826.139**		**-15.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	8.025	200.625	0.325	65.203	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.363	9.082	0.325	2.952	0.000	0.000
Bracket - Uniform portion	1.00	25	1.125	28.125	0.025	0.703	0.000	0.000
Bracket - Tapered portion	1.00	25	0.563	14.063	0.075	1.055	0.000	0.000
Cap - (uniform portion)	1.00	25	6.656	166.406	1.063	176.807	0.000	0.000
Cap - (corbel portion)	1.00	25	5.134	128.353	1.063	136.375	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	0.000	0.000	0.500	0.000	6.000	0.000
Cantilever Return Wall-Traingle portion	1.00	25	0.000	0.000	0.500	0.000	6.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.325	9.100	0.000	0.000
Aproach Slab	1.00	25	6.563	164.063	0.025	4.102	0.000	0.000
				738.716		**396.296**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	27.891	697.263	-2.250	-1568.841	6.000	4183.575
Abutment Shaft	1.00	25	90.998	2274.955	1.192	2711.426	0.000	0.000
Back filling over heel slab	1.00	20	692.321	13846.422	-2.284	-31627.105	0.000	0.000
Front Filling over toe slab	1.00	20	91.800	1836.000	3.313	6082.400	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	60.156	1503.906	-1.857	-2792.969	0.000	0.000
Toe slab	1.00	25	35.000	875.000	3.171	2775.000	0.000	0.000
portion between heel & toe	1.00	25	20.313	507.813	1.150	583.984	0.000	0.000
Vertical Components of active earth pressure	1.00			1508.246	-5.000	-7541.231	0.000	0.000
				23421.598		**-31158.242**		**4183.575**
Total				**26617.826**		**-27935.807**		**4168.575**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		120.476	1978.877	1129.700
due to Earth pressure	1.00	3641.229		15856.307

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.48	1129.70	0.00	0.00
3641.23	15856.31		
3761.704	**16986.007**	**0.000**	**0.000**

Summary of Forces

P	26617.826	KN
ML	-10949.801	kNm
MT	4168.575	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		26617.826		-27935.807		4168.575
CWLL-Max. Reaction case	1.00	1110.641	1.150	1277.237	0.950	1055.109
Vertical Components of LL Surcharge	0.80	247.373	-5.000	-1236.863	0.000	0.000
Total		**27975.840**		**-27895.434**		**5223.684**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		246.294	1978.877	2309.495
due to Earth pressure	1.00	3641.229		15856.307
due to Live load surcharge	0.80	597.210		3289.733

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
246.29	2309.49	0.00	0.00
3641.23	15856.31		
597.21	3289.73		
4484.733	**21455.535**	**0.000**	**0.000**

Summary of Forces

P	27975.840	KN
ML	-6439.899	kNm
MT	5223.684	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				**2457.512**		**2826.139**		**-15.000**
Substructure & Foundation -Portion 1				**738.716**		**396.296**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	27.891	697.263	-2.250	-1568.841	6.000	4183.575
Shaft above HFL	1.00	25	39.088	977.210	1.267	1238.342	0.000	0.000
Shaft below HFL	1.00	15	51.910	778.647	1.192	928.037	0.000	0.000
Back filling above HFL over heel slab	1.00	20	401.981	8039.625	-2.250	-18089.156	0.000	0.000
Back filling below HFL over heel slab	1.00	10	295.281	2952.813	-2.330	-6880.156	0.000	0.000
Front Filling over toe slab	1.00	10	0.000	0.000	2.072	0.000	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	60.156	902.344	-1.857	-1675.781	0.000	0.000
Toe slab	1.00	15	35.000	525.000	3.171	1665.000	0.000	0.000
Portion between Heel & Toe	1.00	15	20.313	304.688	1.150	350.391	0.000	0.000
Vertical Components of active earth pressure	1.00			1508.246	-5.000	-7541.231	0.000	0.000
				16965.769		**-31405.474**		**4183.575**
Total				**20161.997**		**-28183.039**		**4168.575**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		120.476	1978.877	1129.700
due to Earth pressure	1.00	3641.229		13557.017

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
120.48	1129.70	0.00	0.00
3641.23	13557.02		
3761.704	**14686.717**	**0.000**	**0.000**

Summary of Forces

P	20161.997	KN
ML	-13496.323	kNm
MT	4168.575	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		20161.997		-28183.039		4168.575
CWLL-Min. Reaction case	1.00	239.359	1.150	275.263	0.950	227.391
Vertical Components of LL Surcharge	0.80	269.949	-5.000	-1349.743	0.000	0.000
Total		**20671.305**		**-29257.519**		**4395.966**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		289.858	1978.877	2717.995
due to Earth pressure	1.00	3641.229		13557.017
due to Live load surcharge	0.80	651.714		3589.964

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
289.86	2718.00	0.00	0.00
3641.23	13557.02		
651.71	3589.96		
4582.800	**19864.976**	**0.000**	**0.000**

Summary of Forces

P	20671.305	KN
ML	-9392.543	kNm
MT	4395.966	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 (1981.717 - 1969.500)

= 0.000 kNm

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

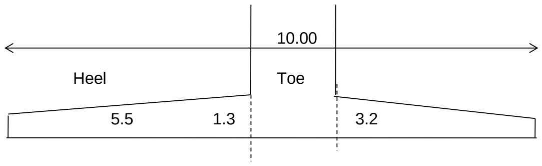
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

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

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	26617.826	-10949.801	4168.575	281.509	176.391	249.494	144.376
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	27975.840	-6439.899	5223.684	274.777	212.954	234.659	172.836
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	20161.997	-13496.323	4168.575	242.086	112.521	210.071	80.506
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	20671.305	-9392.543	4395.966	227.335	137.167	193.574	103.406

Pressure calculation along Face AB:



Case 1 :	281.509	223.69	210.029	176.391
Case 2:	274.777	240.77	232.738	212.954
Case 4:	242.086	170.83	153.982	112.521
Case 5:	227.335	177.74	166.021	137.167

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 257.776 kN/m^2

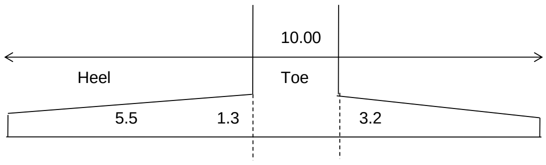
Average Base Pressure for Design of Toe Slab-along Face AB = 222.846 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 252.602 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face AB = 193.210 kN/m^2

Pressure calculation along Face CD:



Case 1 :	249.494	191.68	178.014	144.376
Case 2:	234.659	200.66	192.620	172.836
Case 4:	210.071	138.81	121.967	80.506
Case 5:	193.574	143.98	132.260	103.406

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 220.587 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 182.728 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 220.587 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 161.195 kN/m^2

Moment Calculation

	Rare Combination		Quasi-Parmenant		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	257.78	222.85	252.60	193.21	kN/m^2
Upward moment due to Base pressure	3898.86	1140.97	3820.60	989.23	kNm/m
Downward moment due to backfill	3046.21	0.00	3046.21	0.00	kNm/m
Downward moment due to self weight of slab	283.59	96.00	283.59	96.00	kNm/m
Net Moment	**569.05**	**1044.97**	**490.79**	**893.23**	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 = $\frac{Ecm}{(1 + \phi)}$ = 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	**=**	**569.05**	**1044.97**	**490.79**	**893.23**	kNm/m
Dx	**=**	**1.00**	**1.00**			m
Dy	**=**	**1.25**	**1.25**			m
Section Modulus (ZL) of uncracked sec	**=**	**0.26**	**0.26**			m^3
Bending Stress (M/ZL)	=	2.185	4.013			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)	=	1165.000	1165.000			mm
Ast provided	=	2576.106	4908.739			mm^2/m
Percentage of steel , pt	=	0.0022	0.0042			

k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m	=	0.248	0.325			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	289.440	378.642			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	875.560	786.358			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	790.560	698.858			mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2				
Icr	=	4.476E+10	7.447E+10			mm^4
Maximum compressive stress in concrete	=	**3.680**	**5.313**	**3.174**	**4.542**	**< 16.8, SAFE**
Maximum Tensile stress in steel	=	**186.662**	**182.128**	**160.990**	**155.682**	**< 300, SAFE**

Check For Crack Width in Quasi-Permanent Load Combination

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)	=	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 +	$\frac{0.425\, k1\, k2\, \varphi}{\rho_{\rho\, eff}}$	
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , yc	=	289.440	378.642	mm
$\rho_{\rho\, eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.		
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	212.500	212.500	mm
Ac, eff = Dx * hc,eff	=	212500.000	212500.000	mm
$\rho_{\rho\, eff}$ = As/Ac eff	=	0.012	0.023	
Maximum crack spacing , $S_{r\,max}$	=	535.462	438.983	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct\, eff} (1 + \alpha_e \rho_{\rho\, eff})}{\rho_{\rho\, eff}}$	/ Es	
tensile stress in steel, ,σsc	=	160.990	155.682	N/mm^2
Kt	=	0.500	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	2.771	N/mm^2
αe = Es/Ecm	=	6.190	6.190	
(εsm - εcm)	=	0.00048	0.0005	
Crack width , Wk=Sr max (εsm - εcm)	=	**0.259**	**0.205**	mm
Check	=	**SAFE**	**SAFE**	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom lvl = 1970.750 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			2216.584	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1.35			404.457	0.000	0.000	0.000	0.000
Wearing Course	1.75			682.500	0.000	0.000	0.000	0.000
Bearing Pedestal	1.35	25	1.920	64.800	0.000	0.000	-0.313	-20.250
				3368.341		**0.000**		**-20.250**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	8.025	270.844	-0.825	-223.446	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.363	12.261	-0.825	-10.115	0.000	0.000
Bracket - Uniform portion	1.35	25	1.125	37.969	-1.125	-42.715	0.000	0.000
Bracket - Tapered portion	1.35	25	0.563	18.984	-1.075	-20.408	0.000	0.000
Cap - (uniform portion)	1.35	25	6.656	224.648	-0.087	-19.657	0.000	0.000
Cap - (corbel portion)	1.35	25	5.134	173.276	-0.087	-15.162	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.825	-31.185	0.000	0.000
Aproach Slab	1.35	25	6.563	221.484	-1.125	-249.170	0.000	0.000
				997.267		**-611.858**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1.35	25	90.998	3071.189	0.117	360.009	0.000	0.000
Total				**7436.797**		**-251.849**		**-20.250**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		165.177	1978.877	1342.394
due to Earth pressure	1.5	4292.743		16572.454

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
165.18	1342.39	0.00	0.00
4292.74	16572.45	0.00	0.00
4457.920	**17914.848**	**0.000**	**0.000**

Summary of Forces

P	7436.797	KN
ML	17662.999	kNm
MT	20.250	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		7436.797		-251.849		-20.250
CWLL-Max. Reaction case	1.5	1665.961	0.000	0.000	0.950	1582.663
Total		**9102.758**		**-251.849**		**1562.413**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		353.904	1978.877	2876.178
due to Earth pressure	1.5	4292.743		16572.454
due to Live load surcharge	1.2	794.175		3878.355

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
353.90	2876.18	0.00	0.00
4292.74	16572.45		
794.18	3878.36		
5440.822	**23326.987**	**0.000**	**0.000**

Summary of Forces

P	9102.758	KN
ML	23075.138	kNm
MT	1562.413	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				3368.341		0.000		-20.250
Substructure-Portion 1				997.267		-611.858		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	39.088	1319.234	0.117	208.767	0.000	0.000
Shaft below HFL	1.35	23.500	51.910	1646.838	0.042	93.063	0.000	0.000
Total				**7331.680**		**-310.028**		**-20.250**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		165.177	1978.877	1342.394
due to Earth pressure	1.5	4292.743		14689.765

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
165.18	1342.39	0.00	0.00
4292.74	14689.77	0.00	0.00
4457.920	**16032.159**	**0.000**	**0.000**

Summary of Forces

P	7331.680	KN
ML	15722.131	kNm
MT	20.250	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		7331.680		-310.028		-20.250
CWLL-Max. Reaction case	1.5	359.039	0.000	0.000	0.950	341.087
Total		**7690.719**		**-310.028**		**320.837**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		419.250	1978.877	3407.246
due to Earth pressure	1.5	4292.743		14689.765
due to Live load surcharge	1.2	866.654		4232.306

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
419.25	3407.25	0.00	0.00
4292.74	14689.77		
866.65	4232.31		
5578.647	**22329.317**	**0.000**	**0.000**

Summary of Forces

P	7690.719	KN
ML	22019.289	kNm
MT	320.837	kNm

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

Seismic Effect Factor = 1.50 αh= 0.180 In Longitudinal direction
αh= 0.289 In Transverse direction
αv= 0.193 In Vertical direction

Weight of shaft below Ground level = 687.0 KN

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 = 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			2216.584		213.753	142.502	0.000	0.000	0.000	1979.927		0.000	0.000	1961.616

SIDL except Wearing Course	1.35			404.457		39.003	26.002	0.000	0.000	0.000	1980.966		0.000	0.000	398.455
Wearing Course	1.75			682.500		50.772	33.848	0.000	0.000	0.000	1980.517		0.000	0.000	495.894
Bearing Pedestal	1.35	25	1.92	64.800		6.249	4.166	0.000	0.000	0.000			-0.313	-20.250	
				3368.341		**309.778**	**206.519**		**0.000**	**0.000**				**-20.250**	**2855.964**
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	8.025	270.844	54.169	26.118	17.412	-0.825	-223.446	-14.365	1979.447	471.106	0.000	0.000	227.152
Dirt Wall-Tapered portion	1.35	25	0.363	12.261	2.452	1.182	0.788	-0.825	-10.115	-0.650	1978.329	18.584	0.000	0.000	8.961
Bracket - Uniform portion	1.35	25	1.125	37.969				-1.125	-42.715						
Bracket - Tapered portion	1.35	25	0.563	18.984				-1.075	-20.408						
Cap - (uniform portion)	1.35	25	6.656	224.648	44.930	21.664	14.442	-0.087	-19.657	-1.264	1978.058	328.357	0.000	0.000	158.324
Cap - (corbel portion)	1.35	25	5.134	173.276	34.655	16.710	11.140	-0.087	-15.162	-0.975	1977.758	242.873	0.000	0.000	117.106
RCC Railing or Crash Barrier	1.35	25		37.800				-0.825	-31.185				0.000		
Aproach Slab	1.35	25	6.563	221.484				-1.125	-249.170				0.000		
				997.267	**136.206**	**65.674**	**43.783**		**-611.858**	**-17.254**		**1060.920**		**0.000**	**511.542**
Substructure-Portion 2															
Abutment Shaft	1.35	25	90.998	**3071.189**	**476.844**	**229.919**	**153.280**	**0.117**	**360.009**	**17.968**	**1975.139**	**2092.929**	**0.000**	**0.000**	**1009.145**
Total =				**7436.797**	**613.050**	**605.372**	**403.581**		**-251.849**	**0.714**		**3153.849**		**-20.250**	**4376.651**
							-403.581			-0.714					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1259.017	309.778	1978.877	10232.027	2855.964
due to Substructure		613.050	295.594		3153.849	1520.687
due to Active Earth pressure	1.00	2861.829			11048.303	
due to dynamic increment of EP	1.50	2719.184			13279.135	

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
1259.02	10232.03	0.00	0.00	0.00	0.00	309.78	2855.96
613.05	3153.85	0.00	0.00	0.00	0.00	295.59	1520.69
2861.83	11048.30						
2719.18	13279.14						
7453.08	**37713.31**	**0.00**	**0.00**	**0.00**	**0.00**	**605.37**	**4376.65**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	7840.378	7033.216	KN
ML	37462.179	37460.751	kNm
MT	4356.401	4356.401	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				3368.341		309.778	206.519		0.000	0.000				-20.250	2855.964
Forces from Substructure				4068.456	613.050	295.594	197.062		-251.849	0.714		3153.849		0.000	1520.687
CWLL-Max. Reaction case	0.20			222.13		28.918	19.279	0.000	0.000	0.000	1981.717		0.950	211.022	317.142
Total =				**7658.925**	**613.050**	**634.289**	**422.860**		**-251.849**	**0.714**		**3153.849**		**190.772**	**4693.793**
							-422.860			-0.714					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1304.557	338.696	1978.877	10602.131	3173.106
due to Substructure		613.050	295.594		3153.849	1520.687
due to Active Earth pressure	1.00	2861.829			11048.303	
due to dynamic increment of EP	1.50	2719.184			13279.135	
due to Live load surcharge	0.20	132.363			646.393	
due to dynamic increment of Surcharge	1.50	668.173			4372.448	

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
1304.56	10602.13	0.00	0.00	0.00	0.00	338.70	3173.11
613.05	3153.85	0.00	0.00	0.00	0.00	295.59	1520.69
2861.83	11048.30						
2719.18	13279.14						
132.36	646.39						
668.17	4372.45						
8299.15	**43102.26**	**0.00**	**0.00**	**0.00**	**0.00**	**634.29**	**4693.79**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	8081.785	7236.066	KN
ML	42851.123	42849.696	kNm
MT	4884.565	4884.565	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				3368.341		309.778	206.519		0.000	0.000				-20.250	2855.964
Substructure-Portion 1				997.267	136.206	65.674	43.783		-611.858	-17.254		1060.920		0.000	511.542
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	39.088	1319.234	263.847	127.219	84.812	0.117	154.642	9.942	1976.139	1421.903	0.000	0.000	685.597
Shaft below HFL	1.350	23.500	51.910	1646.838	191.974	92.564	61.709	0.042	68.935	2.583	1973.670	560.564	0.000	0.000	270.286
				2966.072	**455.821**	**219.782**	**146.522**		**223.578**	**12.525**		**1982.467**		**0.000**	**955.884**
Total =				**7331.680**			**396.823**		**-388.280**	**-4.729**				**-20.250**	
							-396.823			4.729					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1259.017	309.778	1978.877	10232.027	2855.964
due to Substructure		592.027	285.457		3043.387	1467.426
due to Active Earth pressure	1.00	2702.381			9793.177	
due to dynamic increment of EP	1.50	2721.080			13283.864	

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
1259.02	10232.03	0.00	0.00	0.00	0.00	309.78	2855.96
592.03	3043.39	0.00	0.00	0.00	0.00	285.46	1467.43
2702.38	9793.18						
2721.08	13283.86						
7274.50	**36352.45**	**0.00**	**0.00**	**0.00**	**0.00**	**595.23**	**4323.39**

Summary of Forces

	Downward	Upward	
P	7728.503	6934.857	KN
ML	35959.446	35968.904	kNm
MT	4343.640	4343.640	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 = 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				3368.341		309.778	206.519		0.000	0.000				-20.250	2855.964
Forces from Substructure				3963.339	592.027	285.457	190.304		-388.280	-4.729		3043.387		0.000	1467.426
CWLL-Min. Reaction case	0.20			47.87		6.232	4.155	0.000	0.000	0.000	1981.717		0.950	45.478	68.349
Total =				**7379.552**	**592.027**	**601.467**	**400.978**		**-388.280**	**-4.729**		**3043.387**		**25.228**	**4391.738**
							-400.978			4.729					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1304.557	316.010	1978.877	10602.131	2924.313
due to Substructure		592.027	285.457		3043.387	1467.426
due to Active Earth pressure	1.00	2702.381			9793.177	

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
1304.56	10602.13	0.00	0.00	0.00	0.00	316.01	2924.31
592.03	3043.39	0.00	0.00	0.00	0.00	285.46	1467.43
2702.38	9793.18						

due to dynamic increment of EP	1.50	2721.080			13283.864	
due to Live load surcharge	0.20	144.442			705.384	
due to dynamic increment of Surcharge	1.50	937.401			5505.534	

2721.08	13283.86						
144.44	705.38						
937.40	5505.53						
8401.89	**42933.48**	**0.00**	**0.00**	**0.00**	**0.00**	**601.47**	**4391.74**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	7780.530	6978.574	KN
ML	42540.467	42549.925	kNm
MT	4416.967	4416.967	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**7436.797**	**403.581**	**-251.849**	**0.714**	**-20.250**
					-403.581		-0.714	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		377.705	1032.59351	1978.877	3069.608	9519.87959
due to Substructure		183.915	985.311802		946.155	5068.95636
due to Active Earth pressure	1.00	2861.829			11048.303	
due to dynamic increment of EP	1.50	815.755			3983.741	

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
377.70	3069.61	0.00	0.00	0.00	0.00	1032.59	9519.88
183.92	946.15	0.00	0.00	0.00	0.00	985.31	5068.96
2861.83	11048.30						
815.76	3983.74						
4239.20	**19047.81**	**0.00**	**0.00**	**0.00**	**0.00**	**2017.91**	**14588.84**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	7840.378	7033.216	KN
ML	18796.671	18795.244	kNm
MT	14609.086	14609.086	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**7658.925**	**422.860**	**-251.849**	**0.714**	**190.772**
					-422.860		-0.714	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		423.245	1128.98642	1978.877	3439.712	10577.0206
due to Substructure		183.915	985.311802		946.155	5068.956
due to Earth pressure	1.00	2861.829			11048.303	
due to dynamic increment of EP	1.50	815.755			3983.741	
due to Live load surcharge	0.20	132.363			646.393	
Surcharge	1.50	200.452			1311.734	

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
423.24	3439.71	0.00	0.00	0.00	0.00	1128.99	10577.02
183.92	946.15	0.00	0.00	0.00	0.00	985.31	5068.96
2861.83	11048.30						
815.76	3983.74						
132.36	646.39						
200.45	1311.73						
4617.56	**21376.04**	**0.00**	**0.00**	**0.00**	**0.00**	**2114.30**	**15645.98**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	8081.785	7236.066	KN
ML	21124.902	21123.474	kNm
MT	15836.749	15836.749	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**7331.680**	**396.823**	**-388.280**	**-4.729**	**-20.250**
					-396.823		4.729	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		377.705	1032.59351	1978.877	3069.608	9519.880
due to Substructure		177.608	951.522289		913.016	4891.419
due to Active Earth pressure	1.00	2702.381			9793.177	
due to dynamic increment of EP	1.50	816.324			3985.159	

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
377.70	3069.61	0.00	0.00	0.00	0.00	1032.59	9519.88
177.61	913.02	0.00	0.00	0.00	0.00	951.52	4891.42
2702.38	9793.18						
816.32	3985.16						
4074.02	**17760.96**	**0.00**	**0.00**	**0.00**	**0.00**	**1984.12**	**14411.30**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	7728.503	6934.857	KN
ML	17367.951	17377.409	kNm
MT	14431.549	14431.549	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.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**7379.552**	**400.978**	**-388.280**	**-4.729**	**25.228**
					-400.978		4.729	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		423.245	1053.36759	1978.877	3439.712	9747.70886
due to Substructure		177.608	951.522289		913.016	4891.419
due to Earth pressure	1.00	2702.381			9793.177	
due to dynamic increment of EP	1.50	816.324			3985.159	
due to Live load surcharge	0.20	144.442			705.384	
Surcharge	1.50	281.220			1651.660	

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
423.24	3439.71	0.00	0.00	0.00	0.00	1053.37	9747.71
177.61	913.02	0.00	0.00	0.00	0.00	951.52	4891.42
2702.38	9793.18						
816.32	3985.16						
144.44	705.38						
281.22	1651.66						
4545.22	**20488.11**	**0.00**	**0.00**	**0.00**	**0.00**	**2004.89**	**14639.13**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	7780.530	6978.574	KN
ML	20095.099	20104.557	kNm
MT	14664.356	14664.356	kNm

Forces along Long. Axis	Forces along Trans. Axis

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 (	1981.717 -	1970.750)	=	0.000	kNm

	FT Cosθ	MT Cosθ	FT Sinθ	MT Sin θ
Normal	0.00	0.00	0.00	0.00

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00		=	0.000	KN
Transverse Moment due to C.F.	=	0.000	x (	1981.717 -	1970.750)	=	0.000	kNm

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		7436.797	17662.999	20.250
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		9102.758	23075.138	1562.413
Normal HFLCase				
Case 3 : DL+SIDL-Normal HFL Case		7331.680	15722.131	20.250
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		7690.719	22019.289	320.837
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	7840.378	37462.179	4356.401
	UP	7033.216	37460.751	4356.401
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	8081.785	42851.123	4884.565
	UP	7236.066	42849.696	4884.565
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	7728.503	35959.446	4343.640
	UP	6934.857	35968.904	4343.640
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	7780.530	42540.467	4416.967
	UP	6978.574	42549.925	4416.967
Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	7840.378	18796.671	14609.086
	UP	7033.216	18795.244	14609.086
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	8081.785	21124.902	15836.749
	UP	7236.066	21123.474	15836.749
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	7728.503	17367.951	14431.549
	UP	6934.857	17377.409	14431.549
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	7780.530	20095.099	14664.356
	UP	6978.574	20104.557	14664.356
	MAX =	**9102.76**	**42851.12**	**15836.75**

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)	=	1.300 m
Depth of Wall (D)	=	12.500 m
Area of Concrete (Ac)	=	16.250 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) = **9102.76** kN

0.1 fcd Ac = 0.1 x 15.63 x 16250000

= 25404167 N

= **25404.17** 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) = 42851.12 kNm = **3428.090 kNm/m**

Check For Depth of Wall :

Mult = 0.165 x fck x b x d^2

= 3428.09 kNm/m

b = 1000.00 mm

Effective Depth Required (dreq) = $SQRT\left(\frac{3428.09 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

(dreq) = 770.460 mm

Total Depth Required (Dreq)	=	877.46 mm	
Total Depth Provided (Dprov)	=	1300.00 mm	OK
Effective depth provided(deff)	=	1193 mm	

R= Mu/(b d^2) = **2.41**

Minimum Longitudinal Reinforcement in wall on each face

= 0.0012 x b x D

Ast min = **1560.00** mm^2/m

Area of Steel Required:

$$\frac{pt}{100} = \frac{Ast_{req}}{b\,D} = \frac{fck\{1 - sqrt(1 - 4.598\,R/fck)\}}{2\,fy}$$

= 0.0061

Ast_{req} = 7881.268 mm^2/m

Ast required = max(Astmin, Astreq) = **7881.27** mm^2/m

Total are of steel required in full length = **98515.85** 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 = 12254 mm

Calculation of reinforcement in numbers

Provide	32	mm dia	-	62.00	nos	=	100530.96	mm^2	OK
Provide	32	mm dia	-	63.00	nos	=			

Percentage of steel = **0.619** %

Check for Moment of Resistance of Section due to Steel

Limiting Depth of Neutral Axis , Xm $= \dfrac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

$= \dfrac{0.0035 \quad x \quad 1193.00}{0.0035 \quad + \quad 0.0022}$

= 735.91 mm

Depth of Neutral Axis , X $= \dfrac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

$= \dfrac{434.78 \quad x \quad 8042.48}{0.36 \quad x \quad 35.00 \quad x \quad 1000.00}$

= 277.52 mm **OK**

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

z = d - 0.416 . X
= 1193.00 - 115.45
= 1077.55 mm

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

MR = fyd . Ast . z

= 434.78 x 8042.48 x 1077.55

= 3.77E+09 Nmm /m

= 3767.91 kNm/m > 3428.09 kNm/m

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

= 0.0012 x b x D
Ast min = **1560.00** mm^2/m
= **19500.00** mm^2

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

Calculation of reinforcement in numbers

Provide	20	mm dia	@	62.00	nos	=	19477.87 mm^2	CHECK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

maximum of following = 0.2500 x 9613.27 = 2403.318 As per IRC: 112-2011 , Clause
= 0.001 x 1.30E+06 = 1300.000 16.3.2

Maximum Horizontal Reinf. **2403** mm^2 per meter

Min dia of bar = 0.250 x 32 = 8 mm
or 8 mm

Maximum Spacing between <= 300 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

Distance between links or ties (ST)	=	1/3	x	1143 =	381
	or	200.00 mm			
Governing Spacing	=	200.00 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)

SL	=		5	x	32 =	160 mm
	or	1/5		x	1143 =	228.6 mm
Min	=		100 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 Reinforrcement:

nk = $\frac{\text{NED}}{\text{Ak fck}}$ = $\frac{9102758.3}{568750000}$ = 0.0160

AC	=	16.250 mm^2			
ACC	=	1.175	x	12.400 =	14.570 mm^2
ρ L	=	0.00620 per meter			
ρ L	=	0.07748			
fyd	=	434.783			
fcd	=	15.633			

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

$\omega_{wd.req}$ = 0.2506
ω_{wd} = max ($\omega_{wd,req}$, 0.12) = 0.2506

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

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

Volumetric ratio,

Asw	=	66853.092 mm^2
SL	=	100.000 mm
b	=	1143.000 mm
ρw	=	0.585
$\omega_{wd.c}$	=	16.267

$\omega_{wd,c}$ ≥ ω_{wd} as per equation 17.7 of IRC:112-2011

$\omega_{wd,c}$ = 16.27 OK

Length of Potential Plastic Hinges Refer clause 17.2.1.4 of IRC:112-2011

nk = $\frac{\text{NED}}{\text{Ak fck}}$ = 0.0160 < 0.30

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1970.750 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			1641.914	0.000	0.000	0.000	0.000
SIDL except Wearing Course	1			299.598	0.000	0.000	0.000	0.000
Wearing Course	1			390.000	0.000	0.000	0.000	0.000
Bearing Pedestal	1	25	1.920	48.000	0.000	0.000	-0.313	-15.000
				2379.512		**0.000**		**-15.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	8.025	200.625	-0.825	-165.516	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.363	9.082	-0.825	-7.493	0.000	0.000
Bracket - Uniform portion	1	25	1.125	28.125	-1.125	-31.641	0.000	0.000
Bracket - Tapered portion	1	25	0.563	14.063	-1.075	-15.117	0.000	0.000
Cap - (uniform portion)	1	25	6.656	166.406	-0.087	-14.561	0.000	0.000
Cap - (corbel portion)	1	25	5.134	128.353	-0.087	-11.231	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.825	-23.100	0.000	0.000
Aproach Slab	1	25	6.563	164.063	-1.125	-184.570	0.000	0.000
				738.716		**-453.228**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1	25	90.998	2274.955	0.117	266.673	0.000	0.000
Total				**5393.183**		**-186.555**		**-15.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		120.476	1978.877	979.105
due to Earth pressure	1	2861.829		11048.303
				12027.408

Summary of Forces at Bottom of abutment shaft

P	5393.183	KN
ML	11840.853	kNm
MT	-15.000	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		5393.183		-186.555		-15.000
CWLL-Max. Reaction case	1	1110.641	0.000	0.000	0.950	1055.109
Total		**6503.824**		**-186.555**		**1040.109**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		246.294	1978.877	2001.628
due to Earth pressure	1	2861.829		11048.303
due to Live load surcharge	0.8	529.450		2585.570
				15635.501

Summary of Forces at Bottom of abutment shaft

P	6503.824	KN
ML	15448.946	kNm
MT	1040.109	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				2379.512		0.000		-15.000

Substructure-Portion 1				738.716		-453.228		0.000
Substructure-Portion 2								
Shaft above HFL	1.000	25.000	39.088	977.210	0.12	114.55	0.00	0.00
Shaft below HFL	1.000	23.500	51.910	1219.880	0.04	51.06	0.00	0.00
				5315.318		**-287.615**		**-15.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		120.476	1978.877	979.105
due to Earth pressure	1	2861.829		9793.177
				10772.282

Summary of Forces at Bottom of abutment shaft

P	5315.318	KN
ML	10484.667	kNm
MT	-15.000	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		5315.318		-287.615		-15.000
CWLL-Max. Reaction case	1	239.359	0.000	0.000	0.950	227.391
Total		**5554.678**		**-287.615**		**212.391**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		289.858	1978.877	2355.673
due to Earth pressure	1	2861.829		9793.177
due to Live load surcharge	0.8	577.769		2821.537
				14970.387

Summary of Forces at Bottom of abutment shaft

P	5554.678	KN
ML	14682.772	kNm
MT	212.391	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 (	1981.717 - 1970.750)	=	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	5393.183	11840.853	-15.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	6503.824	15448.946	1040.109
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	5315.318	10484.667	-15.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	5554.678	14682.772	212.391

IN RARE COMBINATION

Max SLS Moment	=	15448.946	kNm
Max Moment per meter	=	1235.916	kNm/m

IN QUASI-PERMANENT

Max SLS Moment	=	11840.853	kNm
Max Moment per meter	=	947.268	kNm/m

Check For Stresses in Rare and Quasi-Permanent Load Combination

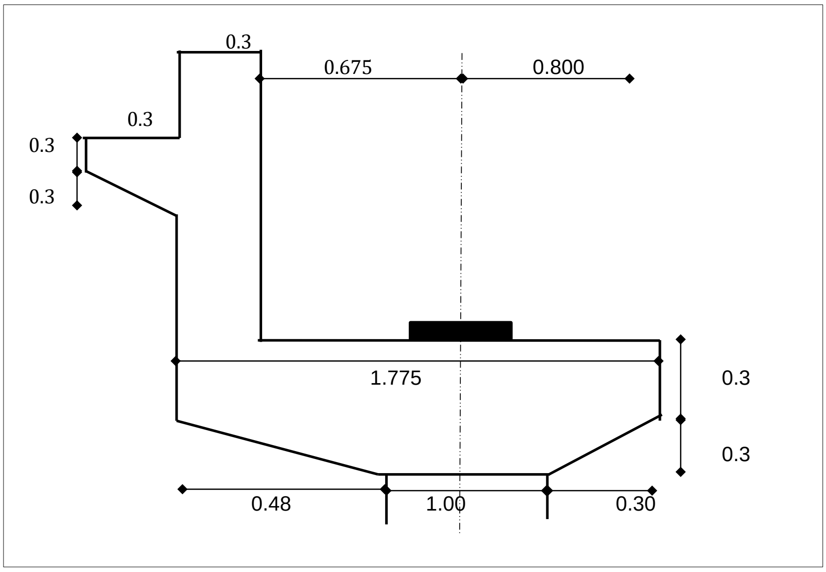
Creep Coeff = 2

		Rare Combination		Quasi permanent	
		Short term	Long Term		
Working bending moment, M	=	1235.92	1235.92	947.27	kNm/m
Dx (unit width of shaft)	=	1.00	1.00	1.00	m
Dy (Thickness of shaft)	=	1.30	1.30	1.30	m
Section Modulus (ZL) of uncracke	=	0.28	0.28	0.28	m^3
Bending Stress (M/ZL)	=	4.388	4.388	3.363	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)	=	1193.000	1193.000	1193.00	mm
Ast provided	=	8042.477	8042.477	8042.48	mm^2/m
Percentage of steel , pt	=	0.0062	0.0062	0.0062	
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m		0.241	0.378	0.378	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	287.627	451.002	451.002	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	905.373	741.998	741.998	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	814.373	650.998	650.998	mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2			
Icr	=	4.874E+10	1.128E+11	1.128E+11	mm^4
Maximum compressive stress in concrete	=	**7.293**	**4.941**	**3.787**	**< 16.8, SAFE**
Maximum tensile stress in concrete	=	22.957	8.129	6.231	
Maximum Tensile stress in steel	=	**127.830**	**132.453**	**101.519**	**< 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 c +	$\frac{0.425\ k1\ k2\ \varphi}{\rho_{\rho\ eff}}$
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	451.002	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	, where Ac,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
Ac, eff = Dx * hc,eff	=	267500.000	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	0.030	
Maximum crack spacing , $S_{r\ max}$	=	435.939	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct\ eff} (1 + \alpha_e \rho_{\rho\ eff})}{\rho_{\rho\ eff}}$	/ Es
tensile stress in steel, ,σsc	=	101.519	N/mm^2
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm^2
αe = Es/Ecm	=	18.571	
(εsm - εcm)	=	0.00030	
Crack width , Wk=Sr max (εsm - ε	=	**0.133**	**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.

Assuming a cap thickness of	=	300 mm				
Volume of abutment cap	=	300	x	1775	x	12500
	=	6.66E+09	mm^3			

as per cl. 710.8.7 of IRC : 78- 2014

Quantity of steel = 1 % of volume

= $\frac{1}{100}$ x 6.66E+09 = 6.66E+07 mm^3

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction

= 3.33E+07 mm^3

Clear cover = 50 mm

Length of bar = 12500 -100 = 12400 mm

Area of steel required in longitudinal direc = $\frac{3.33E+07}{12400}$ = 2683.972 mm^2 (top +Bottom)

Provide	12	Nos. of	12	mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

Provided steel = 1357 mm^2

(b) Transverse steel

Volume of steel to be provided in transverse direction = 3.33E+07 mm^3

Volume of steel required per meter = $\frac{3.33E+07}{12.50}$ = 2.66E+06 mm^3/m

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

Length of each stirrups = 1775 -100 -16 = 1659 mm

Volume of steel provided per meter = 3.34E+06 mm^3/m **OK**

Thickness of Abutment Cap (uniform) s	=	0.3 m
Thickness of Abutment Cap (tapered)	=	0.3 m
C.G. of dirt wall from face of abutment shaft (a)	=	0.325 m
Overall depth of Abutment cap at face of abutment shaft	=	0.600 m
Clear cover	=	50 mm
Diameter of the main bar	=	20 mm

Effective cover	d'	=	60	mm
Effective depth of cap	d	=	0.540	m

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

Hence a / d = 0.325 / 0.54
= 0.602 **< 1.0**

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

Hence s / d = 0.3 / 0.54
= 0.56 **> 0.5** **Proceed with the design**

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	=	16.7766	kN		
Self Weight of Bracket	=	0.135	m^3/m	x	25
	=	3.375	kN		
Total Dead Load	=	20.1516	kN		
Load Factor	=	1.35			
Ultimate Dead Load	=	**27.2046**	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 "	**327.20**	kN

Horizontal Load (H_u)

H_u = 1.7 x actual horizontal force in working load condition
but not less than
= 0.2 * V_u

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 < = 0.15 f_c'**

where; V_u = 327.20 kN
= 327204.61 N

bd = 1000 x 540 sqmm
= 540000.00 mm^2

V_u / b d = 327204.61 / 540000

V_u / b d = **0.61** N / mm^2

f_c' = 28 - day standard cylinder strength of concrete used.
= 0.80 times the standard cube strength

Grade of Concrete M 35

0.15 f_c' = 0.15 x 0.8 x 35
= **4.20** N / mm^2 **Ensured**

Hence **V_u / b d < = 0.15 f_c'** is **Ensured**

STEP II

Calculation of Shear Friction Reinforcement " A_{vf} "

A_{vf} = $\frac{V_u}{\quad}$

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

where; f_{sy} = yeild stress value of the reinforcement used.
= 500.00 N/ mm^2

	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

Type of Surface (1 / 2 / 3 / 4) ? = **1.00**

m = 1.40

(Note: Only monolithic construction is recommended)

$$A_{vf} = \frac{327.21 \times 1000}{0.87 \times 500 \times 1.4}\ mm^2$$

A_{vf} = **549.92** mm^2

STEP III

Calculation for Direct Tension Reinforcement " A_t ".

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

H_u = 68.00 kn

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

= **156.32** mm^2

STEP IV

Calculation for Flexural Tension Reinforcement " A_f "

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

$$= \frac{327.21 \times 1000 \times 325 + 68 \times 1000\ (600 - 60)}{0.87 \times 500 \times 540}$$

= **609.03** mm^2

STEP V

Total Primary Tensile Reinforcement " A_s "

A_s >= $(A_f + A_t)$
>= $(2 / 3\ A_{vf} + A_t)$
>= $(0.04\ f_c' / f_{sy})\ b\ d$

Provide the largest of these three magnitudes as A_s.

$(A_f + A_t)$ = 609.04 + 156.33 mm^2
= 765.35 mm^2

$(2 / 3\ A_{vf} + A_t)$ = 2 / 3 x 549.93 + 156.3: mm^2
= 522.94 mm^2

$(0.04\ f_c' / f_{sy})\ b\ d$ = 0.04 x 0.8 x 35 / 500 x 1000 x 540
= 1209.60 mm^2

Hence A_s = **1209.60** 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 >= 0.25 * A s

Provide the largest of

	>=	0.333 * A_{vf}		these two magnitudes as Ah.
0.25 * A s	=	0.25 x 1209.6		
	=	304.52	mm^2	
0.333*A_{vf}	=	0.333 x 549.93		
	=	183.12	mm^2	
Hence A_h	=	**304.52**	mm^2	

STEP VII

The total steel in vertical stirrups is " $\mathbf{A_v}$ "

V c	=	10 bd	in kgs	***where b & d are in cms***
	=	10 x 100 x 54		
	=	54000.00	kg	
	=	540.00	kN	
Pitch	=	400	mm	

$$A_v = \frac{0.50 * (V_u - V_c) * p}{f_{sy}\, d}$$

$$= \frac{0.5 \times (327.21 - 540) \times 400 \times 1000}{500 \times 540}$$

A_v = **0.00** mm^2

Reinforcement Details

Total Primary Tensile Reinforcement " A_s "

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

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

Horizontal Steel (Closed Stirrups) " A_h "

A_h	=	**304.52**	mm^2
A_h **/m**	=	**304.52**	mm^2 /m

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

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

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 As	=	302.4	mm^2		
Av 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

Self Weight of Dirt Wall	=	8.388	m^3 x	25.00	=	209.707 kN
Self Weight of Dirt Wall/ m	=	209.707 /		12.50	=	16.777 kN

1b. Live Load

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

Vertical Load on Dirt Wall = 200 kN

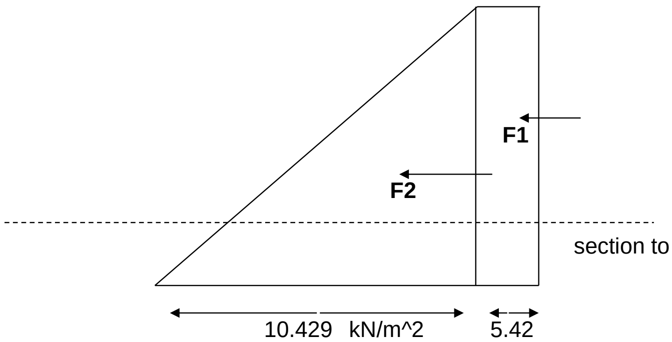
Braking Load

Assuming 20% braking Force i.e. 0.2 *200 = 40.000 kN acting at 1.2 m above deck

Effective Width = 2.79 m

Moment (due to Breaking) = $\dfrac{40.000 \times 3.509}{2.79}$ = 50.305 kNm/m

1c. EARTH PRESSURE



Height of dirt wall = 2.309 m

section to check Shear stress

Normal Earth Pressure

Earth Pressure Diagram

Intensity for rectangular portion	=	0.226	x 20	x 1.2	=	5.421	kN/m^2
F1	=	5.421	x 2.31	x 1.00	=	12.515	kN/m
Intensity for triangular portion	=	0.2259	x 20	x 2.309	=	10.429	kN/m^2
F2	=	0.50	x 10.43	x 2.309	x 1.00 =	12.039	kN/m

Moment @ RL **1978.21** m (at dirt wall base)

M1 = 12.515 x 1.154 = 14.447 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 = 12.039 x 0.970 = 11.674 kN.m/m

Design Horizontal Forces (Normal Case):

Load Factor For Live Load Surcharge = 1.2
Ultimate Moment due to Live Load Surcharge = 17.337 kN.m/m

Load Factor For Earth Pressure = 1.5
Ultimate Moment due to Earth Pressure = 17.512 kN.m/m

Load Factor For Braking Force = 1.5

Ultimate Moment due to Braking Force = 75.457 kN.m/m

Total Ultimate Moment = 110.305 kN.m/m

Material Property:

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	N/mm^2
Design Yield Strength of Reinforcement, fyd	=	434.783	N/mm^2
Modulus of Elasticity of Steel (Es)	=	200000	N/mm^2

(a) Vertical steel on earth face As per Clause 16.3.1 of IRC:112-2011

Adorting clear cover on either face	=	50 mm					
Minimum Dia of Reinforcement	=	16 mm					
Maximum Spacing of Steel	=	150 mm					
Thickness of dirtwall	=	0.300 m					
Available effective depth	=	300	-50	-8	=	242	mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 110.31 kNm/m

Effective Depth of Cap Required (dreq) = $\mathrm{SQRT}\left(\dfrac{110.31 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$ = 138.205 mm

Total Depth Required (Dreq) = 196.20 mm
Total Depth Provided (Dprov) = 300.00 mm **OK**

R= Mu/(b d^2) = 1.884

Area of Steel Required:

$\dfrac{pt}{100} = \dfrac{Ast_{req}}{b\,d} = \dfrac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2\ fy}$ = 0.005

Ast_{req} = 1122.248 mm^2/m

As per Clause 16.3.1 of IRC:112-2011

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

Maximum (Ast_{req}, Ast_{min}) = 1122.248 mm^2/m

Provide 16 mm dia bar @ 150 mm c/c as vertical steel at earth face.

Provide Ast = 1340 mm^2/m) OK

Percentage of Steel Provided = 0.554 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\dfrac{0.0035\ .\ d}{(0.0035 + fyd/ Es)}$ = $\dfrac{0.0035 \times 242}{0.0035 + 0.00217}$
= 149.2797 mm

Depth of Neutral Axis , = $\dfrac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$

= $\dfrac{435 \times 1340}{0.36 \times 35.00 \times 1000}$ = 46.276 mm **OK**

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

z = d - 0.416 . X = 242 - 19.251
= 222.749 mm

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

MR = fyd . Ast . z = 434.78 x 1340 x 222.75

= 1.3E+08 Nmm = 129.816 kNm/m > 110.31 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 280.562 mm^2/m
Governing Ast	=	280.562 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

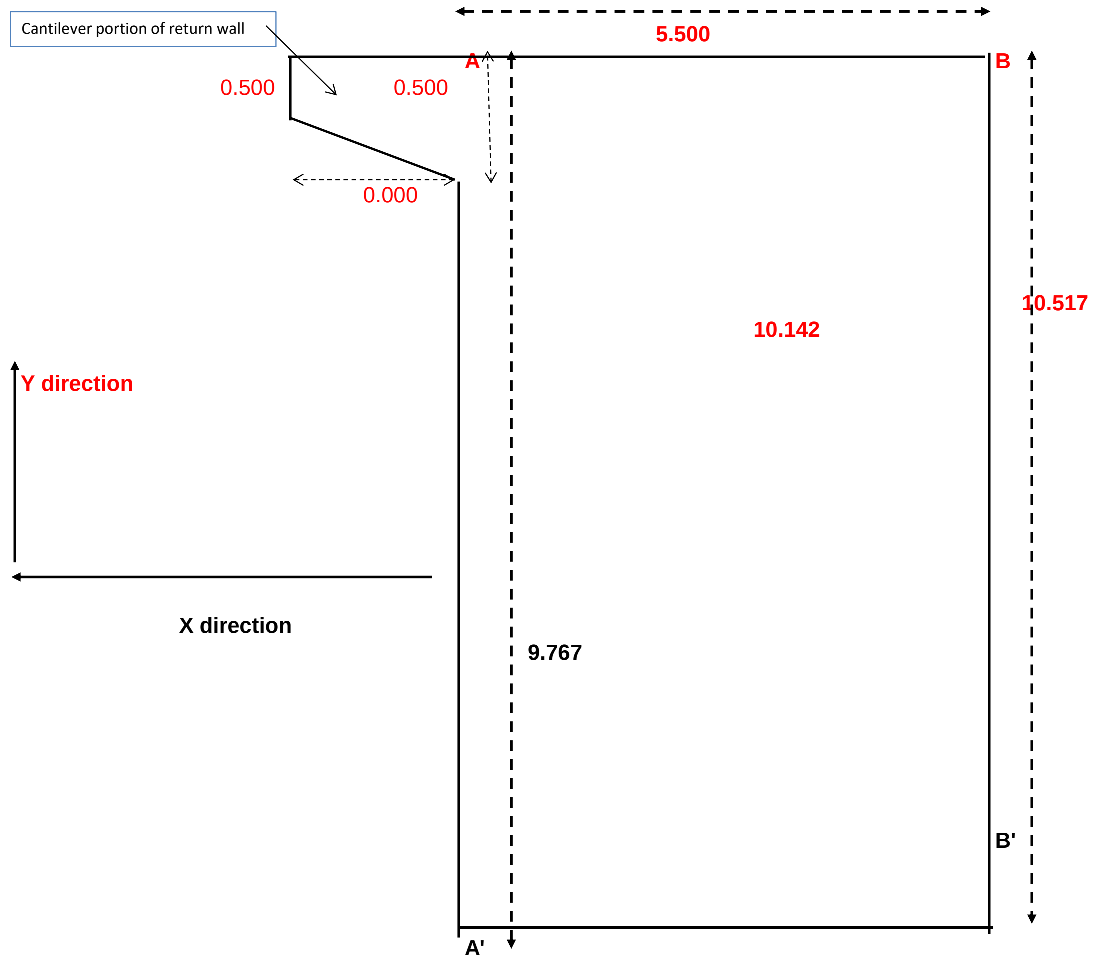
Provide 10 mm dia bar @ 150 mm c/c as vertical steel at earth face.

Provided Ast= 524 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	=	5.50	m
Avg. Heigth of Solid Return	b	=	10.142	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 = 0.542

a/b =	0.5	β1 =	0.631	β2 =	0.632
a/b =	0.75	β1 =	1.246	β2 =	1.186

For uniformly distributed load over entire plate

For, a/b = 0.542 β1 = 0.735

β2 = 0.726

Live Load Surcharge Intensity:

q = 0.2259 x 20.00 x 1.200 = 5.421 kN/m^2

Max. $.\sigma b = \dfrac{\beta 1 \times q \times b^2}{(t1)^2}$

$.\sigma a = \dfrac{\beta 2 \times q \times b^2}{(t2)^2}$

$.\sigma b = \dfrac{0.735 \times 5.421 \times 102.860}{0.250}$

At bottom edge = 1639.426 kN/m2 = **1.639 MPa**

For 1000 mm of width , Z = $\frac{1000 \times 250000}{6}$

= 4.17E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 1.639 x 4.167E+07

= 68309415 Nmm/m = **68.309 kN.m/m**

.σ a = $\frac{0.726 \times 5.421 \times 102.860}{0.250}$

= 1619 = **1.6186 MPa**

For 1000 mm of height , Z = $\frac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence, Moment /m height along X direction -

Mx /m height = 1.6186 x 4.167E+07 = 6.744E+07 Nmm/m

= **67.443 kN.m/m**

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.500	β1 =	0.328	β2 =	0.200
a/b =	0.75	β1 =	0.537	β2 =	0.276

For, a/b = 0.542 β1 = **0.363**

β2 = **0.213**

q = 0.226 x 20.00 x 10.14

= 45.815 kN/m^2

Max. .σ b = $\frac{\beta1 \times q \times b^2}{(t1)^2}$

.σ a = $\frac{\beta2 \times q \times b^2}{(t2)^2}$

σ b = $\frac{0.363 \times 45.815 \times 102.860}{0.25}$

= 6849.41 kN/m^2

= **6.849 MPa**

For 1000 mm of width , Z = $\frac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 6.849 x 4.167E+07

= 285392183 Nmm/m = **285.392 kN.m/m**

.σ a = $\frac{0.213 \times 45.815 \times 102.860}{0.25}$

= 4012.4 kN/m^2 = 4.012 MPa

For 1000 mm of height , Z = $\frac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence Moment /m height along X direction -

Mx /m height = 4.012 x 4.167E+07 = 1.672E+08 Nmm/m

= **167.184 kN.m/m**

Total Moment in Solid Return Wall / m height = 234.627 kN.m/m

Total Moment in Solid Return Wall / m width = 353.702 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	=	332	kN.m/m
Total Moment(My) in Solid Return Wall / m width	=	510	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) =

= **331.708 + 0.00**

= **331.71 kN.m / m**

Adorting clear cover on either face	=	75	mm		
Minimum Dia of Reinforcement	=	20	mm		
Maximum Spacing of Steel	=	150	mm		
Thickness of wall	=	0.500	m		
Available effective depth	=	500		-75	-10
	=	415	mm		

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 331.71 kNm/m

Effective Depth of Cap Required (dreq) = SQRT($\frac{331.71 \text{ x } 1000000}{0.165 \text{ x } 35.00 \text{ x } 1000}$)

Effective Depth of Cap Required (dreq) = 239.663 mm

Total Depth Required (Dreq) = 324.66 mm

Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 1.93

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2 fy}$

= 0.005

Ast_{req} = 1971.358 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011

= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1971.358 mm^2/m

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

Provide Ast= 2094 mm^2/m) OK

Percentage of Steel Provided = 0.505 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \text{ . d}}{(0.0035 + fyd/ Es)}$

$$= \frac{0.0035 \times 415}{0.0035 + 0.00217}$$

= 255.996 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

$$= \frac{434.78 \times 2094}{0.36 \times 35.00 \times 1000}$$

= 72.270 mm OK

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

z = d - 0.416 . X
= 415 - 30.064
= 384.936 mm

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

MR = fyd . Ast . z

= 434.78 x 2094 x 384.936

= 3.51E+08 Nmm

= 350.525 kNm/m > 331.71 kNm/m

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

Provide 12 mm dia bar @ 150 mm c/c as Horizontal steel at non earth face.

Provided Ast = **754 mm^2/m)**

2. Design for Face A'B'

Moment in Solid Return /m width = 510.06 kN.m / m

Adorting clear cover on either face	=	75 mm			
Minimum Dia of Reinforcement	=	25 mm			
Maximum Spacing of Steel	=	125 mm			
Thickness of wall	=	0.500 m			
Available effective depth	=	500	-75	-20	-12.5
	=	392.5 mm			

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 510.06 kNm/m

Effective Depth of Cap Required (dreq) = $SQRT\left(\frac{510.06 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

Effective Depth of Cap Required (dreq) = 297.190 mm

Total Depth Required (Dreq) = 384.69 mm
Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 3.31

Area of Steel Required:

$$\frac{pt}{100} = \frac{Ast_{req}}{b\,d} = \frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2\,fy}$$

= 0.009

Ast_{req} = 3411.077 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 3411.077 mm^2/m

Provide	**25 mm dia bar @**	**125 mm c/c as vertical steel at earth face.**	

Provide Ast=	**3927 mm^2/m)**	**OK**

Percentage of Steel Provided = 1.0005 %

Provide	**12 mm dia bar @**	**250 mm c/c as Vertical steel at non earth face.**

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm $= \dfrac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

$= \dfrac{0.0035 \times 392.5}{0.0035 + 0.00217}$

= 242.12 mm

Depth of Neutral Axis , X $= \dfrac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

$= \dfrac{434.78 \times 3927}{0.36 \times 35.00 \times 1000}$

= 135.507 mm OK

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

z = d - 0.416 . X
= 392.5 - 56.371
= 336.13 mm

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

MR = fyd . Ast . z

= 434.78 x 3927 x 336.129

= 5.74E+08 Nmm

= 573.903 kNm/m > 510.06 kNm/m

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

b) Cantilever Portion of Return Wall

Self-weight of cantilever portion of return wall	=	6	kN/m
Crash Barrier weight	=	10.0	kN/m
Total Load	=	16	kN/m
Moment at Cantilever Face	=	0	kNm
Load Factor	=	1.35	
Design Moment	=	0	kNm
Effective Depth	=	444.000	mm

R= Mu/(b d^2) = 0.00

Area of Steel Required:

$\dfrac{pt}{100} = \dfrac{Ast_{req}}{b\,d} = \dfrac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2\,fy}$

= 0.000

Ast_{req} = 0.000 mm^2

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 266.4 mm^2

Maximum (Ast_{req}, Ast_{min}) = 266.4000 mm^2

Provide	**12**	**4**	**=**	**452**	**mm^2**

DESIGN OF STEEL PLATE GIRDER

C/C of BEARINGS = 28.70 m

C/C of EXPANSION JOINT = 30.0 m

1.0 Design Report

This design note represent design of Steel Concrete Composite Superstructure of 30 m span.

1.1 Introduction:-

The Grillage analogy method has been used for analysis of superstructure.
The Ultimate limit state method has been followed for designing the components of bridges.

1.1.1 Unit System:-

Length	m
Force	kN
Stress	MPa
Moment	kNm
Hog Mom/Com Str	-ve
Sag Mom/Ten Str	+ve

1.1.2 General details of bridge:-

Overall Span	30.0 m
Effective Span	28.7 m
Carriageway width	11.00 m
Total Width	12.50 m
Skew Angle	0.00 deg
Proposed Girder Type	Welded Plate Girder
C/C of Girders	2.5 m
No. of Span	1

1.2 Reference documents :-

1 IRC codes /guidelines/special publications
2 MORTH specification
3 Specialised literature as relevant

1.3 Design Loading

The load which may arise during construction and service life is listed as following

1.3.1 Dead Loads

Dead load of the structure is calculated based on unit weights of materials given below

Structural Steel	78.50 kN/m3
Reinforced concrete	25.00 kN/m3
Green Weight of Concrete	26.00 kN/m3
Assume temporary weight due to shuttering sheet and sup	150 kg/m3
Deck slab Thickness =	0.25 m

1.3.1A Shuttering Weight

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

= 3.6 kN/m^2

1.3.2 Superimposed Dead Load

The thickness of wearing coat has been adopted as 65mm and density is considered as 22.00 kN/m3

Load Intensity due to wearing course = 2 kN/m^2

Weight of Crash Barrier = 10 kN/m

1.3.3 Live Load

Dynamic Impact Factor

Impact Factor for class A loading = 0.213

$$\frac{9}{13.5 + L} = 0.2133$$

Impact factor for class 70R Wheeled loading = 0.213

Congestion factor = 1.15

Live reduction factor = 0.90 for 3 lane

1.3.4 Braking Force

Not considered in the design of superstructure

1.3.5 Live load combinations

Live load combinations has been taken as per the provisions of IRC 6 -2017

1.3.6 Temperature Loading

Differential Temperature Rise

Due to rapid inflow of heat across the section causes differential temperature, differential temperature rise has been considered as per the fig 10(b) of IRC 6- 2017

Differential Temperature Fall

As the temperature falls, section flow the heat to the outside atmosphere which causes negative temperature variation across the section.

Differential temperature fall is considered as per the fig 10(b) of IRC 6- 2017

1.4 Assumptions:-

The following assumptions have been taken while designing.

1 On top slab 65mm thick wearing coat is assumed for SIDL.

2 Girders are assumed laterally unsupported for self wt. stage.

3 Bracings are designed for 50% of wind load. Remaining is shared by deck slab.

4 Modular ration of 15 for permanent loading and 7.5% for transient load is considered.

5 Stud connectors are designed for full shear interaction.

6 Load cycle of 2 x 10^6 of critical loading is consider in fatigue check

1.4 Loads:-

The different types of loads used as per IRC 6 : 2017 are.

1 Dead load.

2 In SIDL , crash barrier, Railing and wearing coat load is considered.

3 2 Lane of Class 70 R or 4 Lane of class A OR 1 lane of 70R + 2 Lane of Class A

1.5 Material properties

1 Grade of Concrete M40

2 Grade of Reinforcement Steel Fe500D

3 Grade of structural steel (Main Girders) E350 (As per table no. 2 of IS 2062:2006)

4 Grade of Secondary Members E350

INPUT DATA

DESIGN PARAMETER

GEOMETRIC DATA

Bridge Type	=	Steel Girder Composite		
No. of Steel Girders	=	5		
Skew angle	=	0	0	
Effective span (SQ)	=	28.70	m	
Projection beyond C/L of bearing (SQ)	=	0.650	m	
C/C of expansion gap (SQ)	=	30.00	m	
Girder projection from c/l of bearing (SQ)	=	0.500	m	
Carriageway width	=	11.00	m	
Overall width of bridge	=	12.50	m	
Spacing of main girder.	=	2.50	m	
No. of cross girder	=	5	nos	
Deck slab thickness at edge	=	0.250	m	
Deck slab thickness at median	=	0.250	m	
Thikcness of deck slab at girder location	=	0.250	m	
Deck slab thickness for effective width calculati	=	0.250	m	
Haunch size at grider top flange	=	0	0	m
Deck slab thickness.(Average) for effective widt	=	0.250	m	

Location of splice

1st splice from centre of bearing	=	9.000	m
2nd splice from centre of bearing	=	19.000	m

Sizes of main girder	I- type section - MID SPAN			
Web plate size (without slab)	=	1265	X 14	mm
Top Flange thickness Plate-1	=	400	X 30	mm
Top Flange thickness Plate-2	=	0	X 0	mm
Bottom Flange thickness Plate-1	=	400	X 30	mm
Bottom Flange thickness Plate-2	at 2nd splice location only	375	X 0	mm

Sizes of main girder	at 2nd splice location only			
Web plate size (without slab)	=	1265	X 14	mm
Top Flange thickness Plate-1	=	400	X 30	mm
Top Flange thickness Plate-2	=	0	X 0	mm
Bottom Flange thickness Plate-1	=	400	X 30	mm
Bottom Flange thickness Plate-2		0	X 0	mm

Sizes of main girder	I- type section -NEAR SUPPORT			
Web plate size (without slab)	=	1265	X 14	mm
Top Flange thickness Plate-1	=	400	X 30	mm
Top Flange thickness Plate-2	=	0	X 0	mm
Bottom Flange thickness Plate-1	=	400	X 30	mm
Bottom Flange thickness Plate-2	=	0	X 0	mm

Sizes of End cross girder	I- type section				
Web plate size (without slab)	=	1	1025	X 12	mm
Top Flange thickness Plate-1	=		200	X 12	mm
Top Flange thickness Plate-2	=		0	X 0	mm
Bottom Flange thickness Plate-1	=		200	X 12	mm
Bottom Flange thickness Plate-2	=		0	X 0	mm

sizes of End cross girder I- type section

Web plate size (without slab)	=	1076	X 12	mm
Top Flange thickness Plate-1	=	200	X 12	mm
Top Flange thickness Plate-2	=	0	X 0	mm
Bottom Flange thickness Plate-1	=	200	X 12	mm
Bottom Flange thickness Plate-2	=	0	X 0	mm

Width of Crash Barrier	=	0.50	m
Width of footpath	=	0.00	m
Cantilever side1	=	1.500	m
Cantilever side2	=	1.500	m
Unit weight of wet concrete	=	26.00	KN/m3
Depth of cross girder	=	1.325	m

PARTIAL SAFETY FACTOR

γ_m	=	1.10	For Structural Steel Against Yield Stress
γ_m	=	1.25	For Shear Connector

Grade of main steel		E430	
Yield stress (Main Girders)		430	Mpa
Grade of Secondary Members		330	Mpa

Grade of concrete	M	40	
Type of steel	HYSD	Fe 500	D

Ultimate strength of shear connector	For 25 mm dia Stud	146	kN
Fatigue strength of shear connector		27	kN
permissible stress in fillet weld		131	for Grade E300 and abov
permissible stress in fillet weld		108	for Grade upto E250

BENDING MOMENT SUMMARY AT MID SPAN

Stage I

Self weight of Girder alone

		COMBINATIONS					
		ULS		SLS			
		Basic	Accidental	Rare	Frequent	Quasi	
Partial Safety Factor		1.35	1	1		1	
Max Bending moment (Outer Girder) at mid	=	664.20	492.00	492.00			KN-m
Max Shear Force (Outer Girder) at Support	=	92.75	68.70	68.70			KN
Max Reaction (Outer Girder)	=	88.11	65.26	65.26			KN
Deflection (outer girder)	=					13.50	mm
Max Bending moment (Inner Girder)	=	664.20	492.00	492.00			KN-m
Max Shear Force (Inner Girder)	=	92.75	68.70	68.70			KN
Max Reaction (Inner Girder)	=	88.11	65.26	65.26			KN
Deflection inner girder	=					13.50	mm
Max Reaction at Bearing level	=	88.11	65.26	65.26			KN

Stage II

Concrete Load

		Basic	Accidental	Rare		Quasi	
Partial Safety Factor		1.35	1	1		1	
Max Bending moment (Outer Girder)	=	2479.95	1837.00	1837.00			KN-m
Max Shear Force (Outer Girder)	=	346.95	257.00	257.00			KN
Max Reaction (Outer Girder)	=	361.49	267.77	267.77			KN
Deflection (outer girder)						55.40	mm
Max Bending moment (Inner Girder)	=	2254.50	1670.00	1670.00			KN-m
Max Shear Force (Inner Girder)	=	314.55	233.00	233.00			KN
Max Reaction (Inner Girder)	=	328.62	243.43	243.43			KN
Deflection inner girder						50.35	mm
Max Reaction at Bearing level	=	361.49	267.77	267.77			KN

Erection load

		Basic	Accidental	Rare		Quasi	
Partial Safety Factor		1	1	-		-	
Max Bending moment (Outer Girder)	=	1017.00	1017.00	-		-	KN-m
Max Shear Force (Outer Girder)	=	142.00	142.00	-		-	KN
Max Reaction (Outer Girder)	=	148.00	148.00	-		-	KN
Max Bending moment (Inner Girder)	=	1017.00	1017.00	-		-	KN-m
Max Shear Force (Inner Girder)	=	142.00	142.00	-		-	KN
Max Reaction (Inner Girder)	=	148.00	148.00	-		-	KN
Max Reaction at Bearing level	=	148.00	148.00	-		-	KN

Stage III

Composite action stage

SIDL

wearing coat

		Basic	Accidental	Rare	Frequent	Quasi	
Partial Safety Factor		1.75	1	1.2		1.2	
Max Bending moment (Outer Girder)	=	719.25	411	493.2			KN-m
Max Shear Force (Outer Girder)	=	138.075	78.9	94.68			KN
Max Reaction (Outer Girder)	=	144.1423	82.367	98.8404			KN
Deflection (outer girder)						9.24	mm
Max Bending moment (Inner Girder)	=	680.75	389	466.8		0	KN-m
Max Shear Force (Inner Girder)	=	125.65	71.8	86.16		0	KN
Max Reaction (Inner Girder)	=	131.1153	74.923	89.9076		0	KN

Deflection inner girder						9.24	mm
Max Reaction at Bearing level	=	144.1423	82.367	98.8404		0	KN

Crash barrier + RCC Railing

		Basic	Accidental	Rare		Quasi	
Partial Safety Factor		1.35	1	1		1	
Max Bending moment (Outer Girder)	=	704.7	522	522			KN-m
Max Shear Force (Outer Girder)	=	147.15	109	109			KN
Max Reaction (Outer Girder)	=	201.636	149.36	149.36			KN
Deflection (outer girder)						13.95	mm

Max Bending moment (Inner Girder)	=	0	0	0			KN-m
Max Shear Force (Inner Girder)	=	0	0	0			KN
Max Reaction (Inner Girder)	=	0	0	0			KN
Deflection inner girder						0.00	mm
Max Reaction at Bearing level	=	201.64	149.36	149.36			KN

Live Load Combinations

Impact+Congestion factor +reduction 1.256

1-70R + 2-Class A OR 4 CLASS A

Partial Safety Factor		1.5	0.75	1	0.75	0.75	
Max Bending moment (Outer Girder)	=	3200.2	1600.1	2133.5	1600.1		KN-m
Max Shear Force (Outer Girder)	=	555.7	277.8	370.4	277.8		KN
Max Reaction (Outer Girder)		555.7	277.8	370.4	277.8		
Deflection						11.3	mm

Max Bending moment (Inner Girder)	=	2354.5	1177.3	1569.7	1177.3		KN-m
Max Shear Force (Inner Girder)	=	587.7	293.8	391.8	293.8		KN
Max Reaction (Outer Girder)		595.2	297.6	396.8	297.6		
Deflection						11.30	mm
Max Reaction at Bearing level	=	595.2	297.6	396.8	297.6		KN

SV Load

Impact+Congestion factor +reduction 1.000

Partial Safety Factor		1.15	1.15	1	0.75	0.75	
Max Bending moment (Outer Girder)	=	0.00	0.00	0.00	0.00		KN-m
Max Shear Force (Outer Girder)	=	0.00	0.00	0.00	0.00		KN
Max Reaction (Outer Girder)		0.00	0.00	0.00	0.00		
Deflection outer girder						0.00	mm

Max Bending moment (Inner Girder)	=	3382.15	3382.15	2941.00	2205.75		KN-m
Max Shear Force (Inner Girder)	=	748.65	748.65	651.00	488.25		KN
Max Reaction (Outer Girder)		747.50	747.50	650.00	487.50		
Deflection inner girder						0	mm
Max Reaction at Bearing level	=	747.50	747.50	650.00	487.50		KN

Max Live loads with impact factor +congestion factor

		Basic	Accidental	Rare	Frequent	
Max Bending moment (Outer Girder)	=	3200.24	1600.12	2133.49	1600.12	KN-m
Max Shear Force (Outer Girder)	=	555.66	277.83	370.44	277.83	KN
Max Reaction (Outer Girder)		555.66	277.83	370.44	277.83	

Max Bending moment (Inner Girder)	=	3382.15	3382.15	2941.00	2205.75	KN-m
Max Shear Force (Inner Girder)	=	748.65	748.65	651.00	488.25	KN
Max Reaction (Inner Girder)		747.50	747.50	650.00	487.50	

Max Reaction at Bearing level	=	747.50	747.50	650.00	487.50	KN

DL with Erection Load

		Basic	Accidental	Rare	
Max Bending moment (Outer Girder)	=	4161.15	3346.00	2329.00	KN-m
Max Shear Force (Outer Girder)	=	581.70	467.70	325.70	KN
Max Bending moment (Inner Girder)	=	3935.70	3179.00	2162.00	KN-m
Max Shear Force (Inner Girder)	=	549.30	443.70	301.70	KN
Max Reaction at Bearing level	=	597.59	481.03	333.03	KN

DL+SIDL

		Basic	Accidental	Rare	Frequent	
Max Bending moment (Outer Girder)	=	4568.10	3262.00	3344.20		KN-m
Max Shear Force (Outer Girder)	=	724.92	513.60	529.38		KN
Max Bending moment (Inner Girder)	=	3599.45	2551.00	2628.80		KN-m
Max Shear Force (Inner Girder)	=	532.95	373.50	387.86		KN
Max Reaction at Bearing level	=	547.85	383.61	398.60		KN

SIDL+ LL

Max Bending moment (Outer Girder)	=	4624.19	2533.12	3148.69	KN-m
Max Shear Force (Outer Girder)	=	840.89	465.73	574.12	KN
Max Bending moment (Inner Girder)	=	4062.90	3771.15	3407.80	KN-m
Max Shear Force (Inner Girder)	=	874.30	820.45	737.16	KN
Max Reaction at Bearing level	=	1093.28	979.23	898.20	KN

Total Bending Moment For Critical Case

DL+SIDL+LL	For Outer Girder	=	7768.34	4862.12	5477.69	KN-m
DL+SIDL+LL	For Inner Girder	=	6981.60	5933.15	5569.80	KN-m

Total Shear Force For Critical Case

DL+SIDL+LL -Outer Girder	=	1280.58	791.43	899.82	KN
DL+SIDL+LL -Inner Girder	=	1281.60	1122.15	1038.86	KN

Total Bearing Reaction For Critical Case

DL+SIDL+LL -Outer Girder	=	1542.87	1312.26	1231.23	KN-m
DL+SIDL+LL -Inner Girder	=	416.73	308.69	308.69	KN

DESIGN OF PLATE GIRDER FROM ULTIMATE LIMIT STATE METHOD

Design For Shear

Max Reaction at Bearing Location = 1542.87 KN

Max Shear force for composite section = 1281.60 KN

Material Properties

fy = 430 Mpa.

Partial Safety Factor γ_m = 1.1

Max permissible bending stress in tension or compression.

$$s_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = 391 \text{ Mpa}$$

$$\varepsilon = \sqrt{\frac{250}{fy}} = 0.762$$

Average Shear Stress = fy ÷ γ_m

= 430 ÷ 1.1

= 390.909 MPa

Min. Dimension Requirement

$$\text{Min overall depth} = \frac{L}{25} = \frac{28700}{25} = 1148 \text{ mm}$$

Overall depth of girder Provided excluding deck slab = 1325 mm

Hence OK

$$\text{Min Spacing} = \frac{L}{20} = \frac{28700}{20} = 1435 \text{ mm}$$

Spacing provided = 2500 mm

Hence OK

Top Flange thickness Plate-1	=	30	mm
Top Flange thickness Plate-2	=	0	mm
Bottom Flange thickness Plate-1	=	30	mm
Bottom Flange thickness Plate-2	=	0	mm
Top flange width -plate 1	=	400	mm
Top flange width -plate 2	=	0	mm
bottom flange width- plate 1	=	400	mm
bottom flange width- plate 2	=	0	mm
Depth of the Web Plate	=	1265	mm
Thickness of the web	=	14	mm

Check for Section Class :

Flange Class				Web Class			
b	=	193	mm	d	=	1265	
tf	=	30	mm	tw	=	14	
b/tf	=	6.43	**Compact**	d/tw	=	90.36	**Semicompact**

Calculation of Effective Cross section Refer clause 603.1.3 of IRC:22-2015

tw = 14.00 mm

fy = 430 Mpa

ε = 0.76

$20\,\varepsilon\,.\,t_w$	=	213.50 mm	Part adjacent to the compression flange
$20\,\varepsilon\,.\,t_w$	=	213.50 mm	Part adjacent to the plastic neutral axis
depth of plastic neutral axis	=	662.50 mm	
Ineffetcive portion of web	=	0.00 mm	no requirement of reduction
			Refer class 603.1.3

Now after deducting the ineffective portion of web

d	=	**1265.00**	mm	
d/tw	=	90.36	**Semicompact**	

web plate of size = **1265** **x** **14**

Top Flange Area -

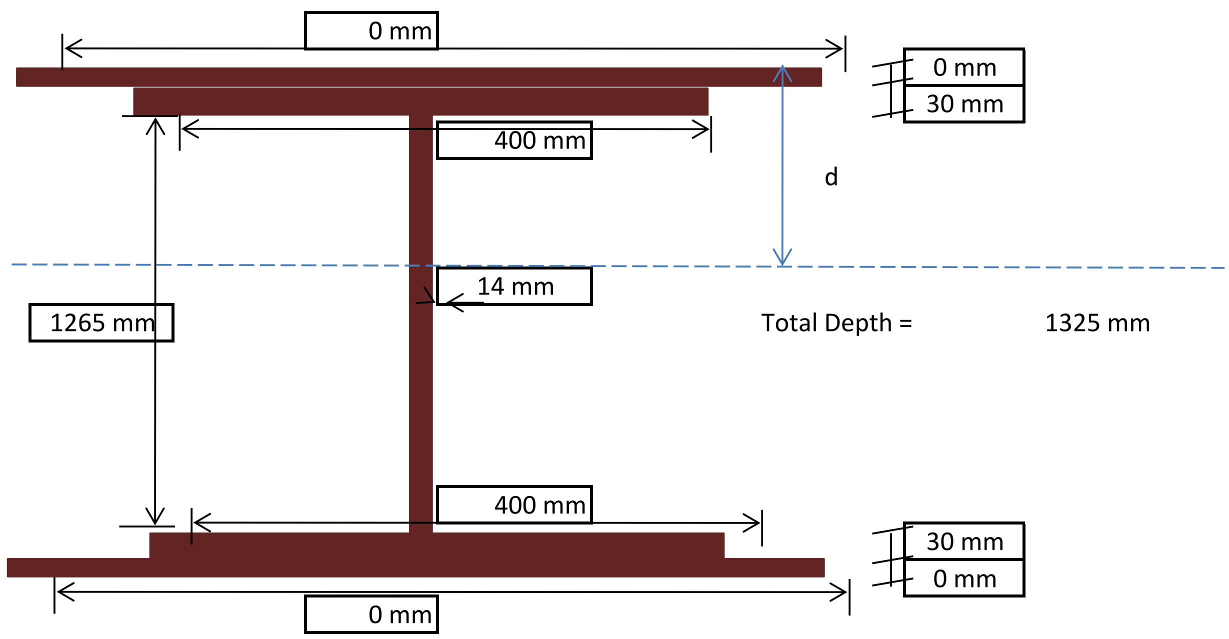
I)Top Plate-1	**400**	**x**	**30**	=	12000	mm^2
II) Top Plate-2	**0**	**x**	**0**	=	0	mm2

A_f	=	12000	mm^2

Bottom Flange Area -

I) Bottom Plate-1	**400**	**x**	**30**	=	12000	mm^2
II)Bottom Plate-2	**0**	**x**	**0**	=	0	mm^2

A_f	=	12000	mm^2



CALCULATION OF SECTIONAL PROPERTIES AFTER DEDUCTION OF INEFFECTIVE WEB AREA

Elastic Properties

SR. NO.	ELEMENT		Ai(mm^2)	Ye (mm) (from top)	AYi	AY^2s	Ixx (self)	It = AY^2s + Ixx
deduction of ineffective web			0	243.50	0	0.0E+00	0.0E+00	0.0E+00
		Σ	41710.00		2.8E+07	2.8E+10	2.4E+09	3.1E+10

Therefore distance of N.A. from top fibre

$$= \frac{\Sigma Ay_c}{\Sigma A} = \frac{2.8E+07}{41710.00} = \boxed{662.50} \text{ mm}$$

M.I. About N. A. = 3.1E+10 - 41710.00 x (6.63E+02)^2

I_{xx} = $\boxed{1.24E+10}$ mm^4

y_b = D - y_t

= 1325 - 662.50 = $\boxed{662.50}$ mm

$$Z_b = \frac{I_{xx}}{y_b} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07} \ mm^3$$

$$Z_t = \frac{I_{xx}}{y_t} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07} \ mm^3$$

Elastic section modulus (Ze) = $\boxed{1.88E+07}$ mm^3

Plastic Properties

Assume distance of neutral axis from Top of top flange is d.

d = 662.50 mm

Area above neutral axis (A1)

A1 =

= 20855 mm^2

Area below neutral axis (A2)

A2 =

= 20855 mm^2

Moment balance about neutral axis

A2 - A1 = $\boxed{0}$

Area in Compression

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Top Flng 1	12000	648	7770000
Top Flng 2	0	663	0
web	8855	316.25	2800394
ineffective w	0	419.00	0
	20855		1.06E+07

Area in Tension

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Bottom Flng 1	12000	648	7770000
Bottom Flng 2	0	663	0
web	8855	316	2800394
	20855		10570394

Y1 = $\frac{\Sigma A_i Y_i}{\Sigma A_i}$ Y2 = $\frac{\Sigma A_i Y_i}{\Sigma A_i}$

= 506.85 mm = 506.85 mm

Plastic section modulus (Zp) = A1*Y1 + A2*Y2 = 2.1E+07 mm^3

Dimension Check

Minimum Web Thickness

a.) For Vertical Stiffener

Serviceability Requirement

$\frac{d}{t_w} \leq 200\varepsilon$

t_w = 8.30 < 14.00

Vertical Stiffener not required

b.) For Horizontal Stiffener

$\frac{d}{t_w} \leq 200\varepsilon$ $3d \geq c \geq d$

$\frac{c}{t_w} \leq 200\varepsilon$ $0.74d \leq c < d$

$\frac{d}{t_w} \leq 270\varepsilon$ $c < d$

t_w = 6.14 < 14.00

Horizontal Stiffener not required

To avoid Flange Buckling

$\frac{d}{t_w} \leq 345\varepsilon_f^2$ Stiffener not provided

$\frac{d}{t_w} \leq 345\varepsilon_f^2$ $c \geq 1.5\ d$

$\frac{d}{t_w} \leq 345\varepsilon_f$ $c < 1.5\ d$

t_w = 4.81 < 14.00

Hence OK

DESIGN OF WEB FOR SHEAR :

Max Shear force at support of composite section = 1281.60 KN

a.) Shear Capacity

$V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

$A_v = d*t_w$ (For Welded Section)

V_n = 4396.70 KN > 1281.60 KN

Safe in Shear

b.) Shear Buckling Design

Simple Post - Critical Method

K_v = 5.35 When stiffeners are provided only at support

= $4.0 + 5.35/(c/d)^2$ = 4.00 For c/d < 1.0

= $5.35 + 4.0/(c/d)^2$ = 5.35 For c/d $\geq$ 1.0

c / d = 0.00

K_v = 5.35

$$\tau_{cr,e} = \frac{K_v \pi^2 E}{12(1-\mu^2)(d/t_w)^2} = 118.45 \text{ Mpa}$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\ \tau_{cr,e}}} = 1.45$$

i.) $\lambda_w \leq 0.8$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}} = 248.26 \text{ Mpa}$$

ii.) $0.8 \leq \lambda_w \leq 1.2$

$$\tau_b = [1-0.8(\lambda_w-0.8)] \times \frac{f_{yw}}{\sqrt{3}} = 119.62 \text{ Mpa}$$

iii.) $\lambda_w \geq 1.2$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\ \lambda_w^2} = 118.45 \text{ Mpa}$$

Therefore, τ_b = 118.45 Mpa

Shear force for web buckling

V_n = V_{cr} = $Av*\tau_b$ = 1907.05 KN > 1281.60 KN

Safe in Buckling

Design of Bearing Stiffener

Note: Same stiffener performs the function of load carrying as well as bearing stiffener

Max factored reaction at bearing = 1542.9 kN

Check for web buckling

Width of bearing b_1 = 400 mm

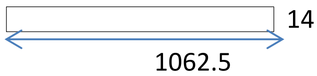
Depth of Girder h = 1325 mm

Dispersion of load at 45° to the level

of half the depth of cross section n_1 = h/2 = 662.5 mm

$b_1 + n_1$ = 1062.5 mm

Buckling stress of web σ_b = 47.2 MPa



I_y = 242958.333 mm^4

A = 14875 mm^2

ry = 4.04 mm

Buckling strength of web F_{cdw} = $(b_1+n_1)*t_w*\sigma_b$

= 967.25 kN

< 1542.9 kN

Web is not safe in buckling-Hence Load Carrying required

Check for local crushing of web

Total depth of bottom flange t_{fb} = 30 mm

Dispersion of load at through the flange

to the web junction at 1:2.5 slope n_2 = $2.5*\ t_{fb}$

= 75 mm

γ_{m0} = 1.1

f_{yw} = 430 MPa

Local crushing strength of web F_{cdw} = $(b_1+n_2)*t_w*f_{yw}/\gamma_{m0}$

= 2599.545 kN

\> 1542.9 kN

Web is safe in local crushing-Hence bearing stiffener not required

Bearing Stiffner

Try 2 plates 200 mm wide as the stiffener

Bearing strength $f_y/(0.8*\gamma_{m0})$ = 488.636 MPa

Bearing area required = $\frac{1542871.45}{488.636}$ = 3157.5044 mm^2

Provide plate thickness = 16

permissible outstand of plate = 20 tqε = 244.0 > 200 Safe

14 tqε = 170.8 > 200

Therefore, Outstand for calculating bearing area = 171 mm

Bearing area provided = 5465.549 > 3157.5 Safe

Gross Cross Sectional Area available

A = 2 x 1366.387 + (280 x 32)

= 11692.774 mm2

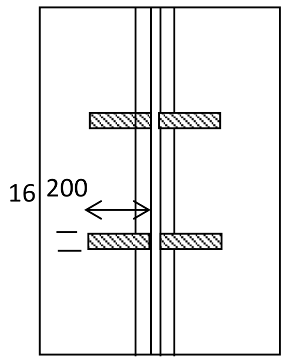
Length of web plate which acts along with stiffener plates in

in bearing the reaction = 280 mm

Least Moment of Inertia about the centre line of web

I = 191.97E+6 mm^4

PLAN



$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1.92E+08}{11692.77438}} = \boxed{128.13}\ mm$$

Effective Length

L_{eff} = 0.65 x d = 0.65 x 1265
= $\boxed{822.25}$ mm

l = l/r = 6.42

Buckling Strength σ_{ac} = $\boxed{414.874}$ N/mm^2

$$\text{Area required} = \frac{1542.871 \times 1000}{414.874}$$

= $\boxed{3719}$ mm^2 < $\boxed{11692.774}$ mm^2 Hence OK

DESIGN OF JOINT BETWEEN FLANGE PLATE & WEB : -

Assuming size of the fillet weld = 10 mm

The permissible stress in the fillet weld = 131 Mpa

The shear strength of the fillet weld

P = Pq . L . t

= Pq . t N/mm (t = Throat Thickness)

= 104.8 x 0.707 x 10

= $\boxed{740.94}$ N/mm.

First moment of area of Flangle plate @ C.G. is top plate

Q = 400 x 30 x (662.5 - 15.0)

= $\boxed{7.8E+06}$ mm^3

The shear flow is ,

$$q = \frac{VQ}{I}$$

$$= \frac{1281.6 \times 1000 \times 7.8E+06}{1.2E+10}$$

= $\boxed{801.41}$ N/mm

Providing 10 mm fillet weld whose strength is = $\boxed{740.94}$ N/mm.

be used at both faces, then the required length of weld for 1000 mm length of girder is ,

$$\text{weld lengt} = \frac{801.41 \times 1000}{2 \times 740.94}$$

= $\boxed{540.8}$ mm/1000 mm Length

However Provide 10.00 mm fillet weld continous throughout the length. **safe**

First moment of area of Flangle plate @ C.G. is bottom plate

Q = 400 x 30 x (662.5 - 15.0)

= $\boxed{7.8E+06}$ mm^3

The shear flow is ,

$$q = \frac{VQ}{I}$$

$$= \frac{1281.6 \times 1000 \times 7.8E+06}{1.2E+10}$$

= $\boxed{801.41}$ N/mm

Providing 10 mm weld whose strength is = $\boxed{740.94}$ N/mm.
be used at both faces, then the required length of weld for 1000 mm length of girder is ,

$$\text{weld lengt} = \frac{801.41 \times 1000}{2 \times 740.94}$$

= $\boxed{540.8}$ mm/1000 mm Length

However Provide 10 mm fillet weld continous throughout the length. safe

SECTIONAL PROPORTIES AT MID SECTION

Material Properties

fy = 430 Mpa.

Partial Safety Factor γ_m = 1.1

Max permissible bending stress in tension or compression.

$$s_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = 390.909 \text{ Mpa}$$

$$\varepsilon = \sqrt{\frac{250}{fy}} = 0.762$$

Average Shear Stress = fy ÷ γ_m

= 430 ÷ 1.1

= 390.909 MPa

Top Flange thickness Plate-1	=	30	mm
Top Flange thickness Plate-2	=	0	mm
Bottom Flange thickness Plate-1	=	30	mm
Bottom Flange thickness Plate-2	=	0	mm
Depth of the Web Plate	=	1265	mm
Thickness of the web	=	14	mm

Web Area -

web plate of size = **1265 mm** x **14 mm**

Top Flange Area -

I)Top Plate-1	**400**	x	**30 mm**	=	12000	mm^2
II) Top Plate-2	**0**	x	**0 mm**	=	0	mm2

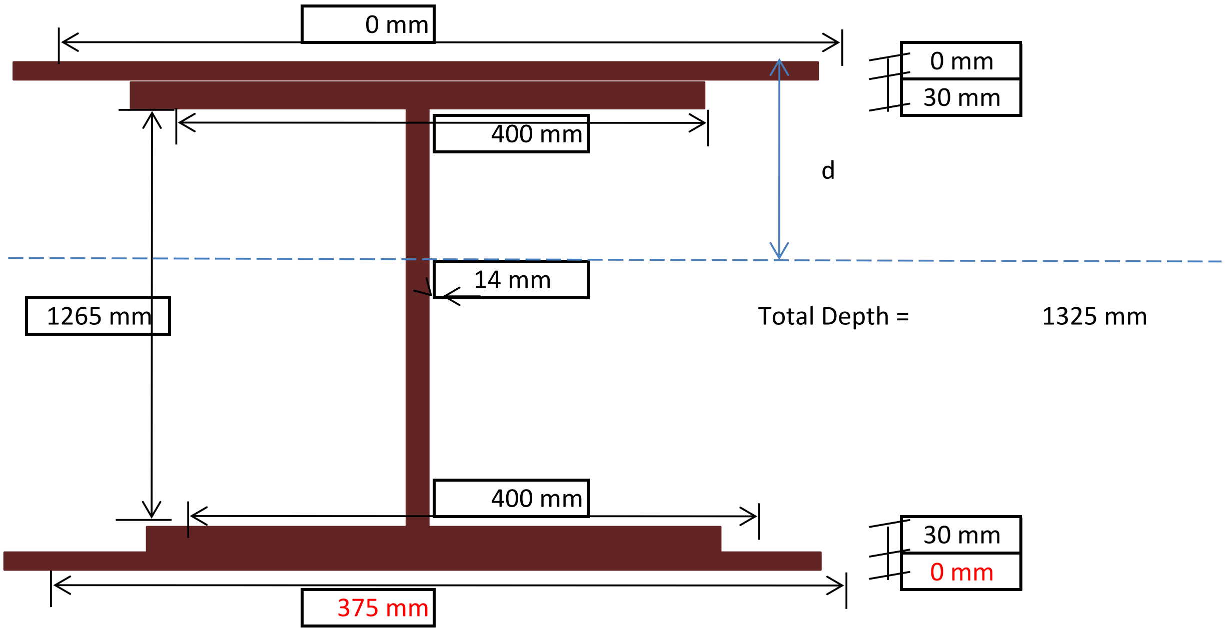
A_f = 12000 mm^2

Bottom Flange Area -

I) Bottom Plate-1	**400**	x	**30**	=	12000	mm^2
II)Bottom Plate-2	**375**	x	0 mm	=	0	mm^2

A_f = 12000 mm^2

For Mid Span Section



Check for Section Class :

Flange Class				Web Class			
b	=	193	mm	d	=	1265	
tf	=	30	mm	tw	=	14	
b/tf	=	6.43	**Compact**	**d/tw**	=	90.36	**Semicompact**

CALCULATION OF SECTIONAL PROPERTIES

Elastic Properties

SR. NO.	ELEMENT	Ai(mm^2)	Ye (mm) (from top)	AYi	AY^2s	Ixx (self)	It = AY^2s + Ixx
1	Top Flange Plate-1	12000	15	180000	2.7E+06	9.0E+05	3.6E+06
2	Top Flange Plate-2	0	0	0	0.0E+00	0.0E+00	0.0E+00
3	Web	17710	663	11732875	7.8E+09	2.4E+09	1.0E+10
4	Bottom Flange Plate-1	12000	1310	15720000	2.1E+10	9.0E+05	2.1E+10
5	Bottom Flange Plate-2	0	1325	0	0.0E+00	0.0E+00	0.0E+00
	Σ	41710.00		2.8E+07	2.8E+10	2.4E+09	3.1E+10

Therefore distance of N.A. from top fibre

$$= \frac{\Sigma Ay_c}{\Sigma A} = \frac{2.8E+07}{41710.00} = \boxed{662.50}\ \text{mm}$$

M.I. About N. A. $= 3.1E+10 - 41710.00 \times (6.63E+02)^{\wedge 2}$

$I_{xx} = \boxed{1.24E+10}\ mm^4$

$y_b = D - y_t$

$= 1325 - 662.50 = \boxed{662.50}\ \text{mm}$

$$Z_b = \frac{I_{xx}}{y_b} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07}\ mm^3$$

$$Z_t = \frac{I_{xx}}{y_t} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07}\ mm^3$$

Elastic section modulus (Ze) $= \boxed{1.88E+07}\ mm^3$

Plastic Properties

Assume distance of neutral axis from Top of top flange is d.

d = 662.50 mm

Area above neutral axis (A1)

A1 = (0 x 0 + 400 x 30 + 14 x (d-0-30)

= 20855 mm^2

Area below neutral axis (A2)

A2 = (375 x 0 + 400 x 30 + 14 x (1325 - d-0-30))

= 20855 mm^2

Moment balance about neutral axis

A2 - A1 = 0

Area in Compression

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Top Flng 1	12000	648	7770000
Top Flng 2	0	663	0
web	8855	316.25	2800394
	20855		1.06E+07

Area in Tension

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Bottom Flng 1	12000	648	7770000
Bottom Flng 2	0	663	0
web	8855	316	2800394
	20855		10570394

$$Y1 = \frac{\Sigma A_i Y_i}{\Sigma A_i}$$

= 506.85 mm

$$Y2 = \frac{\Sigma A_i Y_i}{\Sigma A_i}$$

= 506.85 mm

Plastic section modulus (Zp) = A1*Y1 + A2*Y2 = 2.1E+07 mm^3

SECTION PROPERTY AFTER CHANGING SECTION FROM SLENDER MEMBR TO SEMICOMPACT MEMBER

Max Reaction at Bearing Location = 1542.87 KN

Max Shear force for composite section = 1281.60 KN

Material Properties

fy = 430 Mpa.

Partial Safety Factor γ_m = 1.1

Max permissible bending stress in tension or compression.

$$s_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = 391 \text{ Mpa}$$

$$\varepsilon = \sqrt{\frac{250}{fy}} = 0.762$$

Average Shear Stress = fy ÷ γ_m

= 430 ÷ 1.1

= 390.909 MPa

Min. Dimension Requirement

Min overall depth = $\frac{L}{25} = \frac{28700}{25}$ = 1148 mm

Overall depth of girder Provided excluding deck slab = 1325 mm

Hence OK

Min Spacing = $\frac{L}{20} = \frac{28700}{20}$ = 1435 mm

Spacing provided = 2500 mm

Hence OK

Top Flange thickness Plate-1	=	30	mm
Top Flange thickness Plate-2	=	0	mm
Bottom Flange thickness Plate-1	=	30	mm
Bottom Flange thickness Plate-2	=	0	mm
Top flange width -plate 1	=	400	mm
Top flange width -plate 2	=	0	mm
bottom flange width- plate 1	=	400	mm
bottom flange width- plate 2	=	375	mm
Depth of the Web Plate	=	1265	mm
Thickness of the web	=	14	mm

Check for Section Class :

Flange Class				Web Class			
b	=	193	mm	d	=	1265	
tf	=	30	mm	tw	=	14	
b/tf	=	6.43	**Compact**	d/tw	=	90.36	**Semicompact**

Calculation of Effective Cross section Refer clause 603.1.3 of IRC:22-2015

tw = 14.00 mm

fy = 430 Mpa

ε	=	0.76	
20 ε . t_w	=	213.50 mm	Part adjacent to the compression flange
20 ε . t_w	=	213.50 mm	Part adjacent to the plastic neutral axis
depth of plastic neutral axis	=	662.50 mm	
Ineffetcive portion of web	=	0.00 mm	no requirement of reduction
			Refer class 603.1.3

Now after deducting the ineffective portion of web

d	=	**1265.00** mm	
d/tw	=	90.36	**Semicompact**

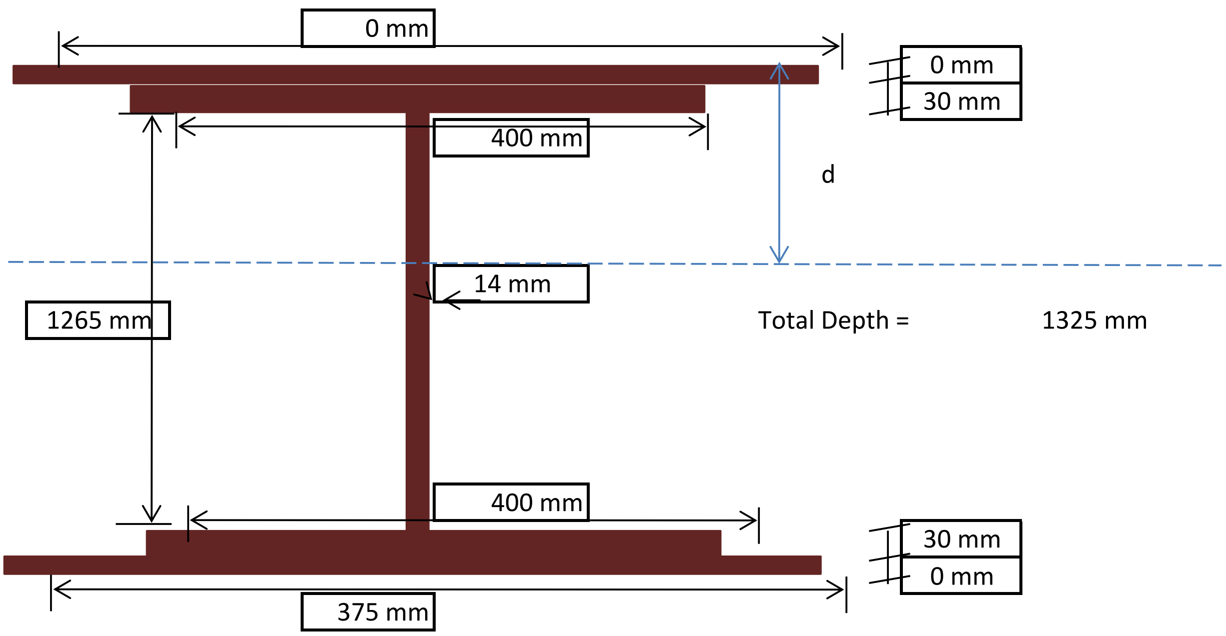
web plate of size = **1265** x **14**

Top Flange Area -

I)Top Plate-1	**400**	**x**	**30**	=	12000 mm^2
II) Top Plate-2	**0**	**x**	**0**	=	0 mm2
			A_f	=	12000 mm^2

Bottom Flange Area -

I) Bottom Plate-1	**400**	**x**	**30**	=	12000 mm^2
II)Bottom Plate-2	**375**	**x**	**0**	=	0 mm^2
			A_f	=	12000 mm^2



Total Depth = 1325 mm

CALCULATION OF SECTIONAL PROPERTIES AFTER DEDUCTION OF INEFFECTIVE WEB AREA

Elastic Properties

SR. NO.	ELEMENT		$Ai(mm^2)$	Ye (mm) (from top)	AYi	AY^2s	Ixx (self)	It = AY^2s + Ixx
deduction of ineffective web			0	243.50	0	0.0E+00	0.0E+00	0.0E+00
		Σ	41710.00		2.8E+07	2.8E+10	2.4E+09	3.1E+10

Therefore distance of N.A. from top fibre

$$= \frac{\Sigma Ay_c}{\Sigma A} = \frac{2.8E+07}{41710.00} = \boxed{662.50} \text{ mm}$$

M.I. About N. A. = 3.1E+10 - 41710.00 x (6.63E+02)^2

I_{xx} = $\boxed{1.24E+10}$ mm^4

y_b = D - y_t

= 1325 - 662.50 = $\boxed{662.50}$ mm

$$Z_b = \frac{I_{xx}}{y_b} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07} \; mm^3$$

$$Z_t = \frac{I_{xx}}{y_t} = \frac{1.2E+10}{662.50} = \boxed{1.9E+07} \; mm^3$$

Elastic section modulus (Ze) = $\boxed{1.88E+07}$ mm^3

Plastic Properties

Assume distance of neutral axis from Top of top flange is d.

d = 662.50 mm

Area above neutral axis (A1)

A1 =

= 20855 mm^2

Area below neutral axis (A2)

A2 =

= 20855 mm^2

Moment balance about neutral axis

A2 - A1 = $\boxed{0}$

Area in Compression

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Top Flng 1	12000	648	7770000
Top Flng 2	0	663	0
web	8855	316.25	2800394
ineffective v	0	419.00	0
	20855		1.06E+07

Area in Tension

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Bottom Flng 1	12000	648	7770000
Bottom Flng 2	0	663	0
web	8855	316	2800394
	20855		10570394

Y1 = $\frac{\Sigma A_i Y_i}{\Sigma A_i}$ = 506.85 mm

Y2 = $\frac{\Sigma A_i Y_i}{\Sigma A_i}$ = 506.85 mm

Plastic section modulus (Zp) = A1*Y1 + A2*Y2 = 2.1E+07 mm^3

DESIGN STAGE I UNDER SELF WEIGHT AT MID SPAN

Max Bending Moment on Outer Girder due to self wt.(M) : 664.20 KNm

(Note: Section is assumed as laterally unsupported beams for Self wt. Stage)

Design bending strength of beam (M_d):

$$M_d = \beta_b Z_p f_{bd}$$

Elastic critical moment (M_{cr}):

$$M_{cr} = \beta_b Z_p f_{cr,b}$$

Elastic section modulus (Ze) = 1.88E+07 mm^3

Plastic section modulus (Zp) = 2.11E+07 mm^3

β_b = 8.87E-01 (1 For Plastic/Compact section, Ze/Zp for semicompact)

Effective length for lateral torsional buckling(L_{LT}) = 0.75*L

21.53 m

Radius of gyration of longitundinal girdei $= \sqrt{\dfrac{I}{A}}$

= 545.81 mm

= 0.55 m

Slenderness ratio $= \dfrac{kL}{r}$ 39.44

h_f = 1.295

t_f = 0.03

h_f/t_f 43.17

$f_{cr,b}$ = 1891.0 Mpa

M_{cr} = 3.55E+04 KNm

λ_{LT} = 0.477

$\phi_{LT} = 0.5(1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}{}^2)$

α_{LT} = 0.49 For welded section

ϕ_{LT} = 0.682

$$\chi_{LT} = \frac{1}{\{\phi_{LT}+[\phi_{LT}^2-\lambda_{LT}^2]^{0.5}\}}$$

= **0.856**

f_{bd} = $\chi_{LT}\,(f_y/\gamma_{m0})$ = **334.56** Mpa

M_d = **6274.8** KNm

> 664.20 KNm

Hence OK

DESIGN STAGE II GREEN CONCRETE STAGE AT MID SPAN

(i.e. Green concrete stage) : -

Assuming the girder is laterally supported throughout the span

Max Bending Moment (self weight, Green concrete weight and temporary load)

= **4161.15** kNm

Elastic section modulus (Ze)	=	1.88E+07	mm^3
Plastic section modulus (Zp)	=	2.11E+07	mm^3
β_b =	=	0.887	(1 For Plastic/Compact section, Ze/Zp for semicompact)

Design bending strength

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_m}$$

= **7331.75** KNm

Design bending strength

M_d = **7331.75** KNm

> 4161.15 KNm

Hence OK

FOR THE COMPOSITE SECTION AT MID SPAN : - OUTER GIRDER

(i.e. after casting of Deck slab) : -

Max permissible bending stress in tension or compression.

$\sigma_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = \boxed{390.909}$ Mpa

Effective Width of Conceret Slab

For outer girder $b_{eff} = \frac{L_0}{8} + X$

$\frac{L_0}{8} \leq \frac{B}{2}$ and $X \leq \frac{B}{2}$

$3.6 \geq 1.25$ $\quad$ $1.50 \geq 1.25$

Hence,

$b_{eff} = 1.25 + 1.25$

$= \boxed{2.5}$ m

Modular Ratio

Modular Ratio for Composite Section with Prefabricated units in Steel for Transient Loads

$m = \frac{E_s}{E_c} = \frac{\text{Modulas of Elasticity of steel of girder}}{\text{Modulas of Elasticity of Cast-in-situ concrete at 28-days}}$

But $\frac{E_s}{E_c} \geq \boxed{7.5}$ For Transient Loading

$\frac{E_s}{K_cE_c} \geq \boxed{15}$ For Permanent loading

K_c Creep factor $= \boxed{0.5}$

$\frac{E_s}{E_c} = \frac{200000}{31623} = \boxed{6.32} \leq 7.5$

$m = \boxed{7.5}$ For Transient Loading

$\frac{E_s}{K_cE_c} = \frac{200000}{15811.388} = \boxed{12.65} \leq 15$

$m = \boxed{15}$ For Permanent loading

For the calculation of equivalent area of deck, divide the effective width of concrete slab by modular ratio

Effective Width of Concrete Slab for outer Girder For Permanent Loading

$\frac{2500.00}{15.00} = \boxed{166.67}$ mm

Max . Bending Moment due to DL+SIDL = $\boxed{4568.10}$ kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**Outer Girder**	41710	912.50	3.8E+07	3.5E+10	1.2E+10	4.72E+10
2	**Deck Slab**	41666.7	125.00	5.2E+06	6.5E+08	2.2E+08	8.68E+08
	Σ	83376.7		4.3E+07			4.80E+10

Therefore of N.A. from Top Fibre of Deck Slab $= \frac{4.3E+07}{8.3E+04}$

$= \boxed{518.95}$ mm

Moment of Inertia about N.A. of the composite Section

	=	4.8E+10	-	8.3E+04	x (	518.9546)^2	
	=		3E+10	mm^4			
Ixx	=	3E+10	=	2.56E+10	mm^4		

y_t	=	519 mm,		$y_{b=}$	1056 mm		
Zt	=	$\frac{Ixx}{y_t}$	=	$\frac{3E+10}{519}$	=	4.93E+07	mm^3
Zb	=	$\frac{Ixx}{y_b}$	=	$\frac{3E+10}{1056}$	=	2.42E+07	mm^3

Elastic section modulus (Ze) = 4.93E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$	=	6.25E+05	mm^2
a	=	$\frac{(f_y/\gamma_m)}{0.36(f_{ck})}$ =	27.1
A_s	=	41710	mm^2
A_f	=	12000	mm^2
$a*A_s$	=	1.13E+06	mm^2
$a*A_f$	=	3.26E+05	mm^2
$b_{eff}d_s + 2aA_f$	=	1.28E+06	mm^2
d_s	=	250	mm
t_f	=	30	mm
b_f	=	400	mm
t_w	=	14	mm
d_c	=	787.50	mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis within steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	452.91	1.18E+10
within steel flange	273.35843	1.20E+10
within web	90.24	1.18E+10

Neutral axis within Top flange of a composite section 273.36 mm depth from top of deck.

Moment of resistance M_p = 12021.63 KNm > 4568.10 KNm

Hence OK

Effective Width of Concrete Slab for Inner Girder For Transient Loading

$\frac{2500.00}{7.50}$ = 333.33 mm

Max . Bending Moment due to DL+SIDL+LL = 7768.34 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	Ay_e(mm^3)	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**outer Girder**	41710	912.50	3.8E+07	3.5E+10	1.2E+10	4.72E+10
2	**Deck Slab**	83333.3	125.00	1.0E+07	1.3E+09	4.3E+08	1.74E+09
	Σ	125043		4.8E+07			4.89E+10

Therefore of N.A. from Top Fibre of Deck Slab = $\frac{4.8E+07}{1.3E+05}$

= 387.68 mm

Moment of Inertia about N.A. of the composite Section

= 4.9E+10 - 1.3E+05 x (387.6819)^2

= 3E+10 mm^4

Ixx = 3E+10 = 3.01E+10 mm^4

y_t = 388 mm, $y_{b=}$ 937 mm

Zt = $\frac{Ixx}{y_t}$ = $\frac{3E+10}{388}$ = 7.76E+07 mm^3

Zb = $\frac{Ixx}{y_b}$ = $\frac{3E+10}{937}$ = 3.21E+07 mm^3

Elastic section modulus (Ze) = 7.76E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 625000 mm^2

a = $\frac{(f_y/\gamma_m)}{0.36(f_{ck})}$ = 27.1

A_s = 41710 mm^2

A_f = 12000 mm^2

$a*A_s$ = 1132279 mm^2

$a*A_f$ = 325758 mm^2

$b_{eff}d_s + 2aA_f$ = 1276515 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis in steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	452.91162	1.18E+10
within steel flange	273.35843	1.20E+10

within web	90.240864	1.18E+10

Neutral axis within Top flange of a composite section 273.36 mm depth from top.

Moment of resistance M_p = $\boxed{12021.63}$ KNm

> $\boxed{7768.34}$ KNm

Hence OK

SUMMARY OF BENDING MOMENT CAPACITY

Load Cases	Inner Girder		
	Ultimate Bending Moment	Plastic Moment Capacity	STATUS
Stage I : DL	4161.15	7331.75	**OK**
Stage II : DL+SIDL	4568.10	12021.63	**OK**
Stage III: DL+SIDL+LL	7768.34	12021.63	**OK**

FOR THE COMPOSITE SECTION AT MID SPAN : - INNER GIRDER

(i.e. after casting of Deck slab) : -

Moment of Interia of the composite section

Max permissible bending stress in tension or compression.

$$\sigma_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = \boxed{390.91} \text{ Mpa}$$

Effective Width of Concrete Slab

For inner girder $b_{eff} = \frac{L_0}{4} \leq \frac{(B_1+B_2)}{2}$

L_0 = 28.70 m

$B_1 = B_2 = B$ = 2.50 m

b_{eff} = 7.18 $\geq$ 2.50

Hence,

$b_{eff} = \boxed{2.5}$ m

Modular Ratio

Modular Ratio for Composite Section with Prefabricated units in Steel for Transient Loads

$$m = \frac{E_s}{E_c} = \frac{\text{Modulas of Elasticity of steel of girder}}{\text{Modulas of Elasticity of Cast-in-situ concrete at 28-days}}$$

But $\frac{E_s}{E_c} \geq \boxed{7.5}$ For Transient Loading

$\frac{E_s}{K_c E_c} \geq \boxed{15}$ For Permanent loading

K_c Creep factor = $\boxed{0.5}$

$$\frac{E_s}{E_c} = \frac{200000}{31623} = \boxed{6.32} \leq 7.5$$

m = 7.5 For Transient Loading

$$\frac{E_s}{K_c E_c} = \frac{200000}{15811.388} = \boxed{12.65} \leq 15$$

m = 15 For Permanent loading

For the calculation of equivalent area of deck, divide the effective width of concrete slab by modular ratio

Effective Width of Concrete Slab for Inner Girder For Permanent Loading

$$\frac{2500.00}{15.00} = \boxed{166.67} \text{ mm}$$

Max . Bending Moment due to DL+SIDL = 3599.45 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	$A_i(mm^2)$	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**Inner Girder**	41710	912.50	3.8E+07	3.5E+10	1.2E+10	4.72E+10
2	**Deck Slab**	41666.7	125.00	5.2E+06	6.5E+08	2.2E+08	8.68E+08
	S	83376.7		4.3E+07			4.80E+10

Therefore N.A. from Top Fibre of Deck Slab $= \frac{4.3E+07}{8.3E+04}$

= 518.95 mm

Moment of Inertia about N.A. of the composite Section

= 4.8E+10 - 8.3E+04 x (518.9546)^2

= 3E+10 mm^4

Ixx = 2.56E+10 = 2.56E+10 mm^4

y_t = 519 mm, $y_{b=}$ 1056 mm

$$Z_t = \frac{Ixx}{y_t} = \frac{3E+10}{519} = 4.93E+07 \text{ mm}^3$$

$$Z_b = \frac{Ixx}{y_b} = \frac{3E+10}{1056} = 2.42E+07 \text{ mm}^3$$

Elastic section modulus (Ze) = 4.93E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$	=	6.25E+05	mm^2	
a	=	$\frac{(f_y/\gamma_m)}{0.36(f_{ck})}$	=	27.1
A_s	=	41710	mm^2	
A_f	=	12000	mm^2	
$a*A_s$	=	1.13E+06	mm^2	
$a*A_f$	=	3.26E+05	mm^2	
$b_{eff}d_s + 2aA_f$	=	1.28E+06	mm^2	
d_s	=	250	mm	
t_f	=	30	mm	
b_f	=	400	mm	
t_w	=	14	mm	
d_c	=	787.50	mm	

$b_{eff}d_s$	>	$a*A_s$			Neutral axis within slab
$b_{eff}d_s$	<	$a*A_s$	<	$b_{eff}d_s + 2aA_f$	Neutral axis within steel flange
$b_{eff}d_s + 2aA_f$	<	$a*A_s$			Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	452.91162	1.18E+10
within steel flange	273.35843	1.20E+10
within web	90.240864	1.18E+10

Neutral axis within Top flange of a composite section 273.36 mm depth from top of deck.

Moment of resistance M_p = 12021.63 KNm > 3599.45 KNm

Hence OK

Effective Width of Concrete Slab for Inner Girder For Transient Loading

$$\frac{2500.00}{7.50} = 333.33 \text{ mm}$$

Max . Bending Moment due to DL+SIDL+LL = 6981.60 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**Inner Girder**	41710	912.50	3.8E+07	3.5E+10	1.2E+10	4.72E+10
2	**Deck Slab**	83333.3	125.00	1.0E+07	1.3E+09	4.3E+08	1.74E+09
	Σ	125043		4.8E+07			4.89E+10

Therefore of N.A. from Top Fibre of Deck Slab = 4.8E+07 / 1.3E+05

= 387.68 mm

Moment of Inertia about N.A. of the composite Section

= 4.9E+10 - 1.3E+05 x (387.6819)^2

= 3E+10 mm^4

Ixx = 3E+10 = 3.01E+10 mm^4

y_t = 388 mm, $y_{b=}$ 1187 mm

Zt = Ixx / y_t = 3E+10 / 388 = 7.76E+07 mm^3

Zb = Ixx / y_b = 3E+10 / 1187 = 2.53E+07 mm^3

Elastic section modulus (Ze) = 7.76E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 625000 mm^2

a = $\frac{(f_y/\gamma_m)}{0.36(f_{ck})}$ = 27.1

A_s = 41710 mm^2

A_f = 12000 mm^2

$a*A_s$ = 1132279 mm^2

$a*A_f$ = 325758 mm^2

$b_{eff}d_s + 2aA_f$ = 1276515 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis in steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	452.91162	1.18E+10
within steel flange	273.35843	1.20E+10
within web	90.240864	1.18E+10

Neutral axis within Top flange of a composite section 273.36 mm depth from top.

Moment of resistance M_p = 12021.63 KNm

\> 6981.60 KNm

Hence OK

SUMMARY OF BENDING MOMENT CAPACITY

	Inner Girder		
Load Cases	Ultimate Bending Moment	Plastic Moment Capacity	STATUS
Stage I : DL	4161.15	7331.75	**OK**
Stage II : DL+SIDL	3599.45	12021.63	**OK**
Stage III: DL+SIDL+LL	6981.60	12021.63	**OK**

DESIGN OF PLATE GIRDER FROM ULTIMATE LIMIT STATE METHOD (AT SPLICE LOCATION)

BENDING MOMENT SUMMARY AT SPLICE LOCATION

Stage I
Self weight of Girder alone

		ULS		SLS		
		COMBINATIONS				
		Basic	Accidental	Rare	Quasi	
Partial Safety Factor		1.35	1	1	1	
Max Bending moment (Outer Girder)	=	575.1	426	426		KN-m
Max Shear Force (Outer Girder)	=	33.885	25.1	25.1		KN
Max Bending moment (Inner Girder)	=	575.1	426	426		KN-m
Max Shear Force (Inner Girder)	=	33.885	25.1	25.1		KN

Stage II
Green concrete Stage

		Basic	Accidental	Rare	Quasi	
Partial Safety Factor		1.35	1	1	1	
Max Bending moment (Outer Girder)	=	2147.85	1591	1591		KN-m
Max Shear Force (Outer Girder)	=	126.63	93.8	93.8		KN
Max Bending moment (Inner Girder)	=	1952.1	1446	1446		KN-m
Max Shear Force (Inner Girder)	=	115.155	85.3	85.3		KN

Erection load

		Basic	Accidental	Rare	Quasi	
Partial Safety Factor		1	1	-	-	
Max Bending moment (Outer Girder)	=	881	881	-	-	KN-m
Max Shear Force (Outer Girder)	=	52	52	-	-	KN
Max Bending moment (Inner Girder)	=	801	801	-	-	KN-m
Max Shear Force (Inner Girder)	=	47.3	47.3	-	-	KN

Stage III
Composite action stage
SIDL
wearing coat

		Basic	Accidental	Rare	Quasi	
Partial Safety Factor		1.75	1	1	1	
Max Bending moment (Outer Girder)	=	927.5	530	530		KN-m
Max Shear Force (Outer Girder)	=	51.233	29.276	29.276		KN
Max Bending moment (Inner Girder)	=	843.5	482	482	0	KN-m
Max Shear Force (Inner Girder)	=	46.55	26.6	26.6	0	KN

Crash barrier + RCC Railing

		Basic	Accidental	Rare	Quasi	
Partial Safety Factor		1.35	1	1	1	
Max Bending moment (Outer Girder)	=	1300.05	963	963		KN-m
Max Shear Force (Outer Girder)	=	59.4	44	44		KN
Max Bending moment (Inner Girder)	=	0	0	0		KN-m
Max Shear Force (Inner Girder)	=	0	0	0		KN
Max Reaction (Inner Girder)	=	0	0	0		KN

Live Load Combinations

Impact+Congestion factor +reduction 1.256

1-70R + 2-Class A OR 4 CLASS A

Partial Safety Factor		1.5	0.75	1	0.75	
Max Bending moment (Outer Girder)	=	3200.2	1600.1	2133.5		KN-m
Max Shear Force (Outer Girder)	=	555.7	277.8	370.4		KN
Max Bending moment (Inner Girder)	=	2354.5	1177.3	1569.7		KN-m
Max Shear Force (Inner Girder)	=	587.7	293.8	391.8		KN

SV Load

Impact+Congestion factor +reduction 1

Partial Safety Factor		1.15	1.15	1	0.75	
Max Bending moment (Outer Girder)	=	0.0	0.0	0.0		KN-m
Max Shear Force (Outer Girder)	=	0.0	0.0	0.0		KN
Max Bending moment (Inner Girder)	=	3382.15	3382.15	2941		KN-m
Max Shear Force (Inner Girder)	=	748.65	748.65	651		KN

Max Live loads

Max Bending moment (Outer Girder)	=	3200.2	1600.1	2133.5	KN-m
Max Shear Force (Outer Girder)	=	555.7	277.8	370.4	KN
Max Bending moment (Inner Girder)	=	3382.2	3382.2	2941.0	KN-m
Max Shear Force (Inner Girder)	=	748.7	748.7	651.0	KN

For Composite action

DL+SIDL

Max Bending moment (Outer Girder)	=	4950.5	3510.0	3510.0	KN-m
Max Shear Force (Outer Girder)	=	271.1	192.2	192.2	KN
Max Bending moment (Inner Girder)	=	3599.5	2551.0	2628.8	KN-m
Max Shear Force (Inner Girder)	=	532.9	373.5	387.9	KN

SIDL+ LL

Max Bending moment (Outer Girder)	=	5427.8	3093.1	3626.5	KN-m
Max Shear Force (Outer Girder)	=	666.3	351.1	443.7	KN
Max Bending moment (Inner Girder)	=	4062.9	3771.2	3407.8	KN-m
Max Shear Force (Inner Girder)	=	874.3	820.5	737.2	KN

Total Bending Moment For Critical Case

DL+SIDL+LL	For Outer Girder	=	8150.7	5110.1	5643.5	KN-m
DL+SIDL+LL	For Inner Girder	=	6981.6	5933.2	5569.8	KN-m

Total Shear Force For Critical Case

DL+SIDL+LL -Outer Girder	=	826.8	470.0	562.6	KN
DL+SIDL+LL -Inner Girder	=	1281.6	1122.2	1038.9	KN

DESIGN OF SPLICE

Panel arrangement for splicing is -

Left panel	=	9.5	m
Left panel 1	=	10	m

Depth of Plate Girder	=	1.325	
Width of Top Flange	=	0.4	m
Thickness Of Top Flange1	=	0.030	m
Thickness Of Top Flange2	=	0.000	m
Thickness of Bottom Flange1	=	0.030	m
Thickness of Bottom Flange1	=	0.000	m
Thickness of Web	=	0.014	m
Width of Bottom Flange	=	0.4	m
Depth of Web	=	1.265	m

WEB SPLICE

Component	Shear force(kN)	Bending Moment(kN.m)
Dead Load	161	2723
SIDL	111	2228
Live Load	749	3382
Total	1020	8333

Size of Web Splice

Number of Splice plates = 2

Height of web splice plate hs = 1177 x mm

Min. Thickness of Splice plate = 1/2 x Web Thk.

= 0.007 m

Hence Provide a plate of 14 mm thk. On both side

= 1177 x 14

1.18 m x 0.014 m

Moment of Inertia of Web Plate = $\frac{tw \times d^3}{12}$

= 0.0024 m^4

Gross Moment of Inertia of Section

For Dead Load	0.012	m^4
For SIDL	0.026	m^4
For Live Load	0.030	m^4

Component	Proportion of moment carried by Web Plate	Moment Carried by Web plate(kN-m)
Dead Load	0.190	517.54
SIDL	0.092	205.74
Live Load	0.078	265.38
	Total	988.66

Component	Y at top of Splice Plate (Yt) (m)	Y at bottom of splice Plate(Yb) (m)
Dead Load	0.589	0.589
SIDL	0.195	0.982
Live Load	0.020	1.113

Component	M.I. of splice plate About N.A.of whole Section	sectional Modulus		Stress at top of Splice Plate	Stress at bottom of Splice Plate
	Ixx (m^4)	Zt(m^3)	Zb(m^3)	N/mm^2	N/mm^2
Dead Load	0.00190	0.003	0.003	160.1	160.1
SIDL	0.00445	0.023	0.005	9.00	45.36
Live Load	0.00723	0.368	0.006	0.72	40.84
		Total		169.8	246.3

Horizontal distance from centroid of bolt pattern to extreme bolt (X_a) = (n-1)x s/2
= 225 mm

Eccentricity of center of bolt pattern from the centerline of splice = Xa + (bolt edge distance from splice sentre)
= 315 mm
= 0.315 m

Additional moment carried by bolts due to shear = 1019.80 x 0.315
= 321.24 kN.m

Hence Moment carried by Web Plate = 1309.90 kN.m

$$ts \text{ required} = \frac{6 \times 1309.90 \times 10^{\wedge}6}{2 \times 344.0 \times 1177^{\wedge}2}$$

= 8.25 mm
< 14 mm **O.K.**

Using High Strength Friction Grip(H.S.F.G.) Bolted joint

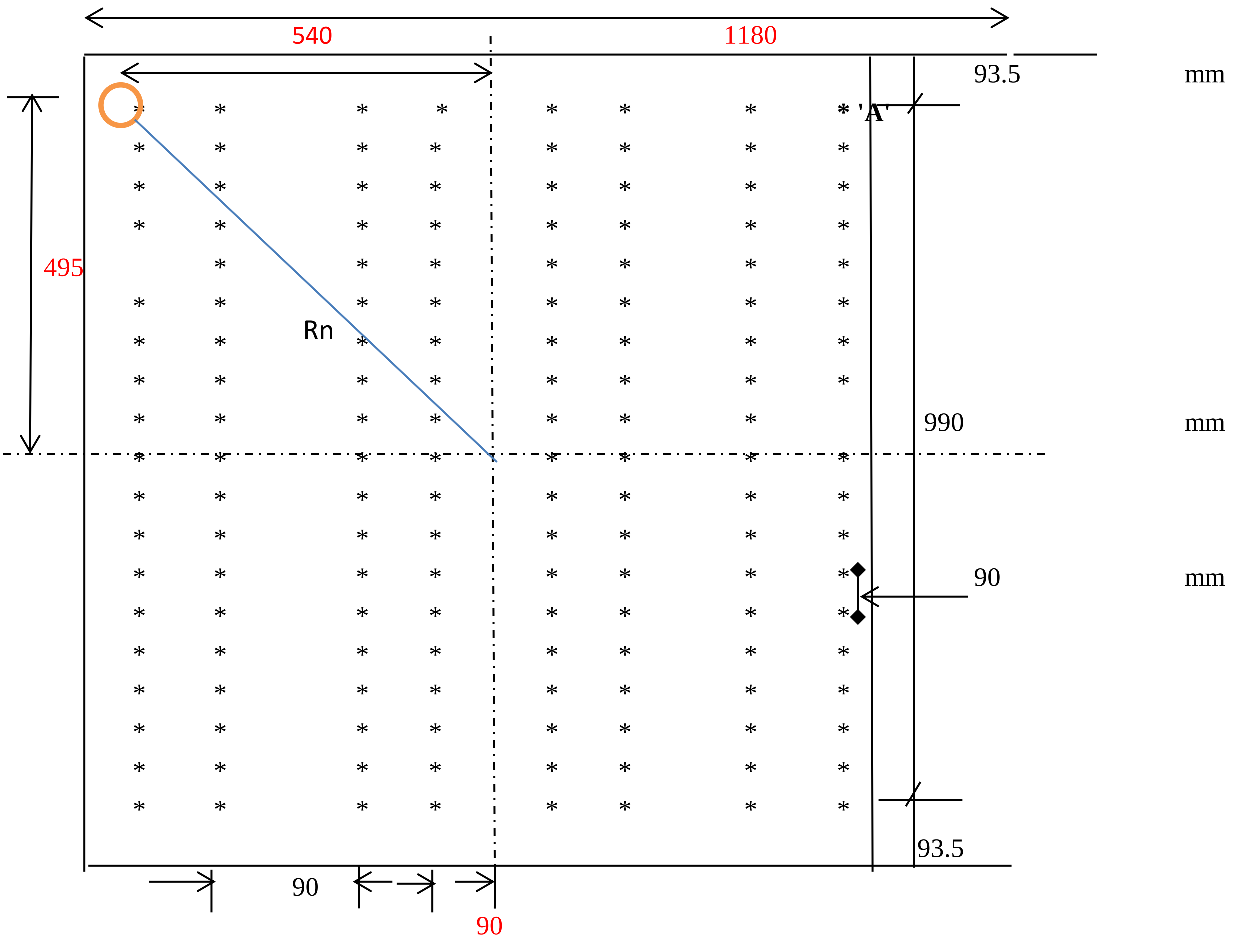
Nominal size of the bolt to be used = 24 mm 8.8 CLASS
Diameter of Bolt hole = 25.5 mm

Provide 6 Vertical rows on each side
Let us provide n = 12 bolts in one vertical line
Providing 90 mm Pitch vertically
Providing 90 mm Pitch Horizontly

Horizontal edge distance = 50 mm
Horizontal edge distance from splice centre = 90 mm
Vertical edge distance 93.5 mm

Minimum Pitch of Bolts = 2.5 x Nominal Dia of bolt
= 60 mm

Max. Pitch of Bolts	=	Min of 12t or 200 mm in compression(t : Thickness of thinner Plate)
	=	168 mm
Minimum edge distance	=	1.5 x Dia of hole
	=	38 mm



Note: Above figure is indicative only for nos. of rows in vertical and horizntal direction

Shear Per Bolt = $\dfrac{\text{SLIP FACTOR X NUMBER OF EFFECTIVE INTERFACES X MIN. BOLT TENSION}}{\text{FACTOR OF SAFETY}}$

= $\dfrac{0.5 \times 2 \times 212}{1.4}$

= 151.429 KN

Bearing Per Bolt= 24 x 14 x $\dfrac{430}{1.1}$

= 131.35 KN

Bolt Value = 131.35 KN

Nos of Bolt from Moment Consideration

= SQRT (6 M/ Pitch x No. Of Vertical Row x Bolt Value)

= $\dfrac{6 \times 1.31E+09}{90 \times 6 \times 131345}$

= 10.527 Nos < 12 **SAFE**

Shear Carryied by Each Bolt= FD1		=	1019.798 /	72
		=	14.16 KN	

FT1 = $\dfrac{Mw \times Rn}{\sum R^2}$

Rn = 732.547 mm

ΣR^2 = ΣX^2 + ΣY^2

ΣX^2	=	12 x (	291600 +	202500
	+	+	129600 +	72900
		+	32400 +	8100
		+	0 +	8100
			0 +	0)
	=	8942400 mm^2		
ΣY^2	= (	245025 +	164025 +	99225
	+	50625 +	18225 +	2025
	+	0 +	0 +	0
	+	0 +	0 +	0
			)x	12
	=	6949800 mm^2		
ΣR^2	=	15892200 mm^2		

FT1 = $\dfrac{1309898.051 \quad x \quad 732.547}{15892200}$ = 60.379

COS θ = B/H = 1.091

FR1 = SQRT (FD1 ^2 + FT1^2 +
2 * FD1 * FT1 * COSθ)
= 75.58 KN < 131.35 **SAFE**

SIZE OF WBE SPLICE PLATE=	**1180 x**	**14 x**	**1177**

Bottom Flange Splice

Moment at splice plate = $M_{dl} + M_{sidl} + M_{ll}$
=

Dimension of bottom flange	=	400	x	30.0
Area of bottom flange	=	12000 mm^2		
Required area of splice plate	=	12600 mm^2		

Flange splice used	nos.		width	thickness	length
Outside plate (No. = 1)	1	x	400 x	20 x	1640
Inside plate (No. = 2)	2	x	164 x	20 x	1640

Area = 14560 mm^2
> 12600 **O.K.**

Component	Mid Depth of tension flange from N.A.(Yb)mm	Moment(N.mm)	Moment of Inertia (I) (mm^4)	Stress (MY/I) Mpa
Dead Load	647.500	2.723E+09	1.24E+10	141.89
Sidl	1041.045	2.228E+09	2.56E+10	90.69
Live Load	1172.318	3.382E+09	3.01E+10	131.73
			Total	364.32 Mpa

Design force on splice element = 12000 x 364.32
= 4371.86 kN
1.10 x Computed force in the flange = 4809.04 kN
0.80 x Capacity of weaker flange = 2460.29 kN

No. of effective interface = 2

Frictional force per bolt(Bolt value) R = $\frac{151.429 \times 2}{2}$
= 151.429 kN
No . Of bolts required for splicing = 32

Provided Nos. Of Bolt Rows = 4 Nos
Nos. Of Bolt Required Per Row = 8.000 Nos
Say 9

Minimum Pitch of Bolts = 2.5 x Nominal Dia of bolt
= 60 mm
Max. Pitch of Bolts = Min of 12t or 200 mm in compression(t : Thickness of thinner Plate)
= 200 mm
Minimum edge distance = 1.5 x Dia of hole
= 38 mm

Pitch of Bolt = 90
Edge Distance = 50
Length of Plate = 1640 mm

Top Flange Splice

Moment at splice plate = $M_{dl} + M_{sidl} + M_{ll}$
=
Dimension of Top flange = 400 x 30
Area of top flange = 12000 mm^2
Required area of splice plate = 12600 mm^2
(5 % extra, Cl. 508.12 of IRC:24-2001)

Flange splice used	nos.	width	thickness	length
Outside plate (No. = 1)	1 x	400 x	20 x	1320
Inside plate (No. = 2)	2 x	167 x	20 x	1320

Area = 14680 x mm^2
> 12600 **O.K.**

Component	Mid Depth of compression flange from N.A.(Yt)mm	Moment(N.mm)	Moment of Inertia (I) (mm^4)	Stress (MY/I)Mpa
Dead Load	647.500	2.723E+09	1.24E+10	141.89
Sidl	503.955	2.228E+09	2.56E+10	43.90
Live Load	372.682	3.382E+09	3.01E+10	41.88

Total 227.68 Mpa

Design force on splice element	=	12000 x	227.68
	=	2732.10 kN	
1.10 x Computed force in the flange	=	3005.31 kN	
0.80 x Max safe force in weaker flange	=	3302.40 kN	
No. of effective interface	=	2	
Frictional force per bolt(Bolt value) R	=	$\frac{151.43 \times 2}{2}$	
	=	151.428571 kN	
No . Of bolts required for splicing	=	22	
Provided Nos. Of Bolt Rows	=	4 Nos	
Nos. Of Bolt Required Per Row	=	5.500 Nos	
	Say	7	
Minimum Pitch of Bolt	=	60 mm	
Minimum Edge Distance	=	38 mm	
Pitch of Bolt	=	90	
Edge Distance	=	50	
Length of Plate	=	1320 mm	

ULS DESIGN OF PLATE GIRDER AT SPLICE LOCATION

Top Flange thickness Plate-1	=	30	mm
Top Flange thickness Plate-2	=	0	mm
Bottom Flange thickness Plate-1	=	30	mm
Bottom Flange thickness Plate-2	=	0	mm
Depth of the Web Plate	=	1265	mm
Thickness of the web	=	14	mm
Overall depth of section	=	1325	mm

Web Plate Area -

web plate of size = **1265** x **14**

Nos. Of Rows of Bolt at Splice location of Web	=	12	
Dia of Bolt Hole	=	25.5	mm
Total Area of Bolts	=	6128.46	mm^2
Moment of Inertia of Bolts	=	249065	mm^4

Top Flange Area -

							effective width
I)Top Plate-1	**400**	x	**30**	=	12000	mm^2	298
II) Top Plate-2	**0**	x	**0**	=	0	mm^2	0

A_f = 12000 mm^2

Nos. Of Rows of Bolt at Splice location of Top Flange	=	4	Nos.
Dia of Bolt Hole	=	25.5	mm
Total Area of Bolts	=	2042.82	mm^2
Reduced area of Top Flange1	=	**9957.18**	mm^2
Reduced area of Top Flange2	=	**0**	mm^2
Moment of Inertia of Bolts	=	83021.5	mm^4

Bottom Flange Area -

							effective width
I) Bottom Plate-1	**400**	x	**30**	=	12000	mm^2	298
II)Bottom Plate-2	**375**	x	**0**	=	0	mm^2	273

Nos. Of Rows of Bolt at Splice location of Bottom Flange	=	4	Nos.
Dia of Bolt Hole	=	25.5	mm
Total Area of Bolts	=	2042.82	mm^2
Moment of Inertia of Bolts	=	20755.4	mm^4
Reduced area of bottom Flange1	=	**9957.18**	mm^2
Reduced area of bottom Flange2	=	**0**	mm^2

CALCULATION OF REDUCED SECTIONAL PROPERTIES AT SPLICE LOCATION

Elastic Properties

SR.	ELEMENT		Ai(mm^2)	Ye (mm)	AYi	AY^2s	Ixx (self)	It =
NO.				(from top)				AY^2s + Ixx

1	Top Flange Plate-1	9957.2	15	1.5E+05	2.2E+06	8.2E+05	3.1E+06
2	Top Flange Plate-2	0.0	0	0.0E+00	0.0E+00	0.0E+00	0.0E+00
3	Web	11581.5	663	7.7E+06	5.1E+09	2.4E+09	7.4E+09
4	Bottom Flange Plate-1	9957.2	1310	1.3E+07	1.7E+10	8.8E+05	1.7E+10
5	Bottom Flange Plate-2	0.0	1325	0.0E+00	0.0E+00	0.0E+00	0.0E+00
deducting ineffective web		0.0	243	0.0E+00	0.0E+00	0.0E+00	0.0E+00
	Σ	31495.9		2.1E+07			2.5E+10

Therefore distance of N.A. from top fibre

$$= \frac{\Sigma Ay_c}{\Sigma A} = \frac{2.1E+07}{31495.90} = \boxed{662.50} \text{ mm}$$

M.I. About N. A. = 2.5E+10 - 31495.90 x (6.63E+02)^2

I_{xx} = $\boxed{1.07E+10}$ mm^4

y_b = D - y_t

= 1325 - 662.50 = $\boxed{662.50}$ mm

$$Z_b = \frac{I_{xx}}{y_b} = \frac{1.1E+10}{662.50} = \boxed{1.6E+07} \text{ mm}^3$$

$$Z_t = \frac{I_{xx}}{y_t} = \frac{1.1E+10}{662.50} = \boxed{1.6E+07} \text{ mm}^3$$

Elastic section modulus (Ze) = $\boxed{1.62E+07}$ mm^3

Plastic Properties

Assume distance of neutral axis from Top of top flange is d.

d = 662.50 mm

Area above neutral axis (A1)

A1 =

= 17795 mm^2

Area below neutral axis (A2)

A2 =

= 17795 mm^2

Moment balance about neutral axis

A2 - A1 = $\boxed{0}$

Area in Compression

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Top Flng 1	9957	648	6.4E+06
Top Flng 2	0	663	0.0E+00
web	8855	316.25	2.8E+06
ineffective \	0	419.00	0.0E+00
	18812		9247667

Area in Tension

S. No.	Area Ai (mm^2)	distance from N.A Yi (mm)	Ai*Yi (mm^3)
Bottom Flng 1	9957.18	648	6.4E+06
Bottom Flng 2	0	663	0.0E+00
web	8855	316	2.8E+06
	18812.2		9.2E+06

$$Y1 = \frac{\Sigma A_i Y_i}{\Sigma A_i} = \boxed{491.58} \text{ mm}$$

$$Y2 = \frac{\Sigma A_i Y_i}{\Sigma A_i} = \boxed{491.58} \text{ mm}$$

Plastic section modulus (Zp) = A1*Y1 + A2*Y2 = $\boxed{1.8E+07}$ mm^3

DESIGN STAGE I: UNDER SELF WEIGHT AT SPLICE LOCATION

Max Bending Moment on Inner Girder due to self wt.(M) : 575.10 KNm

(Note: Section is assumed as laterally unsupported beams for Self wt. Stage)

Design bending strength of beam (M_d):

$M_d = \beta_b Z_p f_{bd}$

Elastic critical moment (M_{cr}):

$M_{cr} = \beta_b Z_p f_{cr,b}$

Ze	=	1.62E+07
Zp	=	18495335
β_b	=	8.74E-01

Effective length for lateral torsional buckling(L_{LT}) = 0.75*L

21.53 m

Radius of gyration of longitundinal girder $= \sqrt{\dfrac{I}{A}}$

= 583.20 mm

= 0.58 m

Slenderness ratio $= \dfrac{kL}{r}$ = 36.91

h_f = 1.295

t_f = 0.03

h_f/t_f = 43.17

$f_{cr,b}$ = 2097.1 Mpa

M_{cr} = 3.39E+04 KNm

λ_{LT} = 0.453

$\phi_{LT} = 0.5(1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}^{2})$

α_{LT} = 0.49 For welded section

ϕ_{LT} = 0.664

$$\chi_{LT} = \frac{1}{\{\phi_{LT}+[\phi_{LT}^{2}-\lambda_{LT}^{2}]^{0.5}\}}$$

= 0.869

f_{bd} = $\chi_{LT}(f_y/\gamma_{m0})$ = 339.70 Mpa

M_d = 5492.8 KNm

> 575.10 KNm

Hence OK

DESIGN STAGE II : GREEN CONCRETE STAGE AT SPLICE LOCATION

(i.e. Green concrete stage) : -

Assuming the girder is laterally supported throughout the span

Max Bending Moment (Self weight, Green concrete weight and temporary load)

= 3603.95 kNm

Elastic section modulus (Ze) = 1.62E+07 mm^3

Plastic section modulus (Zp) = 1.85E+07 mm^3

β_b = = 0.874251 (For Compact section)

Design bending strength

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_m}$$

= 6320.83 KNm

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq \frac{1.2 Z_e f_y}{\gamma_m}$$

β = 0.752 =(2 V/Vd - 1)^2

$$M_{fd} = \frac{Z_p f_y}{\gamma_m} = 5040.60 \text{ KNm}$$

M_{dv} = 5358.05 < 7584.99 KNm

V = 126.63 V_d = 1907.05

126.63 1144.23

V < $0.6 * V_d$

Design bending strength

M_d = 6320.83 KNm

> 3603.95 KNm

Hence OK

DESIGN STAGE III : ULS CHECK OF PLATE GIRDER UNDER COMPOSITE ACTION AT SPLICE LOCATION

Max Bending Moment of girder of composite section = 8150.7 KNm

Max permissible bending stress in tension or compression.

$\sigma_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = 390.909$ Mpa

Hence, b_{eff} = 2.5 m

Effective Width of Concrete Slab for outer Girder For Permanent Loading

= 166.67 mm

Max . Bending Moment due to DL+SIDL = 4950.50 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm²)	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm⁴)	Ixx (self) (mm⁴)	I_t = Ixx (self)+Ay^2e
1	**Outer Girder**	31496	912.50	2.9E+07	2.6E+10	1.1E+10	3.69E+10
2	**Deck Slab**	41666.7	125.00	5.2E+06	6.5E+08	2.2E+08	8.68E+08
	S	73162.6		3.4E+07			3.78E+10

Therefore of N.A. from Top Fibre of Deck Slab $= \frac{3.4E+07}{7.3E+04}$

= 464.01 mm

Moment of Inertia about N.A. of the composite Section

= 3.8E+10 - 7.3E+04 x (464.0124)^2

= 2E+10 mm^4

Ixx = 2.21E+10 = 2.21E+10 mm^4

y_t = 464 mm, $y_{b=}$ 1111 mm

$Zt = \frac{Ixx}{y_t} = \frac{2E+10}{464} = 4.75E+07\ mm^3$

$Zb = \frac{Ixx}{y_b} = \frac{2E+10}{1111} = 1.99E+07\ mm^3$

Elastic section modulus (Ze) = 4.75E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 6.25E+05 mm^2

$a = \dfrac{(f_y/\gamma_m)}{0.36(f_{ck})}$ = 27.1

A_s = 31496 mm^2

A_f = 9957.1794 mm^2

$a*A_s$ = 8.55E+05 mm^2

$a*A_f$ = 2.70E+05 mm^2

$b_{eff}d_s + 2aA_f$ = 1.17E+06 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis within steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	342.001	9.47E+09
within steel flange	260.591	9.44E+09
within web	-128.633	8.57E+09

Neutral axis within Top flange of a composite section 260.59 mm depth from top of deck.

Moment of resistance M_p = 9444.18 KNm > 4950.50 KNm

Hence OK

Effective Width of Concrete Slab for Inner Girder For Transient Loading

333.33 mm

Max . Bending Moment due to DL+SIDL+LL = 8150.74 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	Ay_e(mm^3)	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	Inner Girder	31496	912.50	2.9E+07	2.6E+10	1.1E+10	3.69E+10
2	Deck Slab	83333.3	125.00	1.0E+07	1.3E+09	4.3E+08	1.74E+09
Σ		114829		3.9E+07			3.87E+10

Therefore of N.A. from Top Fibre of Deck Slab = 3.9E+07 / 1.1E+05

= 341.00 mm

Moment of Inertia about N.A. of the composite Section

= 3.9E+10 - 1.1E+05 x (340.9992)^2

= 3E+10 mm^4

Ixx = 3E+10 = 2.53E+10 mm^4

y_t = 341 mm, $y_{b=}$ 1234 mm

Zt = Ixx / y_t = 3E+10 / 341 = 7.43E+07 mm^3

Zb = Ixx / y_b = 3E+10 / 1234 = 2.05E+07 mm^3

Elastic section modulus (Ze) = 7.43E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 625000 mm^2

a = (f_y/γ_m) / $0.36(f_{ck})$ = 27.1

A_s = 31496 mm^2

A_f = 9957 mm^2

$a*A_s$ = 855002 mm^2

$a*A_f$ = 270302 mm^2

$b_{eff}d_s + 2aA_f$ = 1165604 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis in steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	342.0009	9.47E+09
within steel flange	260.5908	9.44E+09
within web	-128.6328	8.57E+09

Neutral axis within Top flange of a composite section 260.59 mm depth from top.

Moment of resistance M_p = 9444.18 KNm

> 8150.74 KNm

Hence OK

SUMMARY OF BENDING MOMENT CAPACITY

	Inner Girder		
Load Cases	Ultimate Bending Moment	Plastic Moment Capacity	STATUS
Stage I : DL	3603.95	6320.83	**OK**
Stage II : DL+SIDL	4950.50	9444.18	**OK**
Stage III: DL+SIDL+LL	8150.74	9444.18	**OK**

DESIGN STAGE III : ULS CHECK OF PLATE GIRDER UNDER COMPOSITE ACTION AT SPLICE LOCATION

Max permissible bending stress in tension or compression.

$$\sigma_{bct} = \frac{fy}{\gamma_m} = 430 \div 1.1 = \boxed{390.909} \text{ Mpa}$$

Effective Width of Concrete Slab $b_{eff} = \boxed{2.5}$ m

For inner girder $b_{eff} = \frac{L_0}{4} \leq \frac{(B_1+B_2)}{2}$

L_0 = 28.70 m

$B_1 = B_2 = B$ = 2.50 m

b_{eff} = 7.18 $\geq$ 2.50

Hence,

For outer girder $b_{eff} = \frac{L_0}{8} + X$

$\frac{L_0}{8} \leq \frac{B}{2}$ and $X \leq \frac{B}{2}$

3.6 $\geq$ 1.25 1.50 $\geq$ 1.25

Hence,

b_{eff} = 1.25 + 1.25

= $\boxed{2.5}$ m

Modular Ratio

Modular Ratio for Composite Section with Prefabricated units in Steel for Transient Loads

$$m = \frac{E_s}{E_c} = \frac{\text{Modulas of Elasticity of steel of girder}}{\text{Modulas of Elasticity of Cast-in-situ concrete at 28-days}}$$

But $\frac{E_s}{E_c} \geq \boxed{7.5}$ For Transient Loading

$\frac{E_s}{K_c E_c} \geq \boxed{15}$ For Permanent loading

K_c Creep factor = 0.5

$\frac{E_s}{E_c}$ = $\frac{200000}{31623}$ = 6.32 ≤ 7.5

m = 7.5 For Transient Loading

$\frac{E_s}{K_c E_c}$ = $\frac{200000}{15811.388}$ = 12.65 ≤ 15

m = 15 For Permanent loading

For the calculation of equivalent area of deck, divide the effective width of concrete slab by modular ratio

Effective Width of Concrete Slab for Inner Girder For Permanent Loading

= 166.67 mm

Max . Bending Moment due to DL+SIDL = 3599.45 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**Inner Girder**	31496	912.50	2.9E+07	2.6E+10	1.1E+10	3.69E+10
2	**Deck Slab**	41666.7	125.00	5.2E+06	6.5E+08	2.2E+08	8.68E+08
S		73162.6		3.4E+07			3.78E+10

Therefore of N.A. from Top Fibre of Deck Slab = $\frac{3.4E+07}{7.3E+04}$

= 464.01 mm

Moment of Inertia about N.A. of the composite Section

= 3.8E+10 - 7.3E+04 x (464.0124)^2

= 2E+10 mm^4

Ixx = 2.21E+10 = 2.21E+10 mm^4

y_t = 464 mm, $y_{b=}$ 1111 mm

Zt = $\frac{Ixx}{y_t}$ = $\frac{2E+10}{464}$ = 4.75E+07 mm^3

Zb = $\frac{Ixx}{y_b}$ = $\frac{2E+10}{1111}$ = 1.99E+07 mm^3

Elastic section modulus (Ze) = 4.75E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 6.25E+05 mm^2

a = $\dfrac{(f_y/\gamma_m)}{0.36(f_{ck})}$ = 27.1

A_s = 31496 mm^2

A_f = 9957.1794 mm^2

$a*A_s$ = 8.55E+05 mm^2

$a*A_f$ = 2.70E+05 mm^2

$b_{eff}d_s + 2aA_f$ = 1.17E+06 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis within steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	342.0009	9.47E+09
within steel flange	260.5908	9.44E+09
within web	-128.6328	8.57E+09

Neutral axis within Top flange of a composite section 260.59 mm depth from top of deck.

Moment of resistance M_p = 9444.18 KNm

> 3599.45 KNm

Hence OK

Effective Width of Concrete Slab for Inner Girder For Transient Loading

= 333.33 mm

Max . Bending Moment due to DL+SIDL+LL = 6981.60 kN.m

Calculation of the Moment of Inertia for Composite Section for Girder

SR. NO.	ELEMENT	Ai(mm^2)	ye(mm)	$Ay_e(mm^3)$	Ay^2_e (mm^4)	Ixx (self) (mm^4)	I_t = Ixx (self)+Ay^2e
1	**Inner Girder**	31496	912.50	2.9E+07	2.6E+10	1.1E+10	3.69E+10
2	**Deck Slab**	83333.3	125.00	1.0E+07	1.3E+09	4.3E+08	1.74E+09
	Σ	114829		3.9E+07			3.87E+10

Therefore of N.A. from Top Fibre of Deck Slab = $\frac{3.9E+07}{1.1E+05}$

= 341.00 mm

Moment of Inertia about N.A. of the composite Section

= 3.9E+10 - 1.1E+05 x (340.9992)^2

= 3E+10 mm^4

Ixx = 3E+10 = 2.53E+10 mm^4

y_t = 341 mm, $y_{b=}$ 1234 mm

Zt = $\frac{Ixx}{y_t}$ = $\frac{3E+10}{341}$ = 7.43E+07 mm^3

Zb = $\frac{Ixx}{y_b}$ = $\frac{3E+10}{1234}$ = 2.05E+07 mm^3

Elastic section modulus (Ze) = 7.43E+07 mm^3

Moment of resistance

Position of plastic neutral axis

$b_{eff}d_s$ = 625000 mm^2

a = $\frac{(f_y/\gamma_m)}{0.36(f_{ck})}$ = 27.1

A_s = 31496 mm^2

A_f = 9957 mm^2

$a*A_s$ = 855002 mm^2

$a*A_f$ = 270302 mm^2

$b_{eff}d_s + 2aA_f$ = 1165604 mm^2

d_s = 250 mm

t_f = 30 mm

b_f = 400 mm

t_w = 14 mm

d_c = 787.50 mm

$b_{eff}d_s$ > $a*A_s$ Neutral axis within slab

$b_{eff}d_s$ < $a*A_s$ < $b_{eff}d_s + 2aA_f$ Neutral axis in steel flange

$b_{eff}d_s + 2aA_f$ < $a*A_s$ Neutral axis within web

Neutral axis	Value of x_u (mm)	Moment Capacity (M_p) (Mpa)
within slab	342.0009	9.47E+09
within steel flange	260.5908	9.44E+09
within web	-128.6328	8.57E+09

Neutral axis within Top flange of a composite section 260.59 mm depth from top.

Moment of resistance M_p = 9444.18 KNm

> 6981.60 KNm

Hence OK

SUMMARY OF BENDING MOMENT CAPACITY

	Inner Girder		
Load Cases	Ultimate Bending Moment	Plastic Moment Capacity	STATUS
Stage I : DL	3603.95	6320.83	**OK**
Stage II : DL+SIDL	3599.45	9444.18	**OK**
Stage III: DL+SIDL+LL	6981.60	9444.18	**OK**

CHECK FOR SERVICEABILITY LIMIT STATE

SERVICEABILITY LIMIT STATE - CHECK FOR STRESS FOR OUTER GIRDER

For checking the stress limit, rare combination is used **(As per Table 3.3, Annex B, IRC:6-2014)**

Limiting Stress for Structural Steel	=	0.87 x fy
	=	0.87 x 430
	=	374.1 MPa
Limiting Stress for Concrete	=	0.33 x fck
	=	0.33 x 40
	=	13.2 MPa

CALCULATION OF STRESS IN DL+SIDL

Effective width = $\frac{2500}{15}$ = 166.67 mm

Max Bending Moment = 3344.2 KN-m

Zt = 4.93E+07 mm^3

Zb = 2.42E+07 mm^3

Compressive Stress at top of girder	=	35.2 MPa
Tensile Stress at bottom of girder	=	138.1 MPa
Compressive Stress at top of deck slab	=	$\frac{M}{z_t \times m}$ (for concrete section)
	=	4.5 MPa

CASE 1: CONSIDERING LIVE LOAD AS LEADING LOAD AND TEMPERATURE AS ACCOMPANYING LOAD

CALCULATION OF STRESS IN LIVE LOAD

Effective width = $\frac{2500}{7.5}$ = 333.33 mm

Max Bending Moment = 2133.5 KN-m

Zt = 7.76E+07 mm^3

Zb = 3.21E+07 mm^3

Compressive Stress at top of girder	=	9.8 MPa
Tensile Stress at bottom of girder	=	66.4 MPa
Compressive Stress at top of deck slab	=	$\frac{M}{z_t \times m}$ (for concrete section)
	=	3.7 MPa

TOTAL STRESS AFTER STAGE III (Mpa)

Limiting Stress for Structural Steel = 374.1 MPa

Limiting Stress for Concrete = 13.2 MPa

	DL+SIDL	LIVE LOAD	TOTAL	
Deck Slab Top (Comp)	4.5	3.7	8.2	Safe
Girder Top (Comp)	35.2	9.8	44.9	Safe
Girder Bottom (Tensile)	138.1	66.4	204.6	Safe

TEMPERATURE AND SHRINKAGE STRESS

Load Factor Thermal load for Accompanying load = 0.6 Rare Combination

	Rising	Falling	Shrinkage	Rising + Shrinkage	Falling + Shrinkage
Deck Slab Top (Comp)	1.5	-0.8	-2.7	-1.2	-3.5
Girder Top (Comp)	-0.3	0.3	-35.6	-35.9	-35.3
Girder Bottom (Tensile)	1.9	-1.5	6.2	8.1	4.7

Stress Summary Considering Live load as Leading Load and Temperature load as accompanying load

	TOTAL IN SHRINKAGE (MPa)	Check	TOTAL IN RISING TEMP (MPa)	Check	TOTAL IN FALLING TEMP (MPa)	Check
Deck Slab Top (Comp)	5.5	Safe	7.0	Safe	4.7	Safe
Girder Top (Comp)	9.3	Safe	9.0	Safe	9.7	Safe
Girder Bottom (Tensile)	210.8	Safe	212.6	Safe	209.3	Safe

CASE 2: CONSIDERING LIVE LOAD AS ACCOMPANYING LOAD AND TEMPERATURE AS LEADING LOAD

CALCULATION OF STRESS IN LIVE LOAD

Effective width = $\frac{2500}{7.5}$ = 333.33 mm

Max Bending Moment = 1600.1 KN-m

Zt = 7.76E+07 mm^3

Zb = 3.21E+07 mm^3

Compressive Stress at top of girder = 7.3 MPa

Tensile Stress at bottom of girder = 49.8 MPa

Compressive Stress at top of deck slab = $\frac{M}{z_t \times m}$ (for concrete section)

= 2.7 MPa

TOTAL STRESS AFTER STAGE III (Mpa)

Limiting Stress for Structural Steel = 374.1 MPa

Limiting Stress for Concrete = 13.2 MPa

	DL+SIDL	LIVE LOAD	TOTAL	
Deck Slab Top (Comp)	4.5	2.7	7.3	Safe
Girder Top (Comp)	35.2	7.3	42.5	Safe
Girder Bottom (Tensile)	138.1	49.8	188.0	Safe

TEMPERATURE AND SHRINKAGE STRESS

Load Factor Thermal load for Accompanying load = 1 Rare Combination

	Rising	Falling	Shrinkage	Rising + Shrinkage	Falling + Shrinkage
Deck Slab Top (Comp)	2.5	-1.3	-2.7	-0.2	-4.0
Girder Top (Comp)	-0.5	0.5	-35.6	-36.1	-35.1
Girder Bottom (Tensile)	3.1	-2.5	6.2	9.3	3.7

Stress Summary Considering Live load as Leading Load and Temperature load as accompanying load

	TOTAL IN SHRINKAGE (MPa)	Check	TOTAL IN RISING TEMP (MPa)	Check	TOTAL IN FALLING TEMP (MPa)	Check
Deck Slab Top (Comp)	4.6	Safe	7.1	Safe	3.3	Safe
Girder Top (Comp)	6.9	Safe	6.4	Safe	7.4	Safe
Girder Bottom (Tensile)	194.1	Safe	197.3	Safe	191.7	Safe

SERVICEABILITY LIMIT STATE - CHECK FOR STRESS OF INNER GIRDER

For checking the stress limit, rare combination is used **(As per Table 3.3, Annex B, IRC:6-2014)**

Limiting Stress for Structural Steel	=	0.87 x fy
	=	0.87 x 430
	=	374.1 MPa
Limiting Stress for Concrete	=	0.33 x fck
	=	0.33 x 40
	=	13.2 MPa

CALCULATION OF STRESS IN DL+SIDL

Effective width = $\frac{2500}{15}$ = 166.67 mm

Max Bending Moment = 2628.8 KN-m

Zt = 4.93E+07 mm^3

Zb = 2.42E+07 mm^3

Compressive Stress at top of girder	=	27.7 MPa
Tensile Stress at bottom of girder	=	108.6 MPa
Compressive Stress at top of deck slab	=	$\frac{M}{z_t \times m}$ (for concrete section)
	=	3.6 MPa

CASE 1: CONSIDERING LIVE LOAD AS LEADING LOAD AND TEMPERATURE AS ACCOMPANYING LOAD

CALCULATION OF STRESS IN LIVE LOAD

Effective width = $\frac{2500}{7.5}$ = 333.33 mm

Max Bending Moment = 2941.0 KN-m

Zt = 7.76E+07 mm^3

Zb = 2.53E+07 mm^3

Compressive Stress at top of girder	=	13.5 MPa
Tensile Stress at bottom of girder	=	116.0 MPa
Compressive Stress at top of deck slab	=	$\frac{M}{z_t \times m}$ (for concrete section)
	=	5.1 MPa

TOTAL STRESS AFTER STAGE III (Mpa)

Limiting Stress for Structural Steel = 374.1 MPa

Limiting Stress for Concrete = 13.2 MPa

	DL+SIDL	LIVE LOAD	TOTAL	
Deck Slab Top (Comp)	3.6	5.1	8.6	Safe
Girder Top (Comp)	27.7	13.5	41.1	Safe
Girder Bottom (Tensile)	108.6	116.0	224.6	Safe

TEMPERATURE AND SHRINKAGE STRESS

Load Factor Thermal load for Accompanying load = 0.6 Rare Combination

	Rising	Falling	Shrinkage	Rising + Shrinkage	Falling + Shrinkage
Deck Slab Top (Comp)	1.5	-0.8	-2.7	-1.2	-3.5
Girder Top (Comp)	-0.3	0.3	-35.6	-35.9	-35.3
Girder Bottom (Tensile)	1.9	-1.5	6.2	8.1	4.7

Stress Summary Considering Live load as Leading Load and Temperature load as accompanying load

	TOTAL IN SHRINKAGE (MPa)	Check	TOTAL IN RISING TEMP (MPa)	Check	TOTAL IN FALLING TEMP (MPa)	Check
Deck Slab Top (Comp)	5.9	Safe	7.4	Safe	5.1	Safe
Girder Top (Comp)	5.5	Safe	5.2	Safe	5.8	Safe
Girder Bottom (Tensile)	230.8	Safe	232.7	Safe	229.3	Safe

CASE 2: CONSIDERING LIVE LOAD AS ACCOMPANYING LOAD AND TEMPERATURE AS LEADING LOAD

CALCULATION OF STRESS IN LIVE LOAD

Effective width = $\frac{2500}{7.5}$ = $\boxed{333.33}$ mm

Max Bending Moment = $\boxed{2205.8}$ KN-m

Zt = 7.76E+07 mm^3

Zb = 2.53E+07 mm^3

Compressive Stress at top of girder = 10.1 MPa

Tensile Stress at bottom of girder = 87.0 MPa

Compressive Stress at top of deck slab = $\frac{M}{z_t \times m}$ (for concrete section)

= 3.8 MPa

TOTAL STRESS AFTER STAGE III (Mpa)

Limiting Stress for Structural Steel = 374.1 MPa

Limiting Stress for Concrete = 13.2 MPa

	DL+SIDL	LIVE LOAD	TOTAL	
Deck Slab Top (Comp)	3.6	3.8	7.3	Safe
Girder Top (Comp)	27.7	10.1	37.7	Safe
Girder Bottom (Tensile)	108.6	87.0	195.6	Safe

TEMPERATURE AND SHRINKAGE STRESS

Load Factor Thermal load for Accompanying load = 1 Rare Combination

	Rising	Falling	Shrinkage	Rising + Shrinkage	Falling + Shrinkage
Deck Slab Top (Comp)	2.5	-1.3	-2.7	-0.2	-4.0
Girder Top (Comp)	-0.5	0.5	-35.6	-36.1	-35.1
Girder Bottom (Tensile)	3.1	-2.5	6.2	9.3	3.7

Stress Summary Considering Live load as Leading Load and Temperature load as accompanying load

	TOTAL IN SHRINKAGE (MPa)	Check	TOTAL IN RISING TEMP (MPa)	Check	TOTAL IN FALLING TEMP (MPa)	Check
Deck Slab Top (Comp)	4.7	Safe	7.2	Safe	3.3	Safe
Girder Top (Comp)	2.1	Safe	1.6	Safe	2.7	Safe
Girder Bottom (Tensile)	201.8	Safe	204.9	Safe	199.3	Safe

CHECK FOR DEFLECTION

SERVICEABILITY LIMIT STATE - CHECK FOR DEFLECTION

(considering the whole superstructure)=	Length	=	28.70	No of Girders	=	5

According to Cl. 504.5.1 of IRC - 24-2010 the permissible deflection for

(DL + LL + SIDL) =	L/600	=	47.8	mm

However , this restriction shall not apply if minimum in place pre camber is provided to compensate for all dead and superimposed dead load deflections.

For LL only =	L/800	=	35.88	mm
Deflection due to DL and SIDL		=	92.09	mm
Deflection due to LL(70R Wheeled)		=	30.00	mm
Provide Pre-camber of		=	123.00	mm

DESIGN OF SHEAR CONNECTORS

DESIGN OF SHEAR CONNECTOR

H.T. SHEAR CONNECTORS

Shear connector between RC deck slab and fabricated steel girder shall be designed which will be of flexible type deriving resistance to horizontal shear through bending. It is proposed to provide welded stud with minimum ultimate strength of 495Mpa, yield point of 385Mpa and elongation of 18%

Safe Shear resistance of one shear connector is

Shear Stud Connectors with minimum Ultimate Tensile Strength (Futs = 495MPa), Nelson Studs

dia of strud-(d)	=	25	mm(assume)
area	=	490.6	mm^2
h	=	125	mm(assume)
top dia	=	37	mm
f_{ck}	=	40	Mpa

Allowable range of Horizontal shear per stud connector

From strength criteria

Qu = 116.8 KN

Provide 4 -nos near support 3 **nos**

Σ Q = 3 x 116800

= 350400 N

Provide 4 -nos near mid span 3 **nos**

Σ Q = 3 x 116800

= 350400 N

Longitudinal Shear / unit length V_L

= $(V\ Ac\ Y\ /\ I)_{dl,ll}$

At Support panel

	Shear force V(kN)	Width(mm)	Depth(mm)	m	Ac(mm^2)	Y(mm)	I(mm^4)	V Ac Y/I (N/mm)
Dead Load	439.695	2500	250	15	41666.7	393.95	2.56E+10	282.27
SIDL	285.225	2500	250	15	41666.7	393.95	2.56E+10	183.11
Live load	748.65	2500	250	7.5	83333.3	262.68	3.01E+10	544.49
								1009.87

Spacing = 346.98 mm

Provide 3 nos 200 mm c/c Safe

At Mid panel

	Shear force V(kN)	Width(mm)	Depth(mm)	m	Ac(mm^2)	Y(mm)	I(mm^4)	V Ac Y/I (N/mm)
Dead Load	175.5	2500	250	15	41666.7	393.95	2.56E+10	112.67
SIDL	79.55	2500	250	15	41666.7	393.95	2.56E+10	51.07
Live load	522.00	2500	250	7.5	83333.3	262.68	3.01E+10	379.65

Spacing = 644.85 mm

Provide 3 nos 200 mm c/c Safe

For Full Shear connection

$$H_1 = \frac{A_{st} * f_y * 10^{-3}}{\gamma_m}$$

$$H_2 = 0.36 * f_{ck} * A_{ec} * 10^{-3}$$

A_{st} = 28202.45 mm^2

A_{ec} = 625000 mm^2

H_1 = 11024.6 kN

H_2 = 9000 kN

Therefore,

H = 9000 kN

Spacing = 558.69 mm

Provide 3 nos 200 mm c/c Safe

From fatigue criteria (only live load is considered)

Qr = 21.6 KN

Provide 3 -nos near support **3 nos**

Σ Q = 3 x 21600

= 64800 N

Provide 3 -nos near mid span **3 nos**

Σ Q = 3 x 21600

= 64800 N

Longitudinal Shear / unit length V_L

= $(V\ Ac\ Y\ /\ I)_{ll}$

At Support panel

	Shear force V(kN)	Width(mm)	Depth(mm)	m	Ac(mm^2)	Y(mm)	I(mm^4)	VI(N/mm)
Live load	587.68	2500.00	250.00	7.5	83333.3	262.68	3.01E+10	427.42

Spacing = 151.61 mm

Provide 3 nos 100 mm c/c Safe

At Mid panel

	Shear force V(kN)	Width(mm)	Depth(mm)	m	Ac(mm^2)	Y(mm)	I(mm^4)	VI(N/mm)
Live load	587.68	2500.00	250.00	7.5	83333.3	262.68	3.01E+10	427.42

Spacing = 151.61 mm

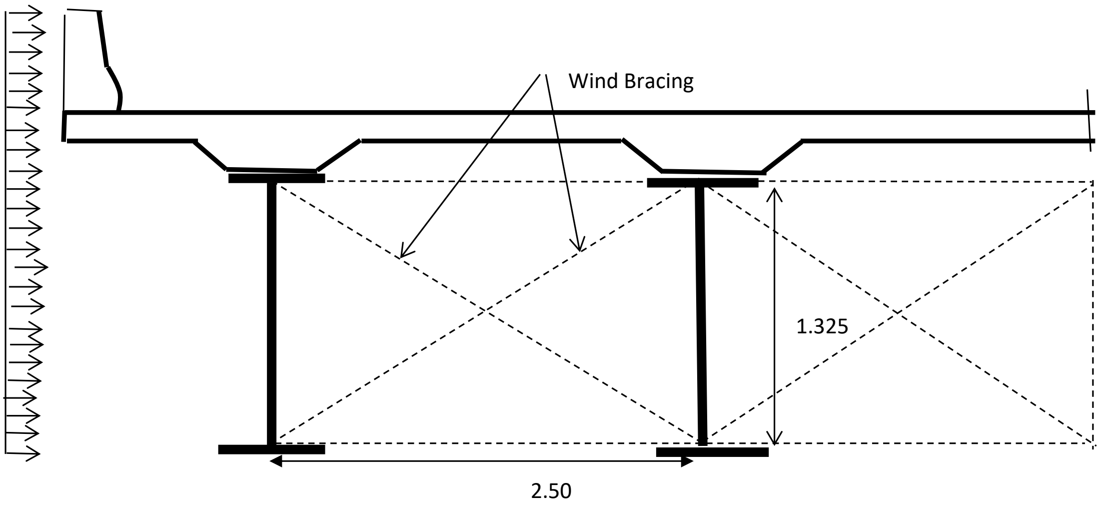
Provide 3 nos 150 mm c/c Safe

DESIGN OF CROSS BRACING

DESIGN OF WIND BRACING

Wind Bracing shall be provided diagonally connecting top flange to the bottom flange at spacing at 4.5 mc/c. Cross-girder are subject to lateral load due to wind force.
Assuming that wind load acting on the upper half of girder, deck and barrier is carried out by the deck slab and wind load on the lower half of girder is carried out by bottom flange

Height of crash barrier	=	1.1	m
Thickness of Deck slab	=	0.250	m
Thickness of Haunch	=	0	m
Overall Depth of steel Plate Girder	=	1.325	m
Total Exposed Depth to wind	=	2.68	m
Spacing of cross Bracing	=	4.50	m
Max Height of the Bridge from Ground Level	=	10.00	m



Wind Bracing shall be provided diagonally connecting top flange to the bottom flange at spacing at = 4.5 mc/c.

Calculation of Wind Load : -

Total length of superstructure = 30.0 m

Wind Force , F = Pz x A1 x G x Cd

i) Wind Force on D.L. : -

Exposed area of superstr. = (1.325 + 1.1 (crash barrier)) x (28.70 + 0.65 x 2)

A1 = 72.75 m^2

Bridge situated in plain terrain

Basic wind speed at bridge site = 50 m/s

wind pressure for 33m/s basic wind speed = 463.7 N/m2

wind pressure for 50m/s basic wind speed (PZ) (47^2/33^2) x 463.7 = 1064.51 N/m2

For Steel Girder Type of Superstructure = 108.51 Kg/m2

G = 2

Cd = 2*(1+c/20.d) = 2.19

F = 34556.15 Kg

ii) Wind force on L.L. : -

F = Pz . A . CD . G

Length of superstructure = 30 m

A = 90 m2

CD = 1.2

G = 2

Wind force = 23438.72 Kg

Total wind force = 34556.15 + 23438.72

Total wind force = 57994.87 Kg

Therefore wind force on each bracing = $\frac{57994.87 \times 4.5}{30}$

= 8699.2 Kg

= 86.99 KN

This force will be shared by Deck slab and Cross frame. Therefore Cross frame will be design for direct force of = (Fw/2)

= 43.50 KN

K = 1

						Area (cm2)	rmin (cm)	Effective Length	L/rvv	Check
Top Chord										
Bottom Chord										
Diagonal C	2	ISA	150	150	10	58.06	4.63	2829.4213	61.1106	OK
						58.06				

Area of Section = 58.06 cm^2

Calculation of Buckling Strength

L = 2829.42 mm

r_{vv} = 46.3 mm

K = 1

$\frac{L}{r_{vv}} = \frac{2829.4213}{46.3} = 61.11$

E = 2.0E+05 MPa

$f_{cc} = \frac{\pi^2 E}{(KL/r_y)^2} = 528.6$

$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = 0.8$

$\chi = \frac{1}{(\Phi+(\Phi^2-\lambda^2)^{0.5})} = 0.668$

$\Phi = 0.5(1+\alpha(\lambda-0.2)+\lambda^2) = 1.0$

α = 0.49 for buckling class 'c' as per Table 4

$f_{cd} = \frac{\chi f_y}{\gamma_{m0}} = 200.5$ MPa

Design Strength of bracing

$P_d = A_e * f_{cd} =$ 1164.120 KN > 43.496 KN

Ok

DESIGN OF BOLTED CONNECTION IN BRACING

The force on the bracing P = 43.50 KN
Lever arm = 1.325 m

Moment in bracing due to wind force = 43.50 x 1.325
= 57.63 KN-m
Gusset plate thickess = 10 mm
Bolted Connections

No of bolts n = sqrt (6 M/ p R) Per vertical line
n = P/ R
Therefore M = $n^2 * p * R / 6$

Where R = Rivet or Bolt value
Nominal size of the bolt to be use = 24 mm
Diameter of Bolt hole = 25.5 mm

Partial safety factor γ_{mb} = 1.25

Strength of bolt in single shear

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{101.9}{1.25}$$
= 81.5 kN

$$V_{nsb} = f_{ub}/3^{0.5}(n_n A_{nb} + n_s A_{sb})$$

= 101.9 kN

Ultimate tensile strength of bolt f_{ub} = **250** MPa
Number of shear planes with threads
intercepting the shear plane n_n = 2
Number of shear planes without threads
intercepting the shear plane n_s = 0
C/s area of the bolt at shank A_{sb} = 452.4 mm^2
Net Shear Area of Bolt A_{nb} = 353.0 mm^2

Stength of Bolt in bearing

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{87.2}{1.25}$$
= 69.8 kN

V_{npb} = $2.5*k_b*d*t*f_u^{1}$
= 87.2 kN

k_b = smallest of $e/3d_0$; $p/3d_0$; f_{ub}/f_u;1
= 0.58

Edge distance e = 45 mm

Pitch	p	=	75	mm
Ultimate tensile strength of bolt	f_{ub}	=	250	MPa
Ultimate tensile strength of plate	f_u	=	430	MPa
	f_u^1	=	$\min(f_{ub}, f_u)$	
		=	250	MPa
Total Thickness of connecting plates		=	10	mm

Bolt Value = 69767.44 **N**

Minimum Pitch p = 2.5* d' 60 mm

Provide Pitch of 75

safe

No of bolts = 43.5/69.77

No of bolts = 0.623

= 0.623 Nos

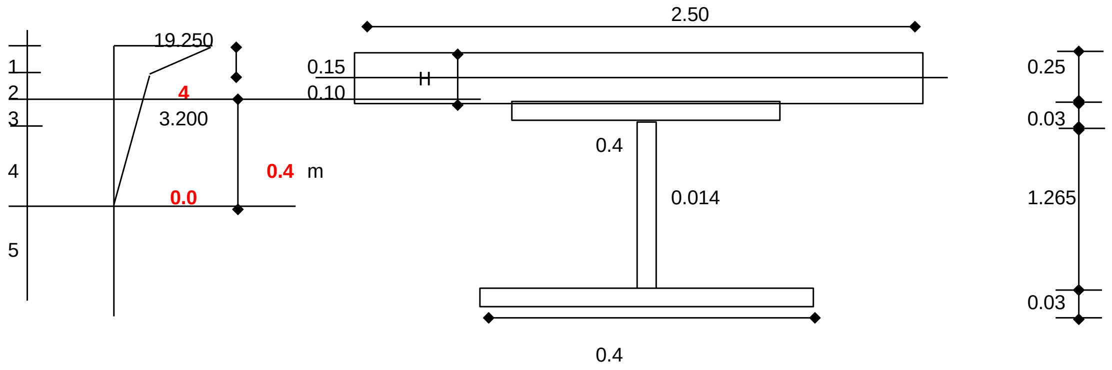
Provide 2 Nos bolts

safe

DESIGN OF PLATE GIRDER CALCULATION FOR TEMPERATURE AND SHRINKAGE STRESSES (MID SPAN)

TEMPERATURE EFFECTS:-

Case (1) : **RISE IN TEMPERATURE**



H(M)	T1(ºC)
0.2	18
0.3	20.5
0.25	19.25
0.6H	4
H+0.4	0.0
0.15	4
0.65	0.0
0.25	3.20
0.28	2.9600
0.650	0

STRENGTH OF CONCRETE	F_{ck}	=	40	
MOD. OF ELASTICITY OF CONCRETE	Ec	=	5000 $\sqrt{F_{ck}}$	(Cl: 604.3 of IRC:22-2008)
			31622.7766	
MOD. OF ELASTICITY OF STEEL	Es	=	2.1X10^5	(Cl: 604.3 of IRC:22-2008)
MOD. RATIO		=	15	(Cl: 604.3 of IRC:22-2008)
COEFF. OF THERMAL EXPENSION	α	=	1.2 x10^$^{-5}$ /ºC	

STRIP	TOP WIDTH m	BOT. WIDTH m	THICKNESS m	AREA m^2	Y m	AY m^3	AY^2 m^4
1	0.167	0.167	0.150	0.025	0.075	0.002	0.000
2	0.167	0.167	0.100	0.017	0.200	0.003	0.001
3	0.400	0.400	0.030	0.012	0.265	0.003	0.001
4	0.014	0.014	0.370	0.005	0.465	0.002	0.001
5	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	Total			0.059		0.0108	0.0028

by solving equation

$\varepsilon o \, \Sigma A \;-\; \theta \, \Sigma Ay \;=\; \alpha \, \Sigma At$ --------------------- (1)

$\varepsilon o \, \Sigma Ay \;-\; \theta \, \Sigma Ay2 \;=\; \alpha \, \Sigma Ayt$ --------------------- (2)

Eigen stress fci

fci = Ec [εo - (y x θ + α x t)]

T Deg:	AT m^2 -Deg	AYT m^3 - Deg
11.625	0.291	0.022
3.600	0.060	0.012
3.080	0.037	0.010
1.480	0.008	0.004
0.000	0.000	0.000
	0.395	0.047

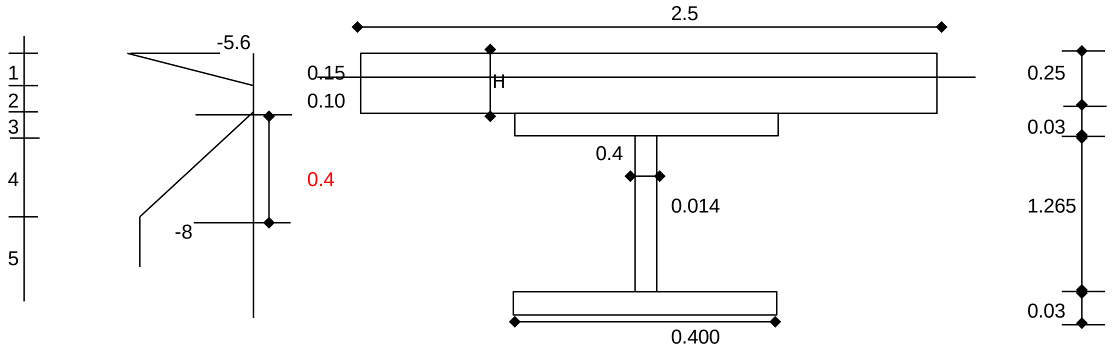
STRAIN AT TOP FIBER, ε = 0.0001514

ROTATION OF PLANE, θ = 0.0003857 rad/m

Y m	T Deg	Yθ Rad:	α t	ε . Therm Strain	σ .therm Mpa
0.000	19.250	0.000	0.000	0.000	-2.518
0.250	3.200	0.000	0.000	0.000	0.523
0.250	3.200	0.000	0.000	0.000	0.523
0.650	0.000	0.000	0.000	0.000	-3.142

EIGEN STRESS AT TOP OF SLAB	=	2.518 N/MM2	Compression
EIGEN STRESS AT TOP OF GIRDER.	=	-0.523 N/MM2	Tension
EIGEN STRESS AT BOTTOM	=	3.142 N/MM2	Compression

Case (2) : **FALL IN TEMPERATURE**



H (M)	T1(°C)
0.2	-4.4
0.3	-6.8
0.25	-5.6
0.6H	0
H+0.4	-8
0.15	0
0.25	0
0.65	-8
0.28	-0.6
0.650	-8

STRENGTH OF CONCRETE	F_{ck}	=	35	
MOD. OF ELASTICITY OF CONCRETE	Ec	=	5000 √ F_{ck}	(Cl: 604.3 of IRC:22-2008)
			29580.39892	
MOD. OF ELASTICITY OF STEEL	Es	=	2.1X10^5	(Cl: 604.3 of IRC:22-2008)
MOD. RATIO		=	15	(Cl: 604.3 of IRC:22-2008)
COEFF. OF THERMAL EXPENSION	α	=	1.2 x10^$^{-5}$ /ºC	

STRIP	TOP WIDTH m	BOT. WIDTH m	THICKNESS m	AREA m^2	Y m	AY m^3	AY^2 m^4
1	0.1667	0.1667	0.1500	0.0250	0.0750	0.0019	0.0001
2	0.1667	0.1667	0.1000	0.0167	0.2000	0.0033	0.0007
3	0.4000	0.4000	0.0300	0.0120	0.2650	0.0032	0.0008
4	0.0140	0.0140	0.3700	0.0052	0.4650	0.0024	0.0011
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Total			0.0588		0.0108	0.0028

by solving equation

εo Σ A	-	θ Σ Ay	=	α Σ At	---------------------	(3)
εo Σ Ay	-	θ Σ Ay2	=	α Σ Ayt	---------------------	(4)

T Deg:	AT m^2 -Deg	AYT m^3 - Deg
-2.800	-0.070	-0.005
0.000	0.000	0.000
-0.300	-0.004	-0.001
-4.300	-0.022	-0.010
0.000	0.000	0.000
	-0.096	-0.017

Eigen stress fci

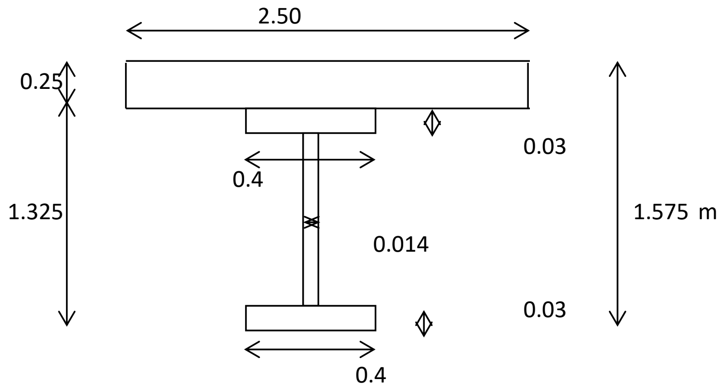
fci = Ec [εo - (y x θ + α x t)]

STRAIN AT TOP FIBER, ε	=	-2.24228E-05
ROTATION OF PLANE, θ	=	-1.56543E-05 rad/m

Y m	T Deg	Yθ Rad:	α t	ε . Therm Strain	σ .therm Mpa
0.000	-5.600	0.000	0.000	0.000	1.325
0.250	0.000	0.000	0.000	0.000	-0.548
0.250	0.000	0.000	0.000	0.000	-0.548
0.650	-8.000	0.000	0.000	0.000	2.477

EIGEN STRESS AT TOP OF SLAB	=	-1.3245 N/MM2	Tension
EIGEN STRESS AT TOP OF GIRDER.	=	0.5475 N/MM2	Compression
EIGEN STRESS AT BOTTOM	=	-2.4774 N/MM2	Tension

CALCULATION OF SHRINKAGE STRESSES IN COMPOSITE GIRDER(MID SPAN)



Strength of concrete	F_{ck}	=	40 Mpa
Mod. Of Elasticity of concrete	E_c	=	15811.388 Mpa
Mod. Of Elasticity of steel	E_s	=	210000 Mpa
Modular Ratio	m	=	13.28
Coeff. Of thermal expansion	a	=	1.20E-05
Shrinkage Strain	ε_n	=	0.0002
Cross-section area of steel	A_s	=	41710.00 mm^2
Cross-section area of slab	A_c	=	625000 mm^2
Moment of intertia of steel beam	I_s	=	1.24E+10 mm^4

Stresses due to shrinkage

f_{st},f_{sb} = $\varepsilon_n E_s(r^2 \underline{+} Dz/2)/(q^2+r^2+z^2)$

D = 1575.000 mm

z = 537.50 mm

q^2 = $m(I_s/A_c)$ = 2.64E+05 mm^2

r^2 = I_s/A_s = 2.98E+05 mm^2

Stresses at top

f_{st} = 35.60 Mpa

Stresses at bottom

f_{sb} = -6.19 Mpa

DESIGN OF WALL TYPE ABUTMENT WITH OPEN FOUNDATION

FOR MINOR BRIDGE AT CH. 152+790

Details of Superstructure:

Skew Angle of Bridge	=	36	Degree	=	0.628 Radians	COS θ =	0.809
						SIN θ =	0.588

Radius of Curvature of Superstructure	=	0	m
Design speed of vehicle	=	100	kmph

		Right Dimensions	Skew Dimensions
Span -c/c of Brg.	=	7.750m	9.580m
Thickness of Expansion Joint	=	0.020m	0.025m
Slab projection Beyond C/L of Bearing (Back Side)	=	0.170m	0.210m
Slab projection Beyond C/L of Bearing (Span Side)	=	0.170m	0.210m
Span -c/c of E.J.	=	8.110m	10.02m
Type of Superstructure	=	RCC SOLID SLAB	
Width of Crash barrier (Both Side)	=	0.500m	
Width of Carriageway	=	8.000m	
Projection beyond crash barrier	=	0.000m	
Thickness of Wearing coat	=	0.065m	
Length of Approach Slab (Right)	=	3.500m	3.500m
Width of Footpath on both side	=	1.500m	
Railing/kerb on footpath edge	=	0.500m	
Total Width of Superstructure	=	11.000m	
Median Width minus 20mm gap	=	0.480m	

Bearings

Type of Bearing	=	Tar Paper Bearing	
Coeff. Of Friction for POT-PTFE Bearing	=	0.5	
Type of Soil	=	1	Hard or Rocky Strata
NBC of soil -Normal Case	=	250	kN/m^2 (as per geotechnical report with ground improvement)
SBC of soil-Normal Case	=	281	kN/m^2
SBC of soil-Seismic Case	=	351	kN/m^2
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	0.371m
Width of Dirt wall at Bottom	=	0.300m	0.371m
Height of Uniform portion	=	0.600m	
Height of Trapering portion	=	0.141m	
Length of Dirt Wall at top (Uniform portion)	=	11.240m	13.893m
Length of Dirt Wall at bottom (Tapering Portion)	=	11.240m	13.893m

Abutment Cap

Width of Abutment cap of Uniform portion	=	0.720m	0.890m
Width of Abutmentcap at bottom of Tapering Portion	=	0.720m	0.890m
Projection of Abutment Cap (Span Side)	=	0.000m	0.000m
Projection of Abutment Cap Back Side	=	0.000m	0.000m
Abutmentcap thickness (Uniform portion)	=	0.300m	
Abutmentcap thickness (Tapering Portion)	=	0.000m	
Length of Abutment Cap at top (Uniform portion)	=	11.240m	13.893m
Length of Abutment Cap at bottom (Tapering Portion)	=	11.240m	13.893m

Abutment- Wall Type

Thickness of Abutment	=	0.720m	
Width of abutment shaft	=	11.240m	13.893m
Thickness of Abutment shaft at Top	=	0.720m	0.890m
Thickness of Abutment shaft at HFL	=	0.876m	1.083m
Thickness of Abutment shaft at Bottom	=	1.000m	1.236m

Solid Return Wall

Length of Return wall	=	3.500m
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	=	0.600m
Height of wall at abutment	=	2.667m
Length of Return wall	=	4.001m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation

Along Traffic Direction:

Total Width of Footing	=	7.000m SQ
abutment pedestal width	=	1.000m
abutment pedestal Height	=	0.000m
Width of Toe Slab	=	2.500m
Width of Heel Slab	=	3.500m
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	=	3.500m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	7.575m
Height of back fill at back fill edge of heel slab	=	6.875m

Across Traffic Direction:

Width of foundation -Uniform portion	=	13.893m (skew dimension)
Width of foundation -Tapering portion	=	13.893m (skew dimension)

Levels

Deck Level at Median Edge=	**1955.575m**	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	**1955.325m**	Height of Superstructure	=	0.800m
Deck level at center line =	1955.575m	Min. Height of Footpath Side Pedestal (1)	=	0.000m
Soffit Level at center of bridge =	1954.710m	Height of Pedestal (2)	=	0.000m
Abutment cap top level =	1954.709m	Height of Pedestal (3)	=	0.000m
Abutment cap bottom lvl (uniform portion ends	1954.409m	Height of Pedestal (4)	=	0.000m
Abutment cap bottom lvl (corbel portion ends)	1954.409m	Distance of nearest girder to c.l. of deck	=	0.000m
Abutment shaft top level =	1954.409m	Height (Avg.) of Dirt Wall	=	0.741m
Ground level/LBL =	1950.651m	Abutment shaft Above G.L	=	3.758m
Abutment shaft bottom level =	1948.575m	Abutment Shaft below G.L	=	2.076m
Foundation level =	1947.575m	Height of abutment shaft	=	5.834m
HFL	1951.151m	MSL	=	1950.651m
		Wedge over girder flange	=	0.0020m

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 Characteistic Strength, εc2	=	0.02	
Ultimate Strain, εcu2	=	0.035	
Ecm	=	32308.250	N/mm^2
Steel Grade	=	Fe 500D	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa (1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00	Mpa
Dry weight of Concrete	=	25	kN/m^3
Dry unit weight of soil	=	20	kN/m^3
Permissible Crack Width	=	0.3	mm - For Moderrate/ severe Exposure Condition
Maximum compressive stress in concrete under rare combination	=	0.48 fck	
	=	16.8	N/mm^2
Maximum tensile stress in steel under rare combination	=	300	N/mm^2

Creep Coefficient

For Abutment Shaft	=	1.2	for 365 days
For Footing	=	1.2	for 365 days

Clear Cover to Reinforcement

Earth Face	=	75	mm
Non-Earth Face	=	50	mm

Seismic Data: NO NEED TO CHECK FOR SEISMIC EFFECT

Seismic Zone		=	5	
Z	=Zone factor	=	0	
I	=Importance factor	=	1.2	
R	=Response Reduction factor	=	3	in Longitudinal direction
		=	1	In Transverse direction

Properties of backfill material :

c	=	0
ϕ	=	30
θ	=	90
β	=	0
δ	=	20.0

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.130m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.148m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

Self weight of Slab	=	0.80	x	10.02	x	11.00	x	25.00
	=	2205.50	KN					
Reaction at one end	=	1102.75	KN					
Transverse Eccentricity	=	0.000	m					

Super Imposed Dead Load Reactions (Excluding Wearing Course)

Weight of Crash barrier	=	2	x	8.00	x	10.02
	=	160.40	KN			
Reaction at one end	=	80.20	KN			
Transverse Eccentricity	=	0.00	m			

Reaction Due to Wearing Course only

Weight due to Wearing Coat	=	2.2	x	10.024984	x	11
	=	242.6046	KN			
Reaction at one end	=	121.3023	KN			
Transverse Eccentricity	=	0.00	m			

Carriageway Live Load Reactions

Reduction Factor = **0.9** (for 3 Lane)

Congestion factor = **1** (As per Table 3 of IRC :112-2014)

MAXIMUM REACTION CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.75	1.94

Min CWLL

Vertical	Transverse ecc
465.85	2.35

SV Loading

Max CWLL

Vertical	Transverse ecc
2564.95	0.30

Min CWLL

Vertical	Transverse ecc
855.05	0.30

MAXIMUM TRASVERSE MOMENT CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.75	1.94

Min CWLL

Vertical	Transverse ecc
465.85	2.35

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for Cl A Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

VOLUME CALCULATION

C.G. Of Footing	=	3.500 m
C.G. Of shaft from toe tip	=	3.000 m
Distance between c.g. of shaft and footing	=	0.500 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	13.89	0.300	0.600	2.501	-0.210	0.290	-3.210	0.000
	-Trapering portion	1	13.89	0.300	0.141	0.588	-0.210	0.290	-3.210	0.000
Bracket (Rectangle)		1	13.89	0.300	0.300	1.250	-0.510	-0.010	-3.510	0.000
(Corbel)	0.5	1	13.89	0.300	0.300	0.625	-0.460	0.040	-3.460	0.000
Cap (uniform portion)		1	13.89	0.720	0.300	3.001	0.000	0.500	-3.000	0.000
Cap (Corbel Portion)		1	13.89 13.89	0.720 0.720	0.000	0.000	0.000	0.500	-3.000	0.000
Shaft above HFL		1	13.89	0.798	3.258	36.130	0.100	0.600	-2.900	0.000
Shaft below HFL		1	13.89	0.938	2.276	29.667	0.030	0.530	-2.970	0.000
Solid Return Wall		1	3.50	0.500	7.350	12.863	-2.368	-1.868	-5.368	6.697
Cantilever Return wall(Rectangular portion)		1	4.00	0.500	0.600	1.200	-2.618	-2.118	-5.618	6.697
Cantilever Return wall(Traingular portion)		1	4.00	0.500	2.067	2.067	-1.952	-1.452	-4.952	6.697
Footing										
Heel Slab		1	13.89	3.500	0.650	31.607		-1.436	-4.936	0.000
Toe Slab		1	13.89	2.500	0.650	22.577		2.026	-1.474	0.000
Portion between Heel and Toe		1	13.89	1.000	1.000	13.893		0.500	-3.000	0.000
Back filling above HFL over Heel Slab		1	13.89	3.500	4.424	215.125		-1.750	-5.250	0.000
Back filling below HFL over Heel Slab		1	13.89	3.500	2.926	142.282		-1.820	-5.320	0.000
Backfill above Heel slab		1	13.89	3.500	7.225	351.329		-1.778	-5.278	0.000
Front Filling over Toe Slab		1	13.89	2.500	2.426	84.263		2.190	-1.310	0.000
Side filling between heel and toe		1	0.00	1.000	2.426	0.000		0.000	0.000	0.000
Aproach Slab		1	13.893	1.750	0.300	7.294	-0.510	-0.010	-3.510	0.000
Back fill above HFL on flared portion of stem		1	13.89	0.156	4.424	9.611		0.106	-3.394	0.000
Back fill below HFL on flared portion of stem		1	13.89	0.124	2.926	5.026		0.041	-3.459	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barri	2	8 kN/m	1.750	28.00kN		-0.210	0.290	-3.210

SECTIONAL PROPERTIES

Width of Footing (B) = 7 m
Length of Footing (L) = 13.893 m

A = 7.000 x 13.893 = 97.254 m^2
ZL = 13.893 x 8.167 = 113.463 m^3
ZT = $\frac{IT1 + IT2}{\text{distance of extreme point from centre}}$

IT1 = 7.000 x 56.937 = 398.56 m^4
IT2 (moment of inertia of triangle) = 7.000 x 3.654 + 0.500 x 7.000 x 5.086 x 37.199
from centre of footing = 687.725 m^4

Moment of inertia of two triangle = 1375.450 m^4

Total moment of inertia = 1774.01 m^4

Distance of extreme point from centre of footing = 4.404 + 5.086 = 9.490 m

Total Section modulus (ZT) = 186.942 m^3

Load Factors (As per IRC:6-2014)

Table 3.1 Partial Safety Factor For Verification of Equilibrium

-Refer Table 3.1 of IRC:6-2014

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Laod,SIDL & Backfill except wearing course	1.050	0.950	1.050	0.950
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.500	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table 3.3 Partial Safety Factor For Verification of Servicibilty Limit State

-Refer Table 3.3 of IRC:6-2014

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Laod+SIDL including wearing course	1.000	1.00	1.00
wearing course	1.200	1.20	1.20
Back Filling Weight	1.000	1.00	1.00
Shrinkage Creep Effect	1.000	1.00	1.00
Earth Pressure due to back filling	1.000	1.000	1.000
CWLL and Associate load and FPLL	1.000	0.750	0.000
Live Load Surcharge	0.800	0.00	0.00

Table 3.4 Partial Safety Factor For Design of Foundation

-Refer Table 3.4 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Possible Load Combination

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

Seismic Coefficient Calculation

(As Per IRC:6-2014 , Clause 219)

Horizontal Seismic Force For Zone 5.0

Feq = Seismic forces to be resisted
Feq = Ah x (Dead load + Appropriate Live load)
Ah = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \cdot \frac{Sa}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0	
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.

$$= 2.0 \left(\frac{D}{1000F}\right)^{1/2}$$

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
= 7.134 m
C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction
= 7.711 m

Abutment Cap Top Level - Foundation Level
= 7.134 m

Dimensions of Abutment Shaft

Length	=	13.89	m
Width	=	0.86	m
Moment of Inertia , $I_{longitudinal}$	=	0.736	m^4
Moment of Inertia , $I_{transverse}$	=	192.195	m^4
Ecm	=	3.231E+07	kN/m^2

Longitudinal Direction				**Transverse Direction**			
Force	=	98.294	KN	Force	=	22876.612	KN
D	=	1304.25	KN	D	=	1583.970	KN
T	=	0.2304	sec	T	=	0.0166	sec

Hard or Rocky Strata

Sa/g	=	2.5	Sa/g	=	2.5

Seismic Coeff. In Longitudinal Direction = 0

Seismic Coeff. In Transverse Direction = 0

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

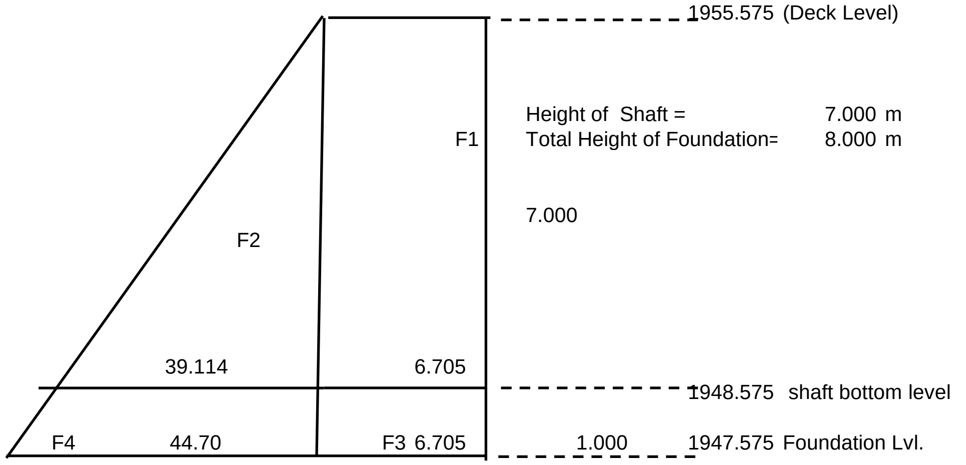
For Design of Foundation (35% increment in Seismic Coeff for Foundation as per IRC:6-2014,Clause No. 219.8)

Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0		
	ϕ	=	30 degree	0.524	radians
	θ	=	90.00 degree	1.571	radians
	$\theta 1$	=	90.00 degree	1.571	radians
	β	=	0	0	radians
	δ	=	20.0 degree	0.349	radians
	Kah	=	0.279 active component		
	Kph	=	3.766 Passive component		
	γ	=	20 kN/m^3		

Equivalent Live Load Surcharge height = 1.2 m
Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL **1948.575 m (at Shaft Base)**
@ RL **1947.575 m (at Foundation Level)**

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.2	=	6.705 kN/m^2	
F1	=	6.705	x	7.000	x	13.893	=	**652.107 kN**	
M1	=	652.11	x	3.50	=	**2282.375 kN.m**		**at Shaft Bottom**	
F3	=	6.705	x	8.000	x	13.893	=	**745.265 kN**	
M3	=	745.265	x	4.000	=	**2981.061 kN.m**		**at Foundation**	

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)

	=	0.279	x	20	x	7.000	=	39.114 kN/m^2
F2	=	0.5	x	39.11	x	7.000	x	13.89
	=	**1901.979 kN**						

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

M2	=	1901.98	x	2.94	=	**5591.818 kN.m**	**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	0.279	x	20	x	8.000	=	44.701 kN/m^2
F4	=	0.5	x	44.70	x	8.000	x	13.89
	=	**2484.217 kN**						
M4	=	2484.22	x	3.36	=	**8346.971 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	7.000	=	33.600 kN/m^2		
F	=	0.5	x	33.600	x	7.000	x	13.893
	=	**1633.864 kN**						

M = 1633.86 x 2.333 = **3812.350 kN.m** **at Shaft Bottom**

Intensity for triangular portion (At Foundation level)
= 4.800 x 8.000 = 38.40 kN/m^2

F = 0.5 x 38.400 x 8.00 x 13.893
= **2134.027 kN**

M = 2134.03 x 2.667 = **5690.738 kN.m** **at Foundation**

Intensity of Passive pressure
= 3.766 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 13.89
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m** **at Foundation**

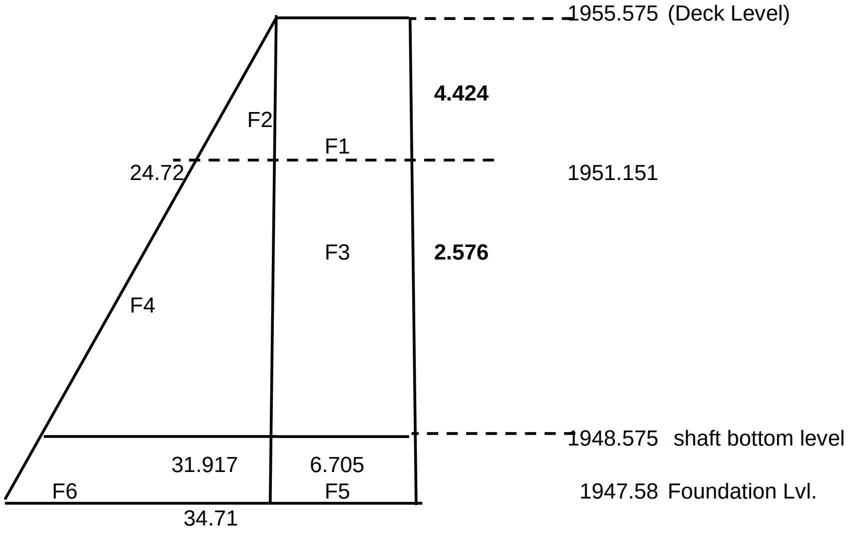
Summary of Moment and Horizontal Force

	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	5591.818	8346.971	1901.979	2484.217
Due to Minimum Fluid Pressure	3812.350	5690.738	1633.864	2134.027
Governing of Two	**5591.818**	**8346.971**	**1901.979**	**2484.217**
Due to Live Load Surcharge	**2282.375**	**2981.061**	**652.107**	**745.265**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	30 degree	0.524 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	20.0 degree	0.349 radians
	Kah	=	0.279 active component	
	Kph	=	3.766 passive component	
	γd	=	20 kN/m^3	
	γwater	=	10 kN/m^3	
Equivalent Live Load Surcharge height		=	1.2 m	

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1948.6 m (at Shaft Base)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.200	=	6.705 kN/m^2	
F1	=	6.705	x	7.000	x	13.893	=	**652.107 kN**	
M1	=	652.11	x	3.50	=	**2282.375 kN.m**		**at Shaft Bottom**	
F3	=	6.705	x	8.000	x	13.893	=	**745.265 kN**	
M3	=	745.27	x	4.00	=	**2981.061 kN.m**		**at Foundation Level**	

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.279	x	20	x	4.424	=	24.720 kN/m^2
(At Shaft bottom level) Below HFL	=	0.279	x	10	x	2.576	=	7.197 kN/m^2
F2	=	0.5	x	24.72	x	4.424	x	13.89
	=	**759.696 kN**						

$$F4 = \frac{(24.72 + 31.92)}{2} \times 2.58 \times 13.89$$

= **1013.496 kN**

Total Force = 1773.192 kN

M2	=	759.70	x	4.43	=	**3368.553 kN.m**
M4	=	1013.50	x	1.23	=	**1250.091 kN.m**

Total Mome = 4618.64 kN.m at Shaft Bottom

Intensity for

triangular portion

Upto HFL = 0.279 x 20 x 4.424 = 24.720 kN/m^2

at Foundation level = 0.279 x 10 x 3.576 = 9.991 kN/m^2

F2 = 0.5 x 24.72 x 4.424 x 13.89
= **759.696 kN**

F6 = $\frac{(24.72 + 34.71)}{2}$ x 3.58 x 13.89

= **1476.337 kN**

Total Force = 2236.033 kN

M2 = 759.70 x 5.43 = **4128.249 kN.m**

M6 = 1476.34 x 1.69 = **2491.772 kN.m**

Total Mome = 6620.02 kN.m Foundation Lvl.

Intensity of Passive pressure:
= 3.766 x 10 x 0.00 = 0.000 kN/m^2
Force due to passive @ Foundation, F
= 0.5 x 0.000 x 0.00 x 13.89
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m Foundation Lvl.**

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	4618.644	6620.022	1773.192	2236.033
Due to Minimum Fluid Pressure	3812.350	5690.738	1633.864	2134.027
Governing of Two	**4618.644**	**6620.022**	**1773.192**	**2236.033**
Due to Live Load Surcharge	**2282.375**	**2981.061**	**652.107**	**745.265**
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

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

α h	=	0.000	
α v	=	0.000	
φ	=	30.00	0.524
δ	=	20.00	0.349
α	=	0.00	0.000
β	=	0.00	0.000

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

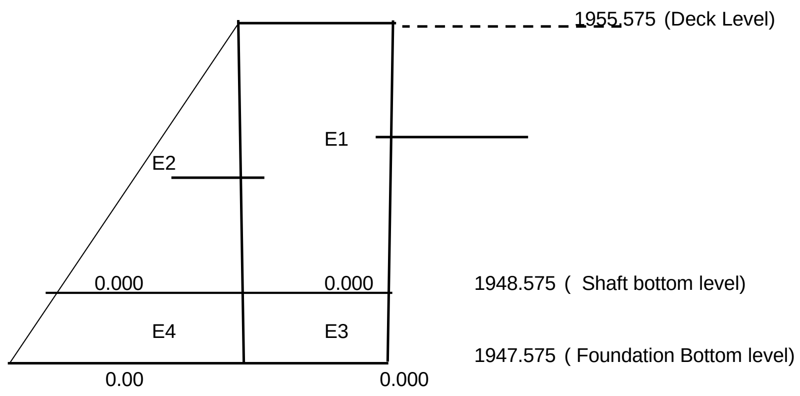
$\lambda = \tan^{-1} \dfrac{\alpha\ h}{(1 \pm \alpha\ v)}$ = 0.000 / 0.000

		1	2
C_a	=	0.279	0.279

Ca	=	0.279		
Ka	=	**0.279**		
Dynamic Increment	=	0.279	-0.279	**0.000**

3 Earth Pressure : **DRY CASE (Seismic case)**

Equivalent Live Load Surcharge height		=	1.2 m
Assuming	γdry	=	20 kN/m^3
	γwater	=	10.00 kN/m^3



Earth Pressure Diagram for Dynamic Increment

Horizontal Forces and Moments @ RL **1948.6 m (at Shaft Base)**
1947.6 m (at Foundation Bottom Level)

Due to Dynamic Live Load Surcharge

	=	0.000	x	20	x	1.2	=	0.000	kN/m^2
at Shaft Bottom Level									
E1	=	0.000	x	7.000	x	13.893	=	**0.000**	kN
M1	=	0.000	x	4.690			=	**0.000**	kN.m
at Foundation Bottom Level									
E3	=	0.000	x	8.000	x	13.893	=	**0.000**	kN
M3	=	0.000	x	5.360			=	**0.000**	kN.m

Due to Dynamic Active Earth Pressure

(At Shaft bottom level)

	=	0.000	x	20	x	7.000	=	0.000	kN/m^2

(at Foundation Bottom Level)

	=	0.000	x	20	x	8.000	=	0.000	kN/m^2
E2	=	0.50	x	0.00	x	7.00	x	13.893	
	=	**0.000**	kN						
E4	=	0.50	x	0.00	x	8.00	x	13.893	
	=	**0.000**	kN						
M2	=	0.00	x	3.50	=	**0.000**	kN.m	**(Shaft bottom level)**	
M4	=	0.00	x	4.00	=	**0.000**	kN.m	**(Foundation Bottom level)**	

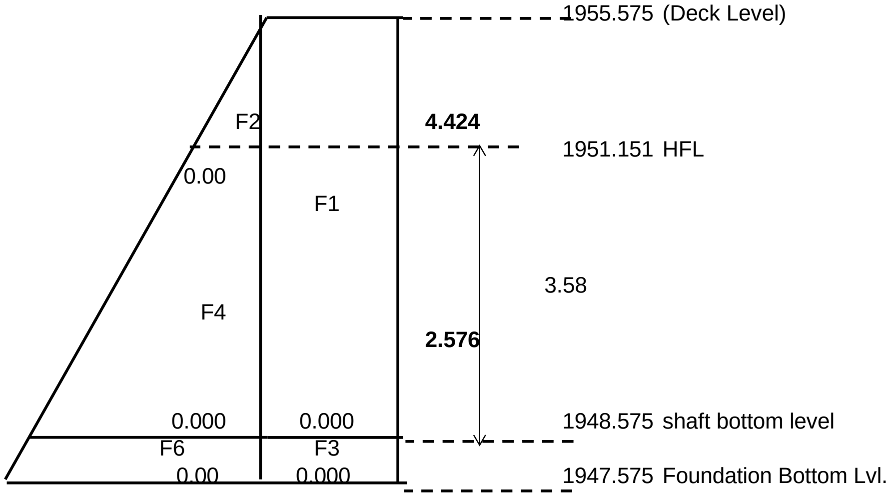
Summary of Moment and Horizontal Force **Dry Seismic Case**

	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)	5591.818	8346.971	1901.979	2484.217
Due to active Earth Pressure (dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	5591.818	8346.971	1901.979	2484.217
Due to Minimum Fluid Pressure	3812.350	5690.738	1633.864	2134.027
Governing of Two	**5591.818**	**8346.971**	**1901.979**	**2484.217**
Due to Live Load Surcharge (Static)	**2282.375**	**2981.061**	**652.107**	**745.265**
Due to Live Load Surcharge(Dynamic)	**0.000**	**0.000**	**0.000**	**0.000**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Dynamic Increment	=	0.000
γd	=	20 kN/m^3
γwater	=	10 kN/m^3
Equivalent Live Load Surcharge height	=	1.2 m

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL **1948.575 m (at Shaft Base)**
1947.575 m (at Foundation Bottom Level)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.000	x	20	x	1.200	=	0.000 kN/m^2

at Shaft Bottom Level

F1	=	0.000	x	7.000	x	13.893	=	**0.000 kN**
M1	=	0.00	x	4.62	=	**0.000 kN.m**		

at Foundation Bottom Level

F3	=	0.000	x	8.000	x	13.893	=	**0.000 kN**
M3	=	0.00	x	5.28	=	**0.000 kN.m**		

Due to Dynamic Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.000	x	20	x	4.424	=	0.000 kN/m^2
(At Shaft bottom level) Below HFL	=	0.000	x	10	x	2.576	=	0.000 kN/m^2
(At Foundati bottom level) Below HFL	=	0.000	x	10	x	3.576	=	0.000 kN/m^2

F2 = 0.5 x 0.00 x 4.42 x 13.89
= **0.000 kN**

$$F4 = \frac{(0.00 + 0.00)}{2} \times 2.58 \times 13.89$$
= **0.000 kN**

$$F6 = \frac{(0.00 + 0.00)}{2} \times 3.58 \times 13.89$$
= **0.000 kN**

Total Force (F2 + F4)	**=**	**0.000 kN**	**at Shaft Bottom Level**
Total Force (F2 + F6)	**=**	**0.000 kN**	**at Foundation Bottom Level**

M2	=	0.00	x	4.79	=	**0.000 kN.m**
M4	=	0.00	x	0.00	=	**0.000 kN.m**

Total Mome	**=**	**0.000**	**kN.m**	**at Shaft Bottom**			
M2	=	0.00	x	5.79	=	**0.000**	**kN.m**
M6	=	0.00	x	0.00	=	**0.000**	**kN.m**
Total Mome	**=**	**0.000**	**kN.m**	**at Foundation Bottom Level**			

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom Lvl kN	At Foundation Bottom Lvl kN
Due to active Earth Pressure(Static	4618.644	6620.022	1773.192	2236.033
Due to active Earth Pressure (Dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	4618.644	6620.022	1773.192	2236.033
Due to Minimum Fluid Pressure	3812.350	5690.738	1633.864	2134.027
Governing of Two	**4618.644**	**6620.022**	**1773.192**	**2236.033**
Due to Live Load Surcharge(Stat	**2282.375**	**2981.061**	**652.107**	**745.265**
Due to Live Load Surcharge (Dynamic Increment)	**0.000**	**0.000**	**0.000**	**0.000**
Due to passive pressure		**0.000**		**0.000**

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1102.75	1.35	1.35	1488.71	1488.71	
SIDL except wc	=	80.20	1.35	1.35	108.27	108.27	
WC	=	121.30	1.75	1.75	212.28	212.28	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmin	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmin	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmax-Transv. Moment Case		0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A

Braking Force = 0.2 x 1000 + 0.05 x 554

= 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1809.26	0	904.630	904.630		
DL+SIDL+LL-Max Reaction case	1809.26	341.55	904.630	904.630	1- 70RW + 2-CLASS A	Dry Case
	1809.26	0	904.630	904.630	SV Loading	
DL+SIDL+LL-Min Reaction case	1809.26	341.55	904.630	904.630	1- 70RW + 2-CLASS A	HFL Case
	1809.26	0	904.630	904.630	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1809.26	341.55	904.630	904.630		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1809.26	0.00	904.630	904.630	
DL+SIDL+LL-Max Reaction case		1809.26	45.54	904.630	904.630	Dry Case
DL+SIDL+LL-Min Reaction case		1809.26	45.54	904.630	904.630	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1809.26	45.54	904.630	904.630	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1809.26	0.000	904.630	904.630	
DL+SIDL+LL-Max Reaction case		1809.26	45.540	904.630	904.630	Dry Case
DL+SIDL+LL-Min Reaction case		1809.26	45.540	904.630	904.630	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1809.26	45.540	904.630	904.630	

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1102.75	1.35	1.35	1488.71	1488.71
SIDL except wc	=	80.20	1.35	1.35	108.27	108.27
WC	=	121.30	1.75	1.75	212.28	212.28
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1.5	0.75	0.00	0.00
CWLLmin	=	0.00	1.5	0.75	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1809.26	0.000	904.630	904.630		
DL+SIDL+LL-Max Reaction case	1809.26	341.550	904.630	904.630	1- 70RW + 2-CLASS A	Dry Case
	1809.26	0.000	904.630	904.630	SV Loading	
DL+SIDL+LL-Min Reaction case	1809.26	341.550	904.630	904.630	1- 70RW + 2-CLASS A	HFL Case
	1809.26	0.000	904.630	904.630	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1809.26	341.550	904.630	904.630		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1809.26	0.00	904.630	904.630	
DL+SIDL+LL-Max Reaction case		1809.26	45.54	904.630	904.630	Dry Case
DL+SIDL+LL-Min Reaction case		1809.26	45.54	904.630	904.630	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1809.26	45.54	904.630	904.630	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1809.26	0.000	904.630	904.630	
DL+SIDL+LL-Max Reaction case		1809.26	45.540	904.630	904.630	Dry Case
DL+SIDL+LL-Min Reaction case		1809.26	45.540	904.630	904.630	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1809.26	45.540	904.630	904.630	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1102.75	1	1.00	1102.75	1102.75
SIDL except wc	=	80.20	1	1.00	80.20	80.20
WC	=	121.30	1	1.00	121.30	121.30
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1	0.20	0.00	0.00
CWLLmin	=	0.00	1	0.20	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1304.25	0.000	652.125	652.125		
DL+SIDL+LL-Max Reaction case	1304.25	227.700	652.125	652.125	1- 70RW + 2-CLASS A	Dry Case
	1304.25	0.000	652.125	652.125	SV Loading	
DL+SIDL+LL-Min Reaction case	1304.25	227.700	652.125	652.125	1- 70RW + 2-CLASS A	HFL Case
	1304.25	0.000	652.125	652.125	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1304.25	227.700	652.125	652.125		

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	0.000	652.125	652.125	
DL+SIDL+LL-Max Reaction case	1304.25	45.540	652.125	652.125	Dry Case
DL+SIDL+LL-Min Reaction case	1304.25	45.540	652.125	652.125	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1304.25	45.540	652.125	652.125	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	0.000	652.125	652.125	
DL+SIDL+LL-Max Reaction case	1304.25	45.540	652.125	652.125	Dry Case
DL+SIDL+LL-Min Reaction case	1304.25	45.540	652.125	652.125	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1304.25	45.540	652.125	652.125	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1102.75	1.05	1.05	1157.89	1157.89
SIDL except wc	=	80.20	1.05	1.05	84.21	84.21
WC	=	121.30	1.35	1.35	163.76	163.76
FPLL	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1.5	0.00	0.00	0.00
CWLLmin	=	0.00	1.5	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1405.85	0.000	702.927	702.927		
DL+SIDL+LL-Max Reaction case	1405.85	341.550	702.927	702.927	1- 70RW + 2-CLASS A	Dry Case
	1405.85	0.000	702.927	702.927	SV Loading	
DL+SIDL+LL-Min Reaction case	1405.85	341.550	702.927	702.927	1- 70RW + 2-CLASS A	HFL Case
	1405.85	0.000	702.927	702.927	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1405.85	341.550	702.927	702.927		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1405.85	0.00	702.927	702.927	
DL+SIDL+LL-Max Reaction case		1405.85	0.00	702.927	702.927	Dry Case
DL+SIDL+LL-Min Reaction case		1405.85	0.00	702.927	702.927	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1405.85	0.00	702.927	702.927	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1304.25	1405.85	0.000	702.927	702.927	
DL+SIDL+LL-Max Reaction case		1405.85	0.000	702.927	702.927	Dry Case
DL+SIDL+LL-Min Reaction case		1405.85	0.000	702.927	702.927	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1405.85	0.000	702.927	702.927	

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	=	1102.75	1	1	1	1102.75	1102.75	1102.75
SIDL except wc	=	80.20	1	1	1	80.20	80.20	80.20
WC	=	121.30	1.20	1.20	1.20	145.56	145.56	145.56
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmin	=	0.00	1	0.75	0	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1328.51	0.000	664.255	664.255		
DL+SIDL+LL-Max Reaction case	1328.51	227.700	664.255	664.255	1- 70RW + 2-CLASS A	Dry Case
	1328.51	0.000	664.255	664.255	SV Loading	
DL+SIDL+LL-Min Reaction case	1328.51	227.700	664.255	664.255	1- 70RW + 2-CLASS A	HFL Case
	1328.51	0.000	664.255	664.255	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1328.51	227.700	664.255	664.255		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1328.51	0.000	664.255	664.255	
DL+SIDL+LL-Max Reaction case	1328.51	170.775	664.255	664.255	Dry Case
DL+SIDL+LL-Min Reaction case	1328.51	170.775	664.255	664.255	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1328.51	170.775	664.255	664.255	

Normal Case: Quasi Permenant Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1328.51	0.000	664.255	664.255	
DL+SIDL+LL-Max Reaction case	1328.51	0.000	664.255	664.255	Dry Case
DL+SIDL+LL-Min Reaction case	1328.51	0.000	664.255	664.255	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1328.51	0.000	664.255	664.255	

Centrifugal Force Calculation

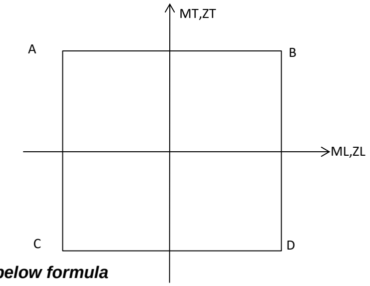
As per clause 212 of IRC:6-2014

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

			Normal Case		Seismic Case	
Design Speed	V	=	100.00	kmph	100.00	kmph
Live Load	W	=	932.75	kN	932.75	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1947.575 m



Properties of Footing Base:

A	=	97.254	m^2
ZL	=	113.463	m^3
ZT	=	186.942	m^3

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

Moment along longitudinal axis	**ML = ML Cos θ + MT Sin θ**
Moment along transverse axis	**MT = MT Cos θ - ML Sin θ**

Case 1 : DL+SIDL-Normal Dry Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure												
Dead Load	1			1102.748	0.630	694.731	0.000	0.000	0.950	1047.611	-2.870	-3006.643
SIDL except Wearing Course	1			80.200	0.630	50.526	0.000	0.000	0.950	76.190	-2.870	-218.665
Wearing Course	1			121.302	0.630	76.420	0.000	0.000	1.000	121.302	-2.870	-348.138
				1304.250		**821.678**		**0.000**		**1245.103**		**-3573.446**
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	1	25	2.501	62.520	0.290	18.131	0.000	0.000	0.950	59.394	-3.210	-190.656
Dirt Wall-Tapered portion	1	25	0.588	14.692	0.290	4.261	0.000	0.000	0.950	13.958	-3.210	-44.804
Bracket - Uniform portion	1	25	1.250	31.260	-0.010	-0.313	0.000	0.000	0.950	29.697	-3.510	-104.237
Bracket - Tapered portion	1	25	0.625	15.630	0.040	0.625	0.000	0.000	0.950	14.849	-3.460	-51.376
Cap - (uniform portion)	1	25	3.001	75.024	0.500	37.512	0.000	0.000	0.950	71.273	-3.000	-213.819
Cap - (corbel portion)	1	25	0.000	0.000	0.500	0.000	0.000	0.000	0.950	0.000	-3.000	0.000
Cantilever Return Wall-Rectangle po	1	25	1.200	30.004	-2.118	-63.556	6.697	200.926	0.950	28.504	-5.618	-160.141
Cantilever Return Wall-Traingle port	1	25	2.067	51.681	-1.452	-75.017	6.697	346.095	0.950	49.097	-4.952	-243.107
RCC Railing or Crash Barrier	1			28.000	0.290	8.120	0.000	0.000	0.950	26.600	-3.210	-85.386
Aproach Slab	1	25	7.294	182.351	-0.010	-1.824	0.000	0.000	0.950	173.233	-3.510	-608.049
				491.163		**-72.061**		**547.022**		**466.605**		**-1701.576**
Substructure & Foundation -Portion 2												
Solid Return wall	1	25	12.863	321.563	-1.868	-600.690	6.697	2153.408	0.950	305.484	-5.368	-1639.851
Abutment Shaft	1	25	65.796	1644.904	0.530	872.177	0.000	0.000	0.950	1562.659	-2.970	-4640.739
Back filling over heel slab	1	20	351.329	7026.589	-1.778	-12495.091	0.000	0.000	0.950	6675.260	-5.278	-35233.745
Front Filling over toe slab	1	20	84.263	1685.270	2.190	3690.551	0.000	0.000	0.950	1601.006	-1.310	-2097.499
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.000	0.000	0.000
Heel slab	1	25	31.607	790.187	-1.436	-1134.628	0.000	0.000	0.950	750.678	-4.936	-3705.270
Toe slab	1	25	22.577	564.420	2.026	1143.311	0.000	0.000	0.950	536.199	-1.474	-790.549
portion between heel & toe	1	25	13.893	347.335	0.500	173.668	0.000	0.000	0.950	329.968	-3.000	-989.905
Vertical Components of active earth pressure	1			904.181	-3.500	-3164.634	0.000	0.000	0.950	858.972	-7.000	-6012.805
				13577.188		**-11493.795**		**2153.408**		**12898.328**		**-56054.251**
Total				**15372.602**		**-10744.178**		**2700.430**		**14610.037**		**-61329.273**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	652.125	1954.710	4652.914
due to Earth pressure	1	2484.217		8346.971

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
527.58	3764.29	383.31	2734.91
2484.22	8346.97	0.00	0.00
3011.798	12111.257	383.310	2734.914

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Earth pressure	1.5	3726.326		12520.456
		4429.253		17535.839

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
3726.33	12520.46	0.00	0.00
4295.006	16577.986	413.170	2947.968

Summary of Forces For SBC		
P	15372.602	KN
ML	1367.079	kNm
MT	5435.344	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		15372.602		-10744.178		2700.430
CWLL-Max. Reaction case	1	932.751	0.630	587.633	1.940	1809.439
Vertical Components of LL Surcharge	1	271.254	-3.500	-949.390	0.000	0.000
Total		**16576.607**		**-11105.935**		**4509.869**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	14610.037		-61329.27
0.000		-2.870	0.00
0.950		-7.000	-1803.84
	14610.037		**-63133.11**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	652.125	1954.710	4652.914
due to Earth pressure	1	2484.217		8346.971
due to Live load surcharge	1	745.265		2981.061

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
527.58	3764.29	383.31	2734.91
2484.22	8346.97	0.00	0.00
745.265	2981.061	0.000	0.000
3757.06	**15092.32**	**383.31**	**2734.91**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Earth pressure	1.5	3726.326		12520.456
due to Live load surcharge	1.2	894.318		3577.273
		5323.571		**21113.112**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
3726.33	12520.46	0.00	0.00
894.32	3577.27	0.00	0.00
5189.324	**20155.259**	**413.170**	**2947.968**

Summary of Forces For SBC		
P	16576.607	KN
ML	3986.383	kNm
MT	7244.783	kNm

Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		15372.602		-10744.178		2700.430
CWLL-Max. Reaction case	1	2564.953	0.630	1615.920	0.300	769.486
Vertical Components of LL Surcharge	1	271.254	-3.500	-949.390	0.000	0.000
Total		**18208.809**		**-10077.648**		**3469.916**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	14610.03659		-61329.2731
0.000		-2.870	0.00
0.950		-7.000	-1803.84
	14610.037		**-63133.11**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	652.125	1954.710	4652.914	527.58	3764.29	383.31	2734.91
due to Earth pressure	1	2484.217		8346.971	2484.22	8346.97	0.00	0.00
due to Live load surcharge	1	745.265		2981.061	745.265	2981.061	0.000	0.000
					3757.06	**15092.32**	**383.31**	**2734.91**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		702.927	1954.710	5015.383	568.68	4057.53	413.17	2947.97
due to Earth pressure	1.5	3726.326		12520.456	3726.33	12520.46	0.00	0.00
due to Live load surcharge	1.2	894.318		3577.273	894.32	3577.27	0.00	0.00
		5323.571		**21113.112**	**5189.324**	**20155.259**	**413.170**	**2947.968**

Summary of Forces For SBC		
P	18208.809	KN
ML	5014.670	kNm
MT	6204.830	kNm

Case 3 : DL+SIDL-Normal HFL Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure				**1304.250**		**821.678**		**0.000**		**1245.103**		**-3573.446**
Substructure & Foundation -Portion 1				**491.163**		**-72.061**		**547.022**		**466.605**		**-1701.576**
Substructure & Foundation -Portion 2												
Solid Return wall	1	25	12.863	321.563	-1.868	-600.690	6.697	2153.408	0.950	305.484	-5.368	-1639.851
Shaft above HFL	1	25	36.130	903.238	0.600	541.611	0.000	0.000	0.950	858.076	-2.900	-2488.737
Shaft below HFL	1	15	29.667	445.000	0.530	235.952	0.000	0.000	0.950	422.750	-2.970	-1255.470
Back filling above HFL over heel sla	1	20	215.125	4302.509	-1.750	-7529.391	0.000	0.000	0.950	4087.384	-5.250	-21458.765
Back filling below HFL over heel slal	1	10	142.282	1422.824	-1.820	-2589.221	0.000	0.000	0.950	1351.682	-5.320	-7190.648
Front Filling over toe slab	1	10	84.263	842.635	2.190	1845.276	0.000	0.000	0.950	800.503	-1.310	-1048.749
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.000	0.000	0.000
Heel slab	1	15	31.607	474.112	-1.436	-680.777	0.000	0.000	0.950	450.407	-4.936	-2223.162
Toe slab	1	15	22.577	338.652	2.026	685.987	0.000	0.000	0.950	321.719	-1.474	-474.329
Portion between Heel & Toe	1	15	13.893	208.401	0.500	104.201	0.000	0.000	0.950	197.981	-3.000	-593.943
Vertical Components of active earth pressure	1			813.849	-3.500	-2848.473	0.000	0.000	0.950	773.157	-7.000	-5412.099
				10315.262		**-10813.088**		**2153.408**		**9799.499**		**-44570.679**
Total				**12110.676**		**-10063.471**		**2700.430**		**11511.207**		**-49845.701**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	652.125	1954.710	4652.914	527.58	3764.29	383.31	2734.91
due to Earth pressure	1	2236.033		6620.022	2236.03	6620.02	0.00	0.00
					2763.61	**10384.31**	**383.31**	**2734.91**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		702.927	1954.710	5015.383	568.68	4057.53	413.17	2947.97
due to Earth pressure	1.5	3354.049		9930.032	3354.05	9930.03	0.00	0.00
		4056.976		**14945.415**	**3922.73**	**13987.56**	**413.17**	**2947.97**

Summary of Forces For SBC		
P	12110.676	KN
ML	320.836	kNm
MT	5435.344	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		12110.676		-10063.471		2700.430
CWLL-Min. Reaction case	1	465.849	0.630	293.485	2.352	1095.807
Vertical Components of LL Surcharge	1	271.254	-3.500	-949.390	0.000	0.000
Total		**12847.779**		**-10719.377**		**3796.237**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	11511.207		-49845.701
0.000	0.000	-2.870	0
0.950	257.692	-7.000	-1803.842
	11768.899		**-51649.542**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	652.125	1954.710	4652.914
due to Earth pressure	1	2236.033		6620.022
due to Live load surcharge	1	745.265		2981.061

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
527.58	3764.29	383.31	2734.91
2236.03	6620.02	0.00	0.00
745.265	2981.061	0.000	0.000
3508.88	**13365.37**	**383.31**	**2734.91**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Earth pressure	1.5	3354.049		9930.032
due to live load surcharge	1.2	894.318		3577.273
		4951.295		**18522.689**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
3354.05	9930.03	0.00	0.00
894.318	3577.273	0.000	0.000
4817.05	**17564.84**	**413.17**	**2947.97**

Summary of Forces For SBC		
P	12847.779	KN
ML	2645.992	kNm
MT	6531.151	kNm

Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		12110.676		-10063.471		2700.430
CWLL-Min. Reaction case	1	855.047	0.630	538.680	0.300	256.514
Vertical Components of LL Surcharge	1	271.254	-3.500	-949.390	0.000	0.000
Total		**13236.977**		**-10474.182**		**2956.944**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	11511.207		-49845.701
0.000	0.000	-2.870	0.00
0.950	257.692	-7.000	-1803.842
	11768.899		**-51649.542**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	652.125	1954.710	4652.914
due to Earth pressure	1	2236.033		6620.022
due to Live load surcharge	1	745.265		2981.061

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
527.58	3764.29	383.31	2734.91
2236.03	6620.02	0.00	0.00
745.265	2981.061	0.000	0.000
3508.88	**13365.37**	**383.31**	**2734.91**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Earth pressure	1.5	3354.049		9930.032
due to live load surcharge	1.2	894.318		3577.273
		4951.295		**18522.689**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
3354.05	9930.03	0.00	0.00
894.318	3577.273	0.000	0.000
4817.05	**17564.84**	**413.17**	**2947.97**

Summary of Forces For SBC		
P	13236.977	KN
ML	2891.187	kNm
MT	5691.858	kNm

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

Seismic Effect Factor = 1 αh= 0.000 In Longitudinal direction

αh= 0.000 In Transverse direction

αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 617.062 KN

Weight of back fill below Ground level = 2018.989 KN

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 = 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			1102.748		0.000	0.000	0.630	694.731	0.000	1955.243		0.000	0.000	0.000
SIDL except Wearing Course	1			80.200		0.000	0.000	0.630	50.526	0.000	1956.024		0.000	0.000	0.000
Wearing Course	1			121.302		0.000	0.000	0.630	76.420	0.000	1955.575		0.000	0.000	0.000
				1304.250		**0.000**	**0.000**		**821.678**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	2.501	62.520	0.000	0.000	0.000	0.290	18.131	0.000	1955.275	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.588	14.692	0.000	0.000	0.000	0.290	4.261	0.000	1954.905	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1	25	1.250	31.260				-0.010	-0.313						
Bracket - Tapered portion	1	25	0.625	15.630				0.040	0.625						
Cap - (uniform portion)	1	25	3.001	75.024	0.000	0.000	0.000	0.500	37.512	0.000	1954.559	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1954.409	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1	25	1.200	30.004	0.000	0.000	0.000	-2.118	-63.556	0.000	1955.275	0.000	6.697	200.926	0.000
Cantilever Return Wall-Traingle port	1	25	2.067	51.681	0.000	0.000	0.000	-1.452	-75.017	0.000	1954.286	0.000	6.697	346.095	0.000
RCC Railing or Crash Barrier	1			28.000				0.290	8.120				0.000	0.000	
Aproach Slab	1	25	7.294	182.351				-0.010	-1.824				0.000	0.000	
				491.163	**0.000**	**0.000**	**0.000**		**-72.061**	**0.000**		**0.000**		**547.022**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	12.863	321.563	0.000	0.000	0.000	-1.868	-600.690	0.000	1951.963	0.000	6.697	2153.408	0.000
Abutment Shaft	1	25	65.796	1644.904	0.000	0.000	0.000	0.530	872.177	0.000	1952.530	0.000	0.000	0.000	0.000
Back filling over heel slab	1	20	351.329	7026.589	0.000	0.000	0.000	-1.778	-12495.091	0.000	1951.963	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1	20	84.263	1685.270				2.190	3690.551				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	31.607	790.187				-1.436	-1134.628				0.000	0.000	
Toe slab	1	25	22.577	564.420				2.026	1143.311				0.000	0.000	
portion between heel & toe	1	25	13.893	347.335				0.500	173.668				0.000	0.000	
Vertical component of active earth pressure	1			904.181				-3.500	-3164.634				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				13577.188	**0.000**	**0.000**	**0.000**		**-11493.795**	**0.000**		**0.000**		**2153.408**	**0.000**
Total =				**15372.602**	**0.000**	**0.000**	**0.000**		**-10744.178**	**0.000**		**0.000**		**2700.430**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect / **For Overturning or Sliding Effect**

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv	Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure												
Dead Load	0.95			1047.611	0.000	-2.870	-3006.643	0.000				
SIDL except Wearing Course	0.95			76.190	0.000	-2.870	-218.665	0.000				
Wearing Course	1.00			121.302	0.000	-2.870	-348.138	0.000				
				1245.103	**0.000**		**-3573.446**	**0.000**				
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	0.95	25	2.501	59.394	0.000	-3.210	-190.656	0.000	1.0	0.000	1955.275	0.000
Dirt Wall-Tapered portion	0.95	25	0.588	13.958	0.000	-3.210	-44.804	0.000	1.0	0.000	1954.905	0.000
Bracket - Uniform portion	0.95	25	1.250	29.697								
Bracket - Tapered portion	0.95	25	0.625	14.849								
Cap - (uniform portion)	0.95	25	3.001	71.273	0.000	-3.000	-213.819	0.000	1.0	0.000	1954.559	0.000
Cap - (corbel portion)	0.95	25	0.000	0.000	0.000	-3.000	0.000	0.000	1.0	0.000	1954.409	0.000
Cantilever Return Wall-Rectangle po	0.95	25	1.200	28.504	0.000	-5.618	-160.141	0.000	1.0	0.000	1955.275	0.000
Cantilever Return Wall-Traingle port	0.95	25	2.067	49.097	0.000	-4.952	-243.107	0.000	1.0	0.000	1954.286	0.000
RCC Railing or Crash Barrier	0.95			26.600		-3.210	-85.386					
Aproach Slab	0.95	25	7.294	173.233		-3.510	-608.049					
				466.605	**0.000**		**-1545.963**	**0.000**		**0.000**		**0.000**
Substructure & Foundation -Portion 2												
Abutment Shaft	0.95	25	65.796	1562.659	0.000	-2.970	-4640.739	0.000	1.0	0.000	1952.530	0.000
Solid Return wall	0.95	25	12.863	305.484	0.000	-5.368	-1639.851	0.000	1.0	0.000	1951.963	0.000
Back filling over heel slab	0.95	20	351.329	6675.260	0.000	-5.278	-35233.745	0.000	1.0	0.000	1951.963	0.000
Front Filling over Pile Cap	0.95	20	84.263	1601.006		-1.310	-2097.499					
Side filling between heel and toe	0.95	20	0.000	0.000		0.000	0.000					
Heel slab	0.95	25	31.607	750.678		-4.936	-3705.270					
Toe slab	0.95	25	22.577	536.199		-1.474	-790.549					
portion between heel & toe	0.95	25	13.893	329.968		-3.000	-989.905					
Vertical component of active earth pressure	0.95			858.972		-7.000	-6012.805					
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000					
				12898.328	**0.000**		**-56054.25**	**0.000**		**0.000**		**0.000**
Total =				**14610.037**	**0.000**		**-61173.66**	**0.000**		**0.000**		**0.000**
					0.000			0.000				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2484.217			8346.971	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
3011.80	**12111.26**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.38
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	3726.326		12520.46
due to dynamic Earth pressure	1.5	0.000		0.00
		4429.253		**17535.839**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3726.326	12520.456	0.000	0.000
0.000	0.000	0.000	0.000
4295.01	**16577.99**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	15372.602	15372.602	KN
ML	1367.079	1367.079	kNm

Summary of Restoring Forces

Vertical Load	14610.037	kN
Moment	-61173.660	kNm

MT	-34.484	-34.484	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 = 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 Superstucture				1304.250		0.000	0.000		821.678	0.000				0.000	0.000
Forces from Substructure				14068.351	0.000	0.000	0.000		-11565.856	0.000		0.000		2700.430	0.000
CWLL-Max. Reaction case	0.20			186.55		0.000	0.000	0.630	117.527	0.000	1956.775		1.940	361.888	0.000
Vertical component of LL Surcharge	0.20			54.25				-3.500	-189.878				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**15613.403**	**0.000**	**0.000**	**0.000**		**-10816.529**	**0.000**		**0.000**		**3062.318**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstucture				1245.103	0.000	0.000	-3573.45	0.000
Forces from Substructure				13364.934	0.000	0.000	-57600.21	0.000
CWLL-Max. Reaction case	0.00			0.00	0.000	-2.870	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-7.000	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.00	
Total =				**14610.037**	**0.000**		**-61173.66**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2484.217			8346.971	
due to Live load surcharge	0.20	149.053			596.212	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
149.05	596.21						
3160.85	**12707.47**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Substructure	1.5	0.000		0.000
due to Active Earth pressure	1.5	3726.326		12520.456
due to dynamic Earth pressure	1.5	0.000		0
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		4429.253		**17535.839**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3726.326	12520.456		
0.000	0.000		
0.000	0.000		
0.000	0.000		
4295.01	**16577.99**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward

Summary of Restoring Forces

Vertical Load	14610.037	kN

P	15613.403	15613.403	KN
ML	1890.940	1890.940	kNm

Moment	-61173.660	kNm

MT	5797.232	5797.232	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 αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1304.250**		**0.000**	**0.000**		**821.678**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**491.163**	**0.000**	**0.000**	**0.000**		**-72.061**	**0.000**		**0.000**		**547.022**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1		12.86	321.56	0.000	0.000	0.000	-1.87	-600.690	0.000	1951.96	0.000	6.70	2153.408	0.000
Shaft above HFL	1	25	36.130	903.238	0.000	0.000	0.000	0.600	541.611	0.000	1952.780	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	29.667	445.000	0.000	0.000	0.000	0.530	235.952	0.000	1950.901	0.000	0.000	0.000	0.000
Back filling above HFL over heel sla	1	20	215.125	4302.509	0.000	0.000	0.000	-1.750	-7529.391	0.000	1953.363	0.00	0.000	0.000	0.00
Back filling below HFL over heel slal	1	10	142.282	1422.824	0.000	0.000	0.000	-1.820	-2589.221	0.000	1949.613	0.00	0.000	0.000	0.00
Front Filling over Pile Cap	1	10	84.263	842.635				2.190	1845.276				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	31.607	474.112				-1.436	-680.777				0.000	0.000	
Toe slab	1	15	22.577	338.652				2.026	685.987				0.000	0.000	
portion between heel & toe	1	15	13.893	208.401				0.500	104.201				0.000	0.000	
Vertical component of active earth pressure	1			813.849				-3.500	-2848.473				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				10315.262	**0.000**	**0.000**	**0.000**		**-10835.526**	**0.000**		**0.000**		**2153.408**	**0.000**
Total =				**12110.676**	**0.000**	**0.000**	**0.000**		**-10085.910**	**0.000**		**0.000**		**2700.430**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**1245.103**	**0.000**		**-3573.446**	**0.000**
Substructure & Foundation -Portion 1				**466.605**	**0.000**		**-1545.963**	**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	0.95	25	12.8625	305.484	0.000	-5.37	-1639.851	0.000
Shaft above HFL	0.95	25	36.130	858.076	0.000	-2.900	-2488.737	0.000
Shaft below HFL	0.95	15	29.667	422.750	0.000	-2.970	-1255.470	0.000
Back filling above HFL over heel sla	0.95	20	215.125	4087.384	0.000	-5.250	-21458.765	0.000
Back filling below HFL over heel slal	0.95	10	142.282	1351.682	0.000	-5.320	-7190.648	0.000
Front Filling over Pile Cap	0.95	10	84.263	800.503		-1.310	-1048.749	
Side filling between heel and toe	0.95	10	0.000	0.000		0.000	0.000	
Heel slab	0.95	15	31.607	450.407		-4.936	-2223.162	
Toe slab	0.95	15	22.577	321.719		-1.474	-474.329	
portion between heel & toe	0.95	15	13.893	197.981		-3.000	-593.943	
Vertical component of active earth pressure	0.95			773.157		-7.000	-5412.099	
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000	
				9799.499	**0.000**		**-44570.679**	**0.000**
Total =				**11511.207**	**0.000**		**-49690.09**	**0.000**
					0.000			0.000

For Overturning or Sliding Effect

Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
	0.000		**0.000**
1.0	0.000	1951.963	0.000
1.0	0.000	1952.780	0.000
1.0	0.000	1950.901	0.000
1.0	0.000	1953.363	0.000
1.0	0.000	1949.613	0.000
	0.000		**0.000**
	0.000		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2236.033			6620.022	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
2763.61	**10384.31**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Substructure	1.5	0.000		0.000
due to Active Earth pressure	1.5	3354.049		9930.032
due to dynamic Earth pressure	1.5	0.000		0.000
		4056.976		**14945.415**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3354.049	9930.032		
0.000	0.000		
3922.73	**13987.56**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	12110.676	12110.676	KN
ML	298.398	298.398	kNm
MT	-34.484	-34.484	kNm

Summary of Restoring Forces

Vertical Load	11511.207	kN
Moment	-49690.088	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				1304.250		0.000	0.000		821.678	0.000				0.000	0.000
Forces from Substructure				10806.425	0.000	0.000	0.000		-10907.587	0.000		0.000		2700.430	0.000
CWLL-Max. Reaction case	0.20			93.17		0.000	0.000	0.630	58.697	0.000	1956.775		2.352	219.161	0.000
Vertical component of LL Surcharge	0.20			54.25				-3.500	-189.878				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**12258.096**	**0.000**	**0.000**	**0.000**		**-10217.091**	**0.000**		**0.000**		**2919.591**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstucture				1245.103	0.000		-3573.446	0.000
Forces from Substructure				10266.104	0.000		-46116.642	0.000
CWLL-Max. Reaction case	0.00			0.00	0.00		-2.87	0.00
Vertical component of LL Surcharge	0.00			0.00		-2.870	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.000	0.000
Total =				**11511.207**	**0.000**		**-49692.96**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2236.033			6620.022	
due to Live load surcharge	0.20	149.053			596.212	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
149.05	596.21						
2912.67	**10980.52**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Substructure	1.5	0.000		0
due to Active Earth pressure	1.5	3354.049		9930.032
due to dynamic Earth pressure	1.5	0		0
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		4056.976		**14945.415**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3354.049	9930.032		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3922.73	**13987.56**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	12258.096	12258.096	KN
ML	763.429	763.429	kNm
MT	5654.505	5654.505	kNm

Summary of Restoring Forces

Vertical Load	11511.207	kN
Moment	-49692.958	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = 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				**1304.250**		**0.000**	**0.000**		**821.678**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**491.163**	**0.000**	**0.000**	**0.000**		**-72.061**	**0.000**		**0.000**		**547.022**	**0.000**
Substructure & Foundation -Portion 2				**13577.188**	**0.000**	**0.000**	**0.000**		**-11493.795**	**0.000**		**0.000**		**2153.408**	**0.000**
Total =				**15372.602**	**0.000**	**0.000**	**0.000**		**-10744.178**	**0.000**		**0.000**		**2700.430**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**1245.103**	**0.000**		**-3573.45**	**0.000**
Substructure & Foundation -Portion 1				**466.605**	**0.000**		**-1545.96**	**0.000**
Substructure & Foundation -Portion 2				**12898.328**	**0.000**		**-56054.25**	**0.000**
Total =				**14610.037**	**0.000**		**-61173.66**	**0.000**
					0.000			0.000

For Overturning or Sliding Effect

Load Factor	FL = 0.3 x αh x P (kN)	C.g. of Force (m)	MLs due to FL
	0.000		**0.000**
	0.000		**0.000**
	0.000		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2484.217			8346.971	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
3011.80	**12111.26**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.383
due to Substructure	1.5	0.000		0
due to Active Earth pressure	1.5	3726.326		12520.456
due to dynamic Earth pressure	1.5	0.000		0
		4429.253		**17535.839**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3726.326	12520.456		
0.000	0.000		
4295.01	**16577.99**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	15372.602	15372.602	KN
ML	1367.079	1367.079	kNm
MT	-34.484	-34.484	kNm

Summary of Restoring Forces

Vertical Load	14610.037	kN
Moment	-61173.660	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**15613.403**	**0.000**	**-10816.53**	**0.000**	**3062.318**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL2	MLs due to Fv
Total =	**14610.037**	**0.000**	**-61173.66**	**0.000**
		0.000		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0.000	1954.710	4652.914	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2484.217			8346.971	
due to Live load surcharge	0.2	149.053			596.212	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
149.05	596.21						
3160.85	**12707.47**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.93	1954.710	5015.383
due to Substructure	1.5	0.00		0.000
due to Active Earth pressure	1.5	3726.33		12520.456
due to dynamic Earth pressure	1.5	0.00		0.000
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		4429.25		**17535.839**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3726.326	12520.456		
0.000	0.000		
0.000	0.000		
0.000	0.000		
4295.01	**16577.99**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	15613.403	15613.403	KN
ML	1890.940	1890.940	kNm

Summary of Restoring Forces

Vertical Load	14610.037	kN
Moment	-61173.660	kNm

MT	5797.232	5797.232	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure	**1304.250**	**0.000**	**821.678**	**0.000**	**0.000**
Substructure & Foundation -Portion 1	**491.163**	**0.000**	**-72.061**	**0.000**	**547.022**
Substructure & Foundation -Portion 2	**10315.262**	**0.000**	**-10835.526**	**0.000**	**2153.408**
Total =	**12110.676**	**0.000**	**-10085.910**	**0.000**	**2700.430**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure	**1245.103**	**0.000**		**-3573.446**	**0.000**
Substructure & Foundatio	**466.605**	**0.000**		**-1545.963**	**0.000**
Substructure & Foundatio	**9799.499**	**0.000**		**-44570.679**	**0.000**
Total =	**11511.207**	**0.000**		**-49690.088**	**0.000**
		0.000			0.000

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.13	0.00	1954.71	4652.91	0.00
due to Substructure	1	0.00	0.00		0.00	0.00
due to Earth pressure	1	2236.03			6620.02	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
2763.61	**10384.31**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		702.927	1954.710	5015.38
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	3354.049		9930.03
due to dynamic Earth pressure	1.5	0.000		0.00
		4056.976		**14945.42**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
568.68	4057.53	413.17	2947.97
0.00	0.00	0.00	0.00
3354.049	9930.032		
0.000	0.000		
3922.73	**13987.56**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	12110.676	12110.676	KN
ML	298.398	298.398	kNm
MT	-34.484	-34.484	kNm

Summary of Restoring Forces

Vertical Load	11511.207	kN
Moment	-49690.088	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**12258.096**	**0.000**	**-10217.09**	**0.000**	**2919.591**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Total =	**11511.207**	**0.000**		**-49692.96**	**0.000**
		0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	652.125	0	1954.710	4652.914	0
due to Substructure	1	0.000	0		0.000	0
due to Earth pressure	1	2236.033			6620.022	
due to Live load surcharge	0.2	149.053			596.212	

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
527.58	3764.29	0.00	0.00	383.31	2734.91	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
149.05	596.21						
2912.67	**10980.52**	**0.00**	**0.00**	**383.31**	**2734.91**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		702.93	1954.710	5015.38	568.68	4057.53	413.17	2947.97
due to Substructure	1.5	0.00		0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.5	3354.05		9930.03	3354.049	9930.032		
due to dynamic Earth pressure	1.5	0.00		0.00	0.000	0.000		
due to Live load surcharge	0	0.00		0.00	0.000	0.000		
due to dynamic increment of live load surcharge	0	0.00		0.00	0.000	0.000		
		4056.98		**14945.42**	**3922.73**	**13987.56**	**413.17**	**2947.97**

Summary of Forces For SBC

	Downward	Upward	
P	12258.096	12258.096	KN
ML	763.429	763.429	kNm
MT	5654.505	5654.505	kNm

Summary of Restoring Forces

Vertical Load	11511.207	kN
Moment	-49692.958	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 (1956.775 - 1947.575) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.20 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1956.775 - 1947.575) = 0.000 kNm

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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= μP + c.A + Fp	FOS	Overturning moment	Restoring Moment = ΣP.eToe+Mp	FOS
	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
Normal Dry Case	kN	kNm	kNm	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	kN/m²	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	15372.602	1367.079	5435.344	175.093	199.190	116.943	141.041	199.190	116.943	4295.006	10227.026	2.38	16577.986	61329.27	3.70
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	16576.607	3986.383	7244.783	174.067	244.335	96.559	166.827	244.335	96.559	5189.324	10227.026	1.97	20155.259	63133.11	3.13
Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case	18208.809	5014.670	6204.830	176.224	264.617	109.842	198.235	264.617	109.842	5189.324	10227.026	1.97	20155.259	63133.11	3.13
								SAFE	**SAFE**			**SAFE**			**SAFE**

Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	12110.676	320.836	5435.344	150.774	156.429	92.624	98.279	156.429	92.624	3922.729	8057.845	2.05	13987.562	49845.70	3.56
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	12847.779	2645.992	6531.151	143.722	190.363	73.849	120.489	190.363	73.849	4817.047	8238.229	1.71	17564.836	51649.54	2.94
Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case	13236.977	2891.187	5691.858	141.073	192.036	80.179	131.142	192.036	80.179	4817.047	8238.229	1.71	17564.836	51649.54	2.94
								SAFE	**SAFE**			**SAFE**			**SAFE**
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	15372.602	1367.079	-34.484	145.834	169.931	146.203	170.300	170.300	145.834	4295.006	10227.026	2.38	16577.986	61173.66	3.69
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	15613.403	1890.940	5797.232	174.888	208.219	112.866	146.198	208.219	112.866	4295.006	10227.026	2.38	16577.986	61173.66	3.69
								SAFE	**SAFE**			**SAFE**			**SAFE**
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	12110.676	298.398	-34.484	121.712	126.972	122.081	127.341	127.341	121.712	3922.729	8057.845	2.05	13987.562	49690.09	3.55
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	12258.096	763.429	5654.505	149.561	163.018	89.067	102.523	163.018	89.067	3922.729	8057.845	2.05	13987.562	49692.96	3.55
								SAFE	**SAFE**			**SAFE**			**SAFE**

Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	15372.602	1367.079	-34.484	145.834	169.931	146.203	170.300	170.300	145.834	4295.006	10227.026	2.38	16577.986	61173.66	3.69
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	15613.403	1890.940	5797.232	174.888	208.219	112.866	146.198	208.219	112.866	4295.006	10227.026	2.38	16577.986	61173.66	3.69
								SAFE	**SAFE**			**SAFE**			**SAFE**
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	12110.676	298.398	-34.484	121.712	126.972	122.081	127.341	127.341	121.712	3922.729	8057.845	2.05	13987.562	49690.09	3.55
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	12258.096	763.429	5654.505	149.561	163.018	89.067	102.523	163.018	89.067	3922.729	8057.845	2.05	13987.562	49692.96	3.55
								SAFE	**SAFE**			**SAFE**			**SAFE**

DESIGN OF FOUNDATION

Foundation Lvl = 1947.575 m

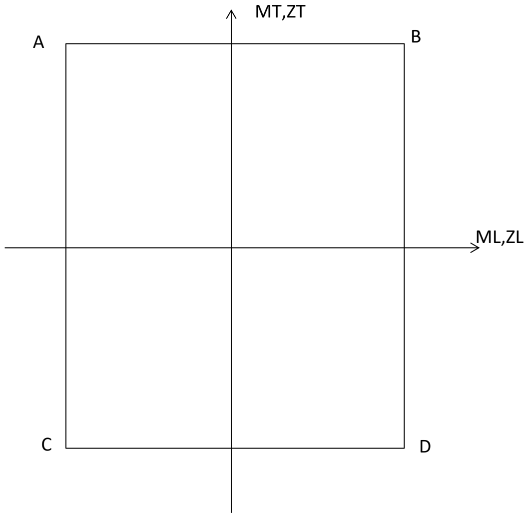
Properties of Footing Base:

A	=	97.254	m^2
ZL	=	113.463	m^3
ZT	=	186.942	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			1488.710	0.630	937.887	0.000	0.000
SIDL except Wearing Course	1.35			108.270	0.630	68.210	0.000	0.000
Wearing Course	1.75			212.279	0.630	133.736	0.000	0.000
				1809.259		**1139.833**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.501	84.402	0.290	24.477	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.588	19.835	0.290	5.752	0.000	0.000
Bracket - Uniform portion	1.35	25	1.250	42.201	-0.010	-0.422	0.000	0.000
Bracket - Tapered portion	1.35	25	0.625	21.101	0.040	0.844	0.000	0.000
Cap - (uniform portion)	1.35	25	3.001	101.283	0.500	50.641	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.500	0.000	0.000	0.000
Cantilever Return Wall-Rectangle p	1.35	25	1.200	40.505	-2.118	-85.801	6.697	271.250
Cantilever Return Wall-Traingle por	1.35	25	2.067	69.770	-1.452	-101.273	6.697	467.229
RCC Railing or Crash Barrier or Crash Barrier	1.35			37.800	0.290	10.962	0.000	0.000
Aproach Slab	1.35	25	7.294	246.174	-0.010	-2.462	0.000	0.000
				663.071		**-97.282**		**738.479**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	12.863	434.109	-1.868	-810.931	6.697	2907.101
Abutment Shaft	1.35	25	65.796	2220.621	0.530	1177.439	0.000	0.000
Back filling over heel slab	1.35	20	351.329	9485.895	-1.778	-16868.373	0.000	0.000
Front Filling over toe slab	1.35	20	84.263	2275.114	2.190	4982.244	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	31.607	1066.753	-1.436	-1531.748	0.000	0.000
Toe slab	1.35	25	22.577	761.966	2.026	1543.470	0.000	0.000
portion between heel & toe	1.35	25	13.893	468.902	0.500	234.451	0.000	0.000
Vertical Components of active earth pressure	1.5			1356.272	-3.500	-4746.951	0.000	0.000
				18464.831		**-15991.318**		**2907.101**
Total				**20937.160**		**-14948.767**		**3645.580**



Summary of Forces About C.G. OF Footing

P	20937.160	KN
ML	2793.515	kNm
MT	7439.459	kNm

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.630	1954.710	6454.532
due to Earth pressure	1.5	3726.326		12520.456

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	5221.83	531.73	3793.88
3726.33	12520.46	0.00	0.00
4458.187	**17742.282**	**531.728**	**3793.879**

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		20937.160		-14948.767		3645.580
CWLL-Max. Reaction case	1.5	1399.127	0.630	881.450	1.940	2714.159
Vertical Components of LL Surcharge	1.2	325.505	-3.500	-1139.268	0.000	0.000
Total		**22661.792**		**-15206.585**		**6359.739**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		904.630	1954.710	6454.532	731.86	5221.83	531.73	3793.88
due to Earth pressure	1.5	3726.326		12520.456	3726.33	12520.46		
due to Live load surcharge	1.2	894.318		3577.273	894.32	3577.27		
		5525.274		**22552.261**	**5352.505**	**21319.555**	**531.728**	**3793.879**

Summary of Forces About C.G. OF Footing

P	22661.792	KN
ML	6112.970	kNm
MT	10153.618	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				**1809.259**		**1139.833**		**0.000**
Substructure & Foundation -Portion 1				**663.071**		**-97.282**		**738.479**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	12.863	434.109	-1.868	-810.931	6.697	2907.101
Shaft above HFL	1.35	25	36.130	1219.371	0.600	731.174	0.000	0.000
Shaft below HFL	1.35	15	29.667	600.750	0.530	318.535	0.000	0.000
Back filling above HFL over heel slab	1.35	20	215.125	5808.388	-1.750	-10164.678	0.000	0.000
Back filling below HFL over heel slab	1.35	10	142.282	1920.812	-1.820	-3495.448	0.000	0.000
Front Filling over toe slab	1.35	10	84.263	1137.557	2.190	2491.122	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	31.607	640.052	-1.436	-919.049	0.000	0.000
Toe slab	1.35	15	22.577	457.180	2.026	926.082	0.000	0.000
Portion between Heel & Toe	1.35	15	13.893	281.341	0.500	140.671	0.000	0.000
Vertical Components of active earth pressure	1.5			1220.774	-3.500	-4272.710	0.000	0.000
				14047.681		**-15024.940**		**2907.101**
Total				**16520.010**		**-13982.389**		**3645.580**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		904.630	1954.710	6454.532	731.86	5221.83	531.73	3793.88
due to Earth pressure	1.5	3354.049		9930.032	3354.05	9930.03		
					4085.910	**15151.858**	**531.728**	**3793.879**

Summary of Forces About C.G. OF Footing

P	16520.010	KN
ML	1169.469	kNm
MT	7439.459	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		16520.010		-13982.389		3645.580
CWLL-Min. Reaction case	1.5	698.773	0.630	440.227	2.352	1643.711
Vertical Components of LL Surcharge	1.2	325.505	-3.500	-1139.268	0.000	0.000
Total		**17544.289**		**-14681.430**		**5289.291**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.630	1954.710	6454.532
due to Earth pressure	1.5	3354.049		9930.032
due to Live load surcharge	1.2	894.318		3577.273

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	5221.83	531.73	3793.88
3354.05	9930.03		
894.32	3577.27		
4980.228	**18729.131**	**531.728**	**3793.879**

Summary of Forces About C.G. OF Footing

P	17544.289	KN
ML	4047.701	kNm
MT	9083.170	kNm

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

Seismic Effect Factor =	1.50	αh=	0.000	In Longitudinal direction	Weight of shaft below Ground level	=	617.06 KN
		αh=	0.000	In Transverse direction	Weight of back fill below Ground level	=	2018.99 KN
		αv=	0.000	In Vertical direction			

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 = 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			1488.710		0.000	0.000	0.630	937.887	0.000	1955.243		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.270		0.000	0.000	0.630	68.210	0.000	1956.024		0.000	0.000	0.000
Wearing Course	1.75			212.279		0.000	0.000	0.630	133.736	0.000	1955.575		0.000	0.000	0.000
				1809.259		**0.000**	**0.000**		**1139.833**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.501	84.402	0.000	0.000	0.000	0.290	24.477	0.000	1955.275	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.588	19.835	0.000	0.000	0.000	0.290	5.752	0.000	1954.905	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.250	42.201				-0.010	-0.422						
Bracket - Tapered portion	1.35	25	0.625	21.101				0.040	0.844						
Cap - (uniform portion)	1.35	25	3.001	101.283	0.000	0.000	0.000	0.500	50.641	0.000	1954.559	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.500	0.000	0.000	1954.409	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle p	1.35	25	1.200	40.505	0.000	0.000	0.000	-2.118	-85.801	0.000	1955.275	0.000	6.697	271.250	0.000
Cantilever Return Wall-Traingle por	1.35	25	2.067	69.770	0.000	0.000	0.000	-1.452	-101.273	0.000	1954.286	0.000	6.697	467.229	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.290	10.962				0.000	0.000	
Aproach Slab	1.35	25	7.294	246.174				-0.010	-2.462				0.000	0.000	
				663.071	**0.000**	**0.000**	**0.000**		**-97.282**	**0.000**		**0.000**		**738.479**	**0.000**

Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	12.863	434.109	0.000	0.000	0.000	-1.868	-810.931	0.000	1951.963	0.000	6.697	2907.101	0.000
Abutment Shaft	1.35	25	65.796	2220.621	0.000	0.000	0.000	0.530	1177.439	0.000	1952.530	0.000	0.000	0.000	0.000
Back filling over heel slab	1.35	20	351.329	9485.895	0.000	0.000	0.000	-1.778	-16868.373	0.000	1951.963	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	84.263	2275.114				2.190	4982.244				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	31.607	1066.753				-1.436	-1531.748				0.000	0.000	
Toe slab	1.35	25	22.577	761.966				2.026	1543.470				0.000	0.000	
portion between heel & toe	1.35	25	13.893	468.902				0.500	234.451				0.000	0.000	
Vertical component of active earth pressure	1.00			904.181				-3.500	-3164.634						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				18012.740	**0.000**	**0.000**	**0.000**		**-14409.001**	**0.000**		**0.000**		**2907.101**	**0.000**
Total =				**20485.070**	**0.000**	**0.000**	**0.000**		**-13366.450**	**0.000**		**0.000**		**3645.580**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	2484.217			8346.971	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
0.00	0.00						
3216.08	**13568.80**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	20485.070	20485.070	KN
ML	202.347	202.347	kNm
MT	-148.298	-148.298	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				1809.259		0.000	0.000		1139.833	0.000				0.000	0.000
Forces from Substructure				18675.811	0.000	0.000	0.000		-14506.283	0.000		0.000		3645.580	0.000
CWLL-Max. Reaction case	0.75			699.56		0.000	0.000	0.630	440.725	0.000	1956.775		1.940	1357.080	0.000
Vertical component of LL Surcharge	0.20			54.251				-3.500	-189.878						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				**21238.884**	**0.000**	**0.000**	**0.000**		**-13115.603**	**0.000**		**0.000**		**5002.660**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1	2484.217			8346.971	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	149.053			596.212	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
0.00	0.00						
149.05	596.21						
0.00	0.00						
3365.13	**14165.01**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	21238.884	21238.884	KN
ML	1049.406	1049.406	kNm
MT	8796.538	8796.538	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1809.259**		**0.000**	**0.000**		**1139.833**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**663.071**	**0.000**	**0.000**	**0.000**		**-97.282**	**0.000**		**0.000**		**738.479**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	12.863	434.109	0.000	0.000	0.000	-1.868	-810.931	0.000	1951.963	0.000	6.697	2907.101	0.000
Shaft above HFL	1.35	25	36.130	1219.371	0.000	0.000	0.000	0.600	731.174	0.000	1952.780	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	15	29.6666504	600.750	0.000	0.000	0.000	0.530	318.535	0.000	1950.901	0.000	0.000	0.000	0.000
Back filling above HFL over heel slab	1.35	20	215.125469	5808.388	0.000	0.000	0.000	-1.750	-10164.678	0.000	1953.363	0.000	0.000	0.000	0.000
Back filling below HFL over heel slab	1.35	10	142.282351	1920.812	0.000	0.000	0.000	-1.820	-3495.448	0.000	1949.613	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	10	84.263	1137.557				2.190	2491.122				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	31.607	640.052				-1.436	-919.049				0.000	0.000	
Toe slab	1.35	15	22.577	457.180				2.026	926.082				0.000	0.000	
portion between heel & toe	1.35	15	13.893	281.341				0.500	140.671				0.000	0.000	
Vertical component of active earth pressure	1.00			813.849				-3.500	-2848.473						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				13640.756	**0.000**	**0.000**	**0.000**		**-13600.704**	**0.000**		**0.000**		**2907.101**	**0.000**
Total =				**16113.086**	**0.000**	**0.000**	**0.000**		**-12558.153**	**0.000**		**0.000**		**3645.580**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	2236.033			6620.022	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
0.00	0.00						
2967.89	**11841.85**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	16113.086	16113.086	KN
ML	-716.305	-716.305	kNm
MT	-148.298	-148.298	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 = 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				1809.259		0.000	0.000		1139.833	0.000				0.000	0.000
Forces from Substructure				14303.827	0.000	0.000	0.000		-13697.986	0.000		0.000		3645.580	0.000
CWLL-Min. Reaction case	0.75			349.39		0.000	0.000	0.630	220.114	0.000	1956.775		2.352	821.856	0.000
Vertical component of LL Surcharge	0.20			54.251				-3.500	-189.878						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				**16516.723**	**0.000**	**0.000**	**0.000**		**-12527.917**	**0.000**		**0.000**		**4467.436**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1	2236.033			6620.022	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	149.053			596.212	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
0.00	0.00						
149.05	596.21						
0.00	0.00						
3116.95	**12438.06**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	16516.723	16516.723	KN
ML	-89.858	-89.858	kNm
MT	8261.314	8261.314	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 αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1809.259**		**0.000**	**0.000**		**1139.833**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**663.071**	**0.000**	**0.000**	**0.000**		**-97.282**	**0.000**		**0.000**		**738.479**	**0.000**
Substructure & Foundation -Portion 2				**18012.740**	**0.000**	**0.000**	**0.000**		**-14409.001**	**0.000**		**0.000**		**2907.101**	**0.000**
Total =				**20485.070**	**0.000**	**0.000**	**0.000**		**-13366.450**	**0.000**		**0.000**		**3645.580**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure		2484.217			8346.971	
due to dynamic increment of EP		0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
0.00	0.00						
3216.08	**13568.80**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	20485.070	20485.070	KN
ML	202.347	202.347	kNm
MT	-148.298	-148.298	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**21238.884**	**0.000**	**-13115.603**	**0.000**	**5002.660**
					0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0
due to Substructure		0.000	0.000		0.000	0
due to Earth pressure		2484.217			8346.971	
due to dynamic increment of EP		0.000			0.000	
due to Live load surcharge		149.053			596.212	
due to dynamic increment of Surcharge		0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2484.22	8346.97						
0.00	0.00						
149.05	596.21						
0.00	0.00						
3365.13	**14165.01**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	21238.884	21238.884	KN
ML	1049.406	1049.406	kNm
MT	8796.538	8796.538	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure				**1809.259**	**0.000**	**1139.833**	**0.000**	**0.000**
Substructure & Foundation -Portion 1				**663.071**	**0.000**	**-97.282**	**0.000**	**738.479**
Substructure & Foundation -Portion 2				**13640.756**	**0.000**	**-13600.704**	**0.000**	**2907.101**
Total =				**16113.086**	**0.000**	**-12558.153**	**0.000**	**3645.580**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	2236.033			6620.022	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
0.00	0.00						
2967.89	**11841.85**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	16113.086	16113.086	KN
ML	-716.305	-716.305	kNm
MT	-148.298	-148.298	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**16516.723**	**0.000**	**-12527.917**	**0.000**	**4467.436**
		0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	6454.532	0
due to Substructure		0.000	0.000		0.000	0
due to Earth pressure		2236.033			6620.022	
due to dynamic increment of EP		0.000			0.000	
due to Live load surcharge		149.053			596.212	
due to dynamic increment of Surcharge		0.000			0.000	

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
731.86	5221.83	0.00	0.00	531.73	3793.88	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2236.03	6620.02						
0.00	0.00						
149.05	596.21						
0.00	0.00						
3116.95	**12438.06**	**0.00**	**0.00**	**531.73**	**3793.88**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	16516.723	16516.723	KN
ML	-89.858	-89.858	kNm
MT	8261.314	8261.314	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 (	1956.775 -	1947.575)	=	0.000	kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.75	x	0.00		=	0.000	KN
Transverse Moment due to C.F.	=	0.000	x (	1956.775 -	1947.575)	=	0.000	kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

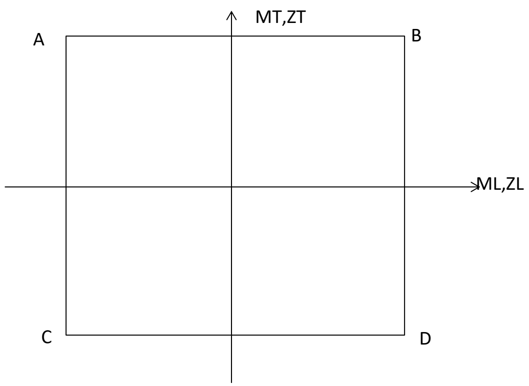
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

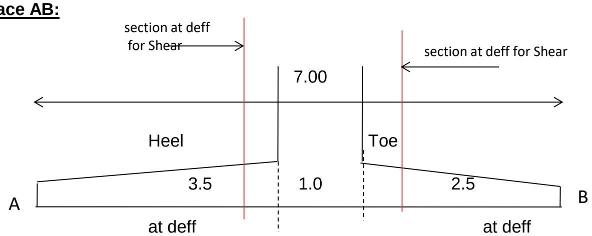
Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

Summary of Design Base Pressure

LOAD CASES	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	20937.160	2793.515	7439.459	230.459	279.700	150.868	200.109
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	22661.792	6112.970	10153.618	233.455	341.208	124.826	232.579
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	16520.010	1169.469	7439.459	199.353	219.967	119.762	140.376
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	17544.289	4047.701	9083.170	193.311	264.659	96.135	167.483
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	20485.070	202.347	-148.298	208.058	211.625	209.645	213.212
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	21238.884	1049.406	8796.538	256.192	274.690	162.082	180.580
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	16113.086	-716.305	-148.298	171.201	158.574	172.787	160.161
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	16516.723	-89.858	8261.314	214.815	213.231	126.431	124.847
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	20485.070	202.347	-148.298	208.058	211.625	209.645	213.212
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	21238.884	1049.406	8796.538	256.192	274.690	162.082	180.580
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	16113.086	-716.305	-148.298	171.201	158.574	172.787	160.161
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	16516.723	-89.858	8261.314	214.815	213.231	126.431	124.847



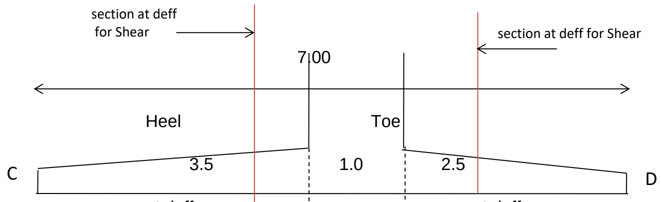
Pressure calculation along Face AB:



		at deff			at deff	
Case : 1	230.459	248.629	255.08	262.114	268.564	279.700
Case : 2	233.455	273.216	287.33	302.724	316.840	341.208
Case : 3	199.353	206.960	209.66	212.605	215.306	219.967
Case : 4	193.311	219.638	228.99	239.178	248.524	264.659
Case : 5	208.058	209.375	209.84	210.351	210.819	211.625
Case : 6	256.192	263.018	265.44	268.083	270.507	274.690
Case : 7	171.201	166.541	164.89	163.084	161.430	158.574
Case : 8	214.815	214.230	214.02	213.797	213.589	213.231
Case : 9	208.058	209.375	209.84	210.351	210.819	211.625
Case : 10	256.192	263.018	265.44	268.083	270.507	274.690
Case : 11	171.201	166.541	164.89	163.084	161.430	158.574
Case : 12	214.815	214.230	214.02	213.797	213.589	213.231

Average MAX Base Pressure for Design of Heel Slab-along Face AB	=	260.816	kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face AB	=	168.044	kN/m^2
Average MAX Base Pressure for Design of Toe Slab-along Face AB	=	321.966	kN/m^2
Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	316.840	kN/m^2
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	273.216	kN/m^2

Pressure calculation along Face CD:



		at deff			at deff	
Case : 1	150.868	169.038	175.49	182.523	188.973	200.109
Case : 2	124.826	164.587	178.70	194.096	208.212	232.579
Case : 3	119.762	127.369	130.07	133.014	135.715	140.376
Case : 4	96.135	122.462	131.81	142.001	151.348	167.483
Case : 5	209.645	210.961	211.43	211.938	212.405	213.212
Case : 6	162.082	168.908	171.33	173.974	176.397	180.580
Case : 7	172.787	168.128	166.47	164.670	163.016	160.161
Case : 8	126.431	125.847	125.64	125.413	125.206	124.847
Case : 9	209.645	210.961	211.43	211.938	212.405	213.212
Case : 10	162.082	168.908	171.33	173.974	176.397	180.580
Case : 11	172.787	168.128	166.47	164.670	163.016	160.161
Case : 12	126.431	125.847	125.64	125.413	125.206	124.847

Average MAX Base Pressure for Design of Heel Slab-along Face CD	=	210.537	kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face CD	=	113.972	kN/m^2
Average Base Pressure for Design of Toe Slab-along Face CD	=	213.338	kN/m^2
Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	212.405	kN/m^2
Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	210.961	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	=	260.816	kN/m^2							
Upward moment due to Base pressure	=	1597.501	kNm/m							
		factor								
Downward moment due to backfill	=	1.35	x	351.329	/	13.893	x	20	x	1.750
	=	1194.834	kNm/m							
Downward moment due to self weight of Heel slab	=	1.35	x	31.607	/	13.893	x	25	x	1.436
	=	110.250	kNm/m							

Net Moment at face of shaft	=	1597.501	-1194.834	-110.250 =	292.417	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	=	113.972	kN/m^2							
Upward moment due to Base pressure	=	698.076	kNm/m							
		factor								
Downward moment due to backfill	=	1.35	x	351.329	/	13.893	x	10	x	1.750
	=	597.417	kNm/m							
Downward moment due to self weight of Heel slab	=	1.35	x	31.607	/	13.893	x	15	x	1.436
	=	66.150	kNm/m							

Net Moment at face of shaft	=	698.076	-597.417	-66.150 =	34.509	kNm/m	**Tension at Bottom of Heel Slab**

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	0.817	m					
effective depth at critical section	=	0.734	m					
Base pressure at deff from face of wall	=	273.216	kN/m^2					
Shear Force due to upward pressure at deff from face of wall	=	264.704	x	2.583 x	13.893	=	9499.336	KN
		factor						
Downward Force due to backfill	=	1.35	x	351.329 x	20	=	9485.895	KN
Downward Force due to self weight of Heel slab	=	1.35	x	31.607 x	25	=	1066.753	KN
Net Shear Force	=	9499.336		-9485.895	-1066.753	=	-1053.312	KN
Net Shear Force / unit meter	=	-1053.312	/	13.893		=	75.814	KN/m

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	321.966	kN/m^2								
Upward moment due to Base pressure	=	1006.144	kNm/m								
Downward moment due to self weight of Toe slab	=	1.35	x	22.577	/	13.893	x	25	x	1.026	
	=	56.2500	kNm/m								
Net Moment at face of shaft	=	1006.144		-56.250 =		949.894	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	=	0.658	m							
Base pressure at deff from face of wall	=	316.840	kN/m^2							
upward shear force due to base pressure	=	329.024	x	1.583 x		13.893		=	7236.306	KN
C.g. Of base pressure	=	0.679	m							
moment due to upward pressure at critical section	=	4916.386	kNm							
$\tan\beta$	=	0.280								
reduction in shear force (V_{ccd})	=	$\frac{M \tan\beta}{d}$	=	1852.145	KN					
Downward force due to self weight of toe slab	=	1.35	x	0.522	x	1.583	x	13.893	x 25	
	=	387.184	KN							
Net Shear Force at deff	=	7236.306	-	387.184	-	1852.145		=	4996.977	KN
Net Shear Force / unit meter	=	4996.977	/	13.893				=	359.665	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	
fctm	=	2.77	N/mm^2	
Grade of Reinforcement Steel	=	Fe	500D	(HYSD Bars)
fy or fyk	=	500.00	N/mm^2	
fyd	=	434.78	N/mm^2	(fy/1.15)
Es	=	200000.00	N/mm^2	

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	**Toe Slab**
Ultimate bending moment,Mu (kNm/m)	**=**	**292.42**	**949.89**
Effective depth required (dreq) (mm)	=	223.67	403.13
Effective depth provided (dpro) (mm)	=	917.00	917.00
Check for provided depth	**=**	**SAFE**	**SAFE**
R = Mu/(b d2)	**=**	**0.35**	**1.13**

Total depth provided (mm)	=	1000.00	1000.00
Limiting depth of neutral axis (mm)	=	565.66	565.66
Actual depth of neutral axis (mm)	=	54.20	108.41
Check for Neutral axis depth	**=**	**OK**	**OK**
Lever arm (z) , mm	=	894.45	871.90
Moment of Resistance w.r.to steel	=	610.87	1190.94
Check for Moment Capacity	**=**	**SAFE**	**SAFE**
Ast reqd (mm2 / m)	=	808.82	2701.26
cl. 16.6.1 (2) of IRC :112-2011			
$A_{S.min} = 0.26\ f_{ctm}\ b_t\ d\ /\ f_{yk} >= 0.0013\ b_t\ d$	=	1321.45	1321.45
Governing Ast (mm2 / m)	=	1321.45	2701.26
Tension Reinforcement			
Dia (mm)	=	**16.00**	**20.00**
Spacing (mm)	=	**200.00**	**200.00**
+ Dia (mm)	=	**12.00**	**20.00**
Spacing (mm)	=	**200.00**	**200.00**
Ast provided (mm2 / m)	=	1570.80	3141.59
Check for Ast provided	**=**	**OK**	**OK**
As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least			
Secondary Reinforcement (mm2/m)	=	314.16	628.32
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	150.00
Ast provided (mm2 /m)	=	565.49	753.98
Check for Ast provided	**=**	**OK**	**OK**

Shear Reinforcement Calculation:

		Along Traffic Direction		
		Heel Slab	Toe Slab	
Ultimate Shear Force (V_{Ed})	=	**75.814**	**359.665**	kN/m
Ast provided	=	1570.796	3141.59	mm2/m
Depth of slab at critical section	=	**816.600**	**743.240**	mm
Effective depth at critical section	=	733.600	658.240	mm
Percentage of steel provided ($\rho 1$)	=	0.0019	0.0042	
cl. 10.3.1 of IRC :112-2011				
$\rho 1 = A_{sl}/(b_w\ d) <= 0.02$	=	OK	OK	
Actual shear stress=v_{ED} = (V_{Ed}/b*0.9d)	**=**	**0.115**	**0.607**	N/mm^2
Max shear capacity, 0.135 fck(1-fck/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) <= 2.0	=	1.522	1.551	
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010				
$v_{min} = 0.031\ K^{3/2}\ fck^{1/2}$	=	0.344	0.354	N/mm^2
$0.12\ K\ (80\ \rho 1\ f_{ck})^{0.33}$	=	0.318	0.421	N/mm^2
$\sigma_{cp} = N_{Ed}\ /\ A_c <= 0.2\ f_{cd}$	=	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	=	315.825	385.809	kN

Check for Shear Reinforcement		**OK, No shear reinf. Req.**	**OK, No shear reinf. Req.**		
Balance Shear Force= $V_{Rd,s} = V_{Ed} - V_{Rd,c}$	=	0.000	0.000	KN/m	
b	=	13.893	13.893	m	
Total Shear Force	=	0.000	0.000	kN	
θ = 0.5 x sin^-1 [vEd / (0.18 fck (1- fck/250))]		0.607	3.217		
cot θ = (< 1 cotθ <2.5)	=	2.500	2.500		
fywd = 0.8 x fy/1.15	=	347.826	347.826	N/mm^2	
Provide Shear Reinforcement					
Legged	=	**0**	**0**		
Dia	=	**0**	**0**	mm	
Area of Shear Reinf, Asw	=	0.000	0.000	mm^2	
z = 0.9 * d	=	660.240	592.416	mm	
Spacing of shear Reinforcement required					
S = Asw*z* fywd* cotθ / V_{Rd}	=	0.000	0.000	mm	
As per Clause 10.3.3.5 of IRC:112-2011					
Asw/ (b S) = $\rho_{w,min}$ = (0.072 $f_{ck}^{0.5}$) / f_{yk}	=	0.001	0.001		
Spacing of shear Reinforcement required	=	0.000	0.000	mm	
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011					
S_{max} = 0.75 d	=	550.200	493.680	mm	
Governing Spacing of Shear Reinf.	=	**0.000**	**0.000**	mm	
Provided Spacing of Shear Reinf.	=	**200**	**150**	mm	

SLS CHECK OF FOUNDATION

Foundation Lvl = 1947.575 m

Properties of Footing Base:

A	=	97.254	m^2
ZL	=	113.463	m^3
ZT	=	186.942	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			1102.748	0.630	694.731	0.000	0.000
SIDL except Wearing Course	1.00			80.200	0.630	50.526	0.000	0.000
Wearing Course	1.20			145.563	0.630	91.705	0.000	0.000
				1328.511		**836.962**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	2.501	62.520	0.290	18.131	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.588	14.692	0.290	4.261	0.000	0.000
Bracket - Uniform portion	1.00	25	1.250	31.260	-0.010	-0.313	0.000	0.000
Bracket - Tapered portion	1.00	25	0.625	15.630	0.040	0.625	0.000	0.000
Cap - (uniform portion)	1.00	25	3.001	75.024	0.500	37.512	0.000	0.000
Cap - (corbel portion)	1.00	25	0.000	0.000	0.500	0.000	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	1.200	30.004	-2.118	-63.556	6.697	200.926
Cantilever Return Wall-Traingle portion	1.00	25	2.067	51.681	-1.452	-75.017	6.697	346.095
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.290	8.120	0.000	0.000
Aproach Slab	1.00	25	7.294	182.351	-0.010	-1.824	0.000	0.000
				491.163		**-72.061**		**547.022**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	12.863	321.563	-1.868	-600.690	6.697	2153.408
Abutment Shaft	1.00	25	65.796	1644.904	0.530	872.177	0.000	0.000
Back filling over heel slab	1.00	20	351.329	7026.589	-1.778	-12495.091	0.000	0.000
Front Filling over toe slab	1.00	20	84.263	1685.270	2.190	3690.551	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	31.607	790.187	-1.436	-1134.628	0.000	0.000
Toe slab	1.00	25	22.577	564.420	2.026	1143.311	0.000	0.000
portion between heel & toe	1.00	25	13.893	347.335	0.500	173.668	0.000	0.000

Vertical Components of active earth pressure	1.00			904.181	-3.500	-3164.634	0.000	0.000
				13577.188		**-11493.795**		**2153.408**
Total				**15396.862**		**-10728.894**		**2700.430**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		664.255	1954.710	4739.463
due to Earth pressure	1.00	2484.217		8346.971

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
537.39	3834.31	390.44	2785.79
2484.22	8346.97		
3021.611	**12181.277**	**390.440**	**2785.786**

Summary of Forces

P	15396.862	KN
ML	1452.383	kNm
MT	5486.216	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		15396.862		-10728.894		2700.430
CWLL-Max. Reaction case	1.00	932.751	0.630	587.633	1.940	1809.439
Vertical Components of LL Surcharge	0.80	217.003	-3.500	-759.512	0.000	0.000
Total		**16546.617**		**-10900.773**		**4509.869**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		664.255	1954.710	4739.463
due to Earth pressure	1.00	2484.217		8346.971
due to Live load surcharge	0.80	596.212		2384.849

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
537.39	3834.31	390.44	2785.79
2484.22	8346.97		
596.21	2384.85		
3617.824	**14566.125**	**390.440**	**2785.786**

Summary of Forces

P	16546.617	KN
ML	3665.353	kNm
MT	7295.656	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				**1328.511**		**836.962**		**0.000**
Substructure & Foundation -Portion 1				**491.163**		**-72.061**		**547.022**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	12.863	321.563	-1.868	-600.690	6.697	2153.408
Shaft above HFL	1.00	25	36.130	903.238	0.600	541.611	0.000	0.000
Shaft below HFL	1.00	15	29.667	445.000	0.530	235.952	0.000	0.000
Back filling above HFL over heel slab	1.00	20	215.125	4302.509	-1.750	-7529.391	0.000	0.000
Back filling below HFL over heel slab	1.00	10	142.282	1422.824	-1.820	-2589.221	0.000	0.000
Front Filling over toe slab	1.00	10	84.263	842.635	2.190	1845.276	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	31.607	474.112	-1.436	-680.777	0.000	0.000
Toe slab	1.00	15	22.577	338.652	2.026	685.987	0.000	0.000
Portion between Heel & Toe	1.00	15	13.893	208.401	0.500	104.201	0.000	0.000
Vertical Components of active earth pressure	1.00			813.849	-3.500	-2848.473	0.000	0.000
				10315.262		**-10813.088**		**2153.408**
Total				**12134.936**		**-10048.187**		**2700.430**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		664.255	1954.710	4739.463
due to Earth pressure	1.00	2236.033		6620.022

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
537.39	3834.31	390.44	2785.79
2236.03	6620.02		
2773.427	**10454.327**	**390.440**	**2785.786**

Summary of Forces

P	12134.936	KN
ML	406.140	kNm
MT	5486.216	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		12134.936		-10048.187		2700.430
CWLL-Min. Reaction case	1.00	465.849	0.630	293.485	2.352	1095.807
Vertical Components of LL Surcharge	0.80	217.003	-3.500	-759.512	0.000	0.000
Total		**12817.788**		**-10514.215**		**3796.237**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		664.255	1954.710	4739.463
due to Earth pressure	1.00	2236.033		6620.022
due to Live load surcharge	0.80	596.212		2384.849

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
537.39	3834.31	390.44	2785.79
2236.03	6620.02		
596.21	2384.85		
3369.639	**12839.176**	**390.440**	**2785.786**

Summary of Forces

P	12817.788	KN
ML	2324.961	kNm
MT	6582.023	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 (1956.775 - 1947.575)

= 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

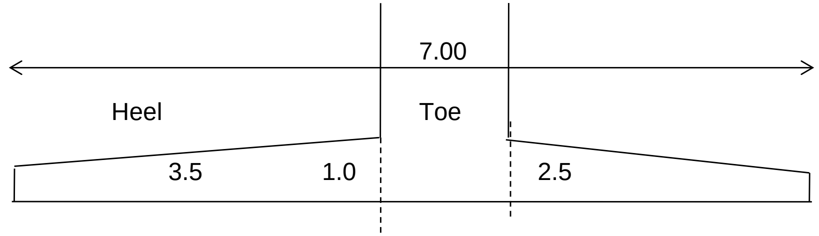
Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

	Design Base Pressure						
LOAD CASES	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	15396.862	1452.383	5486.216	174.863	200.464	116.169	141.770

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	16546.617	3665.353	7295.656	176.860	241.469	98.808	163.417
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	12134.936	406.140	5486.216	150.544	157.703	91.849	99.008
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	12817.788	2324.961	6582.023	146.515	187.497	76.097	117.079

<u>Pressure calculation along Face AB:</u>



Case 1 :	174.863	187.66	191.321	200.464
Case 2:	176.860	209.16	218.395	241.469
Case 4:	150.544	154.12	155.146	157.703
Case 5:	146.515	167.01	172.861	187.497

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 193.012 kN/m^2

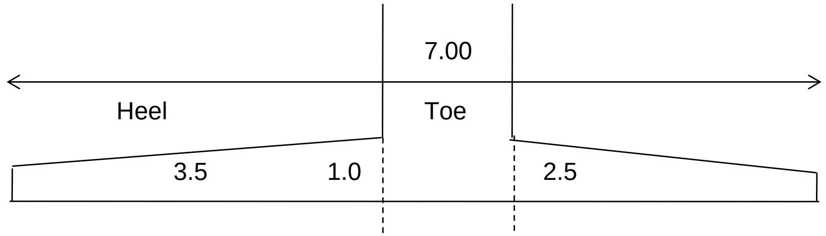
Average Base Pressure for Design of Toe Slab-along Face AB = 229.932 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 181.263 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face AB = 195.892 kN/m^2

<u>Pressure calculation along Face CD:</u>



Case 1 :	116.169	128.97	132.626	141.770
Case 2:	98.808	131.11	140.342	163.417
Case 4:	91.849	95.43	96.452	99.008
Case 5:	76.097	96.59	102.443	117.079

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 122.569 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 151.879 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 122.569 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 137.198 kN/m^2

Moment Calculation

	Rare Combination		Quasi-Parmenant		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	193.01	229.93	181.26	195.89	kN/m^2
Upward moment due to Base pressure	1182.20	718.54	1110.24	612.16	kNm/m
Downward moment due to backfill	885.06	0.00	885.06	0.00	kNm/m
Downward moment due to self weight of slab	81.67	41.67	81.67	41.67	kNm/m
Net Moment	**215.47**	**676.87**	**143.51**	**570.50**	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 = 1.2

Ecm = 32308.25 N/mm^2

Es = 200000.00 N/mm^2

Eceff = $\frac{Ecm}{(1 + \phi)}$ = 1.47E+04

Modular Ratio (m) = Es/ Eceff = 13.62

		Rare Combination		Quasi Permanent Comb.		
		Heel Slab	**Toe Slab**	**Heel Slab**	**Toe Slab**	
Working bending moment, M	**=**	**215.47**	**676.87**	**143.51**	**570.50**	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)	=	1.293	4.061			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	20.000			mm
Effective Depth deff (dy)	=	917.000	917.000			mm
Ast provided	=	1570.796	3141.593			mm^2/m
Percentage of steel , pt	=	0.0017	0.0034			
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m	=	0.194	0.262			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	177.834	240.584			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	739.166	676.416			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	656.166	591.416			mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2				
Icr	=	1.356E+10	2.422E+10			mm^4
Maximum compressive stress in concrete	=	**2.825**	**6.724**	**1.882**	**5.668**	**< 16.8, SAFE**
Maximum Tensile stress in steel	=	**141.970**	**225.118**	**94.554**	**189.740**	**< 300, SAFE**

Check For Crack Width in Quasi-Permanent Load Combination

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)	=	415.000	425.000	mm
Provided Spacing	=	100.000	100.000	mm
Check for Applicability of Formula	=	OK	OK	
Maximum crack spacing , $S_{r\ max}$	=	3.4 c +	$\frac{0.425\ k1\ k2\ \varphi}{\rho_{\rho\ eff}}$	
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , yc	=	177.834	240.584	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.		

hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	207.500	207.500	mm
Ac, eff = Dx * hc,eff	=	207500.000	207500.000	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	0.008	0.015	
Maximum crack spacing , $S_{r\ max}$	=	614.308	479.568	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\dfrac{\sigma_{sc} - k_t f_{ct\ eff} (1 + \alpha_e \rho_{\rho\ eff})}{\rho_{\rho\ eff}}$	/ Es	
tensile stress in steel, ,σsc	=	94.554	189.740	N/mm^2
Kt	=	0.500	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	2.771	N/mm^2
αe = Es/Ecm	=	6.190	6.190	
(εsm - εcm)	=	0.00028	0.0006	
Crack width , Wk=Sr max (εsm - εcm)	**=**	**0.174**	**0.273**	mm
Check	**=**	**SAFE**	**SAFE**	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom lvl = 1948.575 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			1488.710	0.130	193.532	0.000	0.000
SIDL except Wearing Course	1.35			108.270	0.130	14.075	0.000	0.000
Wearing Course	1.75			212.279	0.130	27.596	0.000	0.000
				1809.259		**235.204**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.501	84.402	-0.210	-17.725	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.588	19.835	-0.210	-4.165	0.000	0.000
Bracket - Uniform portion	1.35	25	1.250	42.201	-0.510	-21.523	0.000	0.000
Bracket - Tapered portion	1.35	25	0.625	21.101	-0.460	-9.706	0.000	0.000
Cap - (uniform portion)	1.35	25	3.001	101.283	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.210	-7.938	0.000	0.000
Aproach Slab	1.35	25	7.294	246.174	-0.510	-125.549	0.000	0.000
				552.795		**-186.605**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1.35	25	65.796	2220.621	0.100	221.245	0.000	0.000
Total				**4582.675**		**269.844**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.63	1954.710	5549.90
due to Earth pressure	1.5	2852.97		8387.73

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	4489.97	531.73	3262.15
2852.97	8387.73	0.00	0.00
3584.83	**12877.69**	**531.728**	**3262.151**

Summary of Forces

P	4582.68	KN
ML	13147.54	kNm
MT	3262.15	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		4582.675		269.844		0.000
CWLL-Max. Reaction case	1.5	1399.127	0.130	181.886	1.940	2714.159
Total		**5981.802**		**451.730**		**2714.159**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.630	1954.710	5549.902
due to Earth pressure	1.5	2852.968		8387.727
due to Live load surcharge	1.2	782.528		2738.850

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	4489.97	531.73	3262.15
2852.97	8387.73		
782.53	2738.85		

4367.358	15616.542	531.728	3262.151

Summary of Forces

P	5981.802	KN
ML	16068.272	kNm
MT	5976.310	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				1809.259		235.204		0.000
Substructure-Portion 1				552.795		-186.605		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	36.130	1219.371	0.100	164.010	0.000	0.000
Shaft below HFL	1.35	23.500	29.667	941.174	0.030	38.409	0.000	0.000
Total				**4522.601**		**251.017**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.630	1954.710	5549.902
due to Earth pressure	1.5	2659.788		6927.965

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	4489.97	531.73	3262.15
2659.79	6927.97	0.00	0.00
3391.649	**11417.931**	**531.728**	**3262.151**

Summary of Forces

P	4522.601	KN
ML	11668.948	kNm
MT	3262.151	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		4522.601		251.017		0.000
CWLL-Max. Reaction case	1.5	698.773	0.130	90.841	2.352	1643.711
Total		**5221.374**		**341.858**		**1643.711**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		904.630	1954.710	5549.902
due to Earth pressure	1.5	2659.788		6927.965
due to Live load surcharge	1.2	782.528		2738.850

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
731.86	4489.97	531.73	3262.15
2659.79	6927.97		
782.53	2738.85		
4174.177	**14156.780**	**531.728**	**3262.151**

Summary of Forces

P	5221.374	KN
ML	14498.638	kNm
MT	4905.862	kNm

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

Seismic Effect Factor = 1.50

αh= 0.000 In Longitudinal direction
αh= 0.000 In Transverse direction
αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 617.1 KN

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 = 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			1488.710		0.000	0.000	0.130	193.532	0.000	1955.243		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.270		0.000	0.000	0.130	14.075	0.000	1956.024		0.000	0.000	0.000
Wearing Course	1.75			212.279		0.000	0.000	0.130	27.596	0.000	1955.575		0.000	0.000	0.000
				1809.259		**0.000**	**0.000**		**235.204**	**0.000**				**0.000**	**0.000**
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.501	84.402	0.000	0.000	0.000	-0.210	-17.725	0.000	1955.275	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.588	19.835	0.000	0.000	0.000	-0.210	-4.165	0.000	1954.905	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.250	42.201				-0.510	-21.523						
Bracket - Tapered portion	1.35	25	0.625	21.101				-0.460	-9.706						
Cap - (uniform portion)	1.35	25	3.001	101.283	0.000	0.000	0.000	0.000	0.000	0.000	1954.559	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1954.409	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800				-0.210	-7.938				0.000		
Aproach Slab	1.35	25	7.294	246.174				-0.510	-125.549				0.000		
				552.795	**0.000**	**0.000**	**0.000**		**-186.605**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure-Portion 2															
Abutment Shaft	1.35	25	65.796	**2220.621**	**0.000**	**0.000**	**0.000**	**0.100**	**221.245**	**0.000**	**1952.530**	**0.000**	**0.000**	**0.000**	**0.000**
Total =				**4582.675**	**0.000**	**0.000**	**0.000**		**269.844**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	5549.902	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1901.979			5591.818	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1901.98	5591.82						
0.00	0.00						
2633.84	**10081.78**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4582.675	4582.675	KN
ML	10351.627	10351.627	kNm
MT	-3262.151	-3262.151	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				1809.259		0.000	0.000		235.204	0.000				0.000	0.000
Forces from Substructure				2773.416	0.000	0.000	0.000		34.640	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			186.55		0.000	0.000	0.130	24.252	0.000	1956.775		1.940	361.888	0.000
Total =				**4769.226**	**0.000**	**0.000**	**0.000**		**294.095**	**0.000**		**0.000**		**361.888**	**0.000**

0.000 0.000

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	5549.902	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1901.979			5591.818	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	130.421			456.475	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1901.98	5591.82						
0.00	0.00						
130.42	456.47						
0.00	0.00						
2764.26	**10538.26**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4769.226	4769.226	KN
ML	10832.353	10832.353	kNm
MT	3624.039	3624.039	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				1809.259		0.000	0.000		235.204	0.000				0.000	0.000
Substructure-Portion 1				552.795	0.000	0.000	0.000		-186.605	0.000		0.000		0.000	0.000
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	36.130	1219.371	0.000	0.000	0.000	0.100	121.489	0.000	1952.780	0.000	0.000	0.000	0.000
Shaft below HFL	1.350	23.500	29.667	941.174	0.000	0.000	0.000	0.030	28.451	0.000	1950.901	0.000	0.000	0.000	0.000
				2160.546	**0.000**	**0.000**	**0.000**		**149.940**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**4522.601**			**0.000**		**198.538**	**0.000**				**0.000**	
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	5549.902	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1773.192			4618.644	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1773.19	4618.64						
0.00	0.00						
2505.05	**9108.61**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Downward	Upward	
P	4522.601	4522.60	KN
ML	9307.147	9307.15	kNm

MT	3262.151	3262.15	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 = 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				1809.259		0.000	0.000		235.204	0.000				0.000	0.000
Forces from Substructure				2713.341	0.000	0.000	0.000		-36.665	0.000		0.000		0.000	0.000
CWLL-Min. Reaction case	0.20			93.17		0.000	0.000	0.130	12.112	0.000	1956.775		2.352	219.161	0.000
Total =				**4615.770**	**0.000**	**0.000**	**0.000**		**210.650**	**0.000**		**0.000**		**219.161**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0.000	1954.710	5549.90	0.000
due to Substructure		0.000	0.000		0.00	0.000
due to Active Earth pressure	1.00	1773.192			4618.64	
due to dynamic increment of EP	1.50	0.000			0.00	
due to Live load surcharge	0.20	130.421			456.47	
due to dynamic increment of Surcharge	1.50	0.000			0.00	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1773.19	4618.64						
0.00	0.00						
130.42	456.47						
0.00	0.00						
2635.47	**9565.08**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4615.770	4615.770	KN
ML	9775.734	9775.734	kNm
MT	3481.312	3481.312	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4582.675**	**0.000**	**269.844**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0	1954.710	5549.902	0
due to Substructure		0.000	0		0.000	0
due to Active Earth pressure	1.00	1901.979			5591.818	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1901.98	5591.82						
0.00	0.00						
2633.84	**10081.78**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4582.675	4582.675	KN
ML	10351.627	10351.627	kNm
MT	3262.151	3262.151	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4769.226**	**0.000**	**294.095**	**0.000**	**361.888**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0	1954.710	5549.902	0
due to Substructure		0.000	0		0.000	0.000
due to Earth pressure	1.00	1901.979			5591.818	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	130.421			456.475	
Surcharge	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1901.98	5591.82						
0.00	0.00						
130.42	456.47						
0.00	0.00						
2764.26	**10538.26**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4769.226	4769.226	KN
ML	10832.353	10832.353	kNm
MT	3624.039	3624.039	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4522.601**	**0.000**	**198.538**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0	1954.710	5549.902	0.000
due to Substructure		0.000	0		0.000	0.000
due to Active Earth pressure	1.00	1773.192			4618.644	
due to dynamic increment of EP	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1773.19	4618.64						
0.00	0.00						
2505.05	**9108.61**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4522.601	4522.601	KN
ML	9307.147	9307.147	kNm
MT	3262.151	3262.151	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.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4615.770**	**0.000**	**210.650**	**0.000**	**219.161**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		904.630	0	1954.710	5549.902	0
due to Substructure		0.000	0		0.000	0.000
due to Earth pressure	1.00	1773.192			4618.644	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	130.421			456.475	
Surcharge	1.50	0.000			0.000	

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
731.86	4489.97	0.00	0.00	531.73	3262.15	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1773.19	4618.64						
0.00	0.00						
130.42	456.47						
0.00	0.00						
2635.47	**9565.08**	**0.00**	**0.00**	**531.73**	**3262.15**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4615.77	4615.77	KN
ML	9775.73	9775.73	kNm
MT	3481.31	3481.31	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 (1956.775 - 1948.575) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.20 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1956.775 - 1948.575) = 0.000 kNm

	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

LOAD CASES		Total forces at bottom of abutment shaft		
		P	ML	MT
Normal Dry Case		kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case		4582.675	13147.536	3262.151
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		5981.802	16068.272	5976.310
Normal HFLCase				
Case 3 : DL+SIDL-Normal HFL Case		4522.601	11668.948	3262.151
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		5221.374	14498.638	4905.862
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	4582.675	10351.627	-3262.151
	UP	4582.675	10351.627	-3262.151
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	4769.226	10832.353	3624.039
	UP	4769.226	10832.353	3624.039
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	4522.601	9307.147	3262.151
	UP	4522.601	9307.147	3262.151
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	4615.770	9775.734	3481.312
	UP	4615.770	9775.734	3481.312
Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	4582.675	10351.627	3262.151
	UP	4582.675	10351.627	3262.151
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	4769.226	10832.353	3624.039
	UP	4769.226	10832.353	3624.039
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	4522.601	9307.147	3262.151
	UP	4522.601	9307.147	3262.151
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	4615.770	9775.734	3481.312
	UP	4615.770	9775.734	3481.312

MAX =	**5981.80**	**16068.27**	**5976.31**

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)	=	13.893	m
Area of Concrete (Ac)	=	13.893	m^2
Clear Cover to earth faces	=	75	mm
Clear Cover to non earth faces	=	50	mm
Maximum Dia of Vertical Reinf.	=	25	mm
Dia of Horizontal Reinf.	=	16	mm
Effective cover	=	141	mm

As per Clause 7.6.4.1 of IRC:112-2011

Ultimate axial force (Pu) = **5981.80** kN

0.1 fcd Ac = 0.1 x 15.63 x 13893404.1
= 21720022 N
= **21720.02** 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)	=	16068.27 kNm	=	**1156.540 kNm/m**

Check For Depth of Wall :

Mult = 0.165 x fck x b x d^2
= 1156.54 kNm/m

b = 1000.00 mm

Effective Depth Required (dreq) $= \mathrm{SQRT}\left(\dfrac{1156.54 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

(dreq) = 447.512 mm

Total Depth Required (Dreq)	=	551.01	mm	
Total Depth Provided (Dprov)	=	1000.00	mm	OK
Effective depth provided(deff)	=	859.00	mm	

R= Mu/(b d^2) = **1.57**

Minimum Longitudinal Reinforcement in wall on each face
= 0.0012 x b x D
Ast min = **1200.00** mm^2/m

Area of Steel Required:

$\dfrac{pt}{100} = \dfrac{Ast_{req}}{b\,D} = \dfrac{fck \{1 - \mathrm{sqrt}(1 - 4.598\,R/fck)\}}{2\,fy}$
= 0.0038

Ast_{req} = 3810.874 mm^2/m

Ast required = max(Astmin, Astreq) = **3810.87** mm^2/m

Total are of steel required in full length = **52946.01** mm^2

Provide	25	mm dia	@	100.00	mm c/c	=	4908.74	mm^2/m	OK
Provide	0	mm dia	@	90.00	mm c/c	=			

Effective length of shaft = 13661.404 mm

Calculation of reinforcement in numbers

Provide	25	mm dia	-	137.00	nos	=	67249.72	mm^2	OK
Provide	0	mm dia	-	152.00	nos	=			

Percentage of steel = **0.484** %

Check for Moment of Resistance of Section due to Steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

= $\frac{0.0035 \times 859.00}{0.0035 + 0.0022}$

= 529.88 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 4908.74}{0.36 \times 35.00 \times 1000.00}$

= 169.38 mm **OK**

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

z = d - 0.416 . X
= 859.00 - 70.46
= 788.54 mm

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

MR = fyd . Ast . z

= 434.78 x 4908.74 x 788.54

= 1.68E+09 Nmm /m

= 1682.92 kNm/m > 1156.54 kNm/m

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

= 0.0012 x b x D

Ast min = **1200.00** mm^2/m

= **16672.08** mm^2

Provide	16	mm dia	@	100.00	mm c/c	=	2010.62 mm^2/m	OK

Calculation of reinforcement in numbers

Provide	16	mm dia	@	137.00	nos	=	27545.48 mm^2	OK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

maximum of following = 0.2500 x 6919.36 = 1729.839 As per IRC: 112-2011 , Clause 16.3.2
= 0.001 x 1.00E+06 = 1000.000

Maximum Horizontal Reinf. **1730** mm^2 per meter

Min dia of bar = 0.250 x 25 = 6.25 mm
or 8 mm

Maximum Spacing between <= 300 mm c/c

2 Legged	16 dia	@	200 c/c	=	2010.619 mm^2	OK

Confinement Reinforcement

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

Distance between links or ties (ST) = 1/3 x 843 = 281.000
or 200.00 mm

Governing Spacing = 200.00 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)

SL	=		5	x	25 =	125	mm
	or	1/5		x	843 =	168.6	mm
Min	=		100 mm				

2 Legged	16 dia	@	100 c/c	=	4021.239	mm^2	OK
24 Legged	10 dia	@	100 c/c	=	18849.556	mm^2	
40 links	10 dia	@	100 c/c	=	31415.927	mm^2	
					54286.721	mm^2	

Minimum Confinement Reinforrcement:

nk = $\frac{NED}{Ak\ fck}$ = $\frac{5981802.3}{486269142}$ = 0.0123

AC = 13.893 mm^2

ACC = 0.875 x 13.793 = 12.069 mm^2

ρ L = 0.00486 per meter

ρ L = 0.06753

fyd = 434.783

fcd = 15.633

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

$\omega_{wd.req}$ = 0.2132

ω_{wd} = max ($\omega_{wd,req}$, 0.12) = 0.2132

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

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

Volumetric ratio,

Asw	=	54286.721	mm^2
SL	=	100.000	mm
b	=	843.000	mm
ρw	=	0.644	
$\omega_{wd.c}$	=	17.910	

$\omega_{wd,c}$ ≥ ω_{wd} as per equation 17.7 of IRC:112-2011

$\omega_{wd,c}$ = 17.91 OK

Length of Potential Plastic Hinges Refer clause 17.2.1.4 of IRC:112-2011

nk = $\frac{NED}{Ak\ fck}$ = 0.0123 < 0.30

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1948.575 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			1102.748	0.130	143.357	0.000	0.000
SIDL except Wearing Course	1			80.200	0.130	10.426	0.000	0.000
Wearing Course	1			121.302	0.130	15.769	0.000	0.000
				1304.250		**169.553**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	2.501	62.520	-0.210	-13.129	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.588	14.692	-0.210	-3.085	0.000	0.000
Bracket - Uniform portion	1	25	1.250	31.260	-0.510	-15.943	0.000	0.000
Bracket - Tapered portion	1	25	0.625	15.630	-0.460	-7.190	0.000	0.000
Cap - (uniform portion)	1	25	3.001	75.024	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.210	-5.880	0.000	0.000
Aproach Slab	1	25	7.294	182.351	-0.510	-92.999	0.000	0.000
				409.478		**-138.226**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1	25	65.796	1644.904	0.100	163.885	0.000	0.000
Total				**3358.633**		**195.212**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		664.255	1954.710	4075.207
due to Earth pressure	1	1901.979		5591.818
				9667.026

Summary of Forces at Bottom of abutment shaft

P	3358.633	KN
ML	9862.237	kNm
MT	0.000	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		3358.633		195.212		0.000
CWLL-Max. Reaction case	1	932.751	0.130	121.258	1.940	1809.439
Total		**4291.384**		**316.469**		**1809.439**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		664.255	1954.710	4075.207
due to Earth pressure	1	1901.979		5591.818
due to Live load surcharge	0.8	521.686		1825.900
				11492.925

Summary of Forces at Bottom of abutment shaft

P	4291.384	KN
ML	11809.395	kNm
MT	1809.439	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				1304.250		169.553		0.000
Substructure-Portion 1				409.478		-138.226		0.000
Substructure-Portion 2								
Shaft above HFL	1.000	25.000	36.130	903.238	0.10	89.99	0.00	0.00
Shaft below HFL	1.000	23.500	29.667	697.166	0.03	21.08	0.00	0.00
				3314.133		**142.393**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		664.255	1954.710	4075.207
due to Earth pressure	1	1773.192		4618.644
				8693.851

Summary of Forces at Bottom of abutment shaft

P	3314.133	KN
ML	8836.244	kNm
MT	0.000	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		3314.133		142.393		0.000
CWLL-Max. Reaction case	1	465.849	0.130	60.560	2.352	1095.807
Total		**3779.982**		**202.953**		**1095.807**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		664.255	1954.710	4075.207
due to Earth pressure	1	1773.192		4618.644
due to Live load surcharge	0.8	521.686		1825.900
				10519.751

Summary of Forces at Bottom of abutment shaft

P	3779.982	KN
ML	10722.704	kNm
MT	1095.807	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 (	1956.775 - 1948.575)	=	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	3358.633	9862.237	0.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	4291.384	11809.395	1809.439
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	3314.133	8836.244	0.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	3779.982	10722.704	1095.807

IN RARE COMBINATION

Max SLS Moment	=	11809.395	kNm
Max Moment per meter	=	850.000	kNm/m

IN QUASI-PERMANENT

Max SLS Moment	=	9862.237	kNm
Max Moment per meter	=	709.850	kNm/m

Check For Stresses in Rare and Quasi-Permanent Load Combination

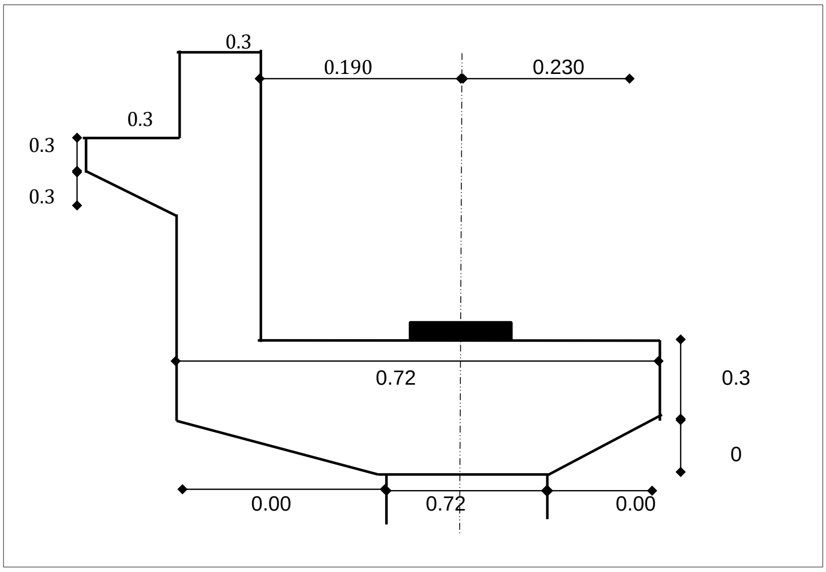
Creep Coeff = 1.2

		Rare Combination		Quasi permanent	
		Short term	Long Term		
Working bending moment, M	=	850.00	850.00	709.85	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 uncracke	=	0.17	0.17	0.17	m^3
Bending Stress (M/ZL)	=	5.100	5.100	4.259	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	14685.57	14685.57	N/mm^2
Modular Ratio (m)	=	6.19	13.62	13.62	
Clear Cover, c	=	75.000	75.000	75.00	mm
Maximum dia used, ϕ	=	25.000	25.000	25.00	mm
Effective Depth deff (dy)	=	859.000	859.000	859.00	mm
Ast provided	=	4908.739	4908.739	4908.74	mm^2/m
Percentage of steel , pt	=	0.0048	0.0048	0.0048	
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m		0.217	0.303	0.303	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	186.115	260.375	260.375	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	672.885	598.625	598.625	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	585.385	511.125	511.125	mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2			
Icr	=	1.591E+10	2.984E+10	2.984E+10	mm^4
Maximum compressive stress in concrete	=	**9.945**	**7.417**	**6.194**	**< 16.8, SAFE**
Maximum tensile stress in concrete	=	35.955	17.052	14.240	
Maximum Tensile stress in steel	=	**193.633**	**198.281**	**165.588**	**< 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)	=	437.500	mm
Provided Spacing	=	100.000	mm
Check for Applicability of Formula	=	OK	
Maximum crack spacing , $S_{r\,max}$	=	3.4 c +	0.425 k1 k2 φ / $\rho_{\rho\,eff}$
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	260.375	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.	
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	352.500	mm
Ac, eff = Dx * hc,eff	=	352500.000	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	0.014	
Maximum crack spacing , $S_{r\,max}$	=	560.196	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\sigma_{sc} - \frac{k_t f_{ct\,eff} (1 + \alpha_e \rho_{\rho\,eff})}{\rho_{\rho\,eff}}$	/ Es
tensile stress in steel, ,σsc	=	165.588	N/mm^2
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm^2
αe = Es/Ecm	=	13.619	
(εsm - εcm)	=	0.00050	
Crack width , Wk=Sr max (εsm - ε	=	**0.278**	**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.

Assuming a cap thickness of	=	225	mm				
Volume of abutment cap	=	225	x	720	x	13893.4	
	=	2.25E+09	mm^3				

as per cl. 710.8.7 of IRC : 78- 2014

Quantity of steel = 1 % of volume

$= \frac{1}{100} \times 2.25E+09 = 2.25E+07\ mm^3$

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction

$= 1.13E+07\ mm^3$

Clear cover = 50 mm

Length of bar = 13893.404 -100 = 13793.4 mm

Area of steel required in longitudinal direc = $\frac{1.13E+07}{13793.404}$ = 815.8724 mm^2 (top +Bottom)

Provide 7 Nos. of 12 mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

Provided steel = 792 mm^2

(b) Transverse steel

Volume of steel to be provided in transverse direction = 1.13E+07 mm^3

Volume of steel required per meter = $\frac{1.13E+07}{13.89}$ = 8.10E+05 mm^3/m

Provide 2 L 12 mm dia bar @ 150 mm c/c stirrups

Length of each stirrups = 720 -100 -12 = 608 mm

Volume of steel provided per meter = 9.17E+05 mm^3/m **OK**

DESIGN OF DIRT WALL

Dirt wall will be designed as a vertical cantilever.

1.) NORMAL CASE

1a. Dead Load

Self Weight of Dirt Wall	=	3.089	m^3 x	25.00	=	77.213 kN
Self Weight of Dirt Wall/ m	=	77.213 /		13.89	=	5.557 kN

1b. Live Load

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

Vertical Load on Dirt Wall = 200 kN

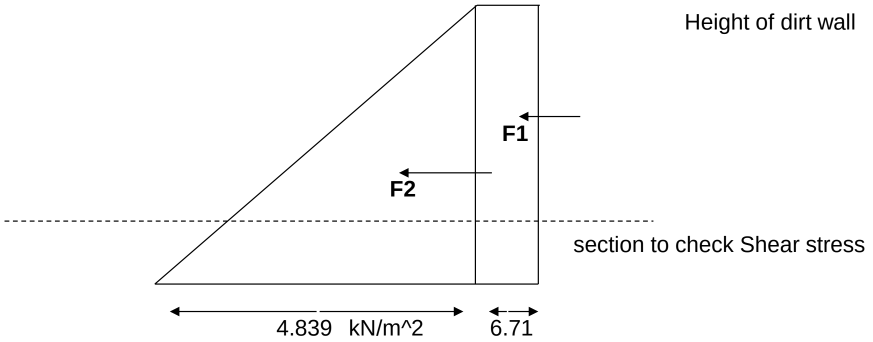
Braking Load

Assuming 20% braking Force i.e. 0.2 *200 = 40.000 kN acting at 1.2 m above deck

Effective Width = 2.79 m

Moment (due to Breaking) = $\frac{40.000 \times 2.066}{2.79}$ = 29.620 kNm/m

1c. EARTH PRESSURE



Height of dirt wall = 0.866 m

Normal Earth Pressure

Earth Pressure Diagram

Intensity for rectangular portion	=	0.279	x 20	x 1.2		=	6.705	kN/m^2
F1	=	6.705	x 0.87	x 1.00		=	5.807	kN/m
Intensity for triangular portion	=	0.2794	x 20	x 0.866		=	4.839	kN/m^2
F2	=	0.50	x 4.84	x 0.866	x 1.00	=	2.095	kN/m

Moment @ RL **1954.71** m (at dirt wall base)

M1	=	5.807	x	0.433	=	2.514	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	=	2.095	x	0.364	=	0.762	kN.m/m

Design Horizontal Forces (Normal Case):

Load Factor For Live Load Surcharge	=	1.2
Ultimate Moment due to Live Load Surcharge	=	3.017 kN.m/m
Load Factor For Earth Pressure	=	1.5
Ultimate Moment due to Earth Pressure	=	1.143 kN.m/m
Load Factor For Braking Force	=	1.5
Ultimate Moment due to Braking Force	=	44.430 kN.m/m
Total Ultimate Moment	**=**	**48.590 kN.m/m**

Material Property:

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	N/mm^2
Design Yield Strength of Reinforcement, fyd	=	434.783	N/mm^2
Modulus of Elasticity of Steel (Es)	=	200000	N/mm^2

(a) Vertical steel on earth face As per Clause 16.3.1 of IRC:112-2011

Adorting clear cover on either face	=	50	mm				
Minimum Dia of Reinforcement	=	12	mm				
Maximum Spacing of Steel	=	200	mm				
Thickness of dirtwall	=	0.300	m				
Available effective depth	=	300	-50	-6	=	244	mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 48.59 kNm/m

Effective Depth of Cap Required (dreq) = SQRT($\frac{48.59 \times 1000000}{0.165 \times 35.00 \times 1000}$) = 91.727 mm

Total Depth Required (Dreq) = 147.73 mm

Total Depth Provided (Dprov) = 300.00 mm **OK**

R= Mu/(b d^2) = 0.816

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\,d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2 fy}$ = 0.002

Ast_{req} = 470.803 mm^2/m

As per Clause 16.3.1 of IRC:112-2011

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

Maximum (Ast_{req}, Ast_{min}) = 470.803 mm^2/m

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

Provide Ast = 565 mm^2/m) OK

Percentage of Steel Provided = 0.232 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \,.\, d}{(0.0035 + fyd/ Es)}$ = $\frac{0.0035 \times 244}{0.0035 + 0.00217}$

= 150.5134 mm

Depth of Neutral Axis , = $\frac{fyd \,.\, Ast}{0.36 \,.\, fck \,.\, b}$

= $\frac{435 \times 565}{0.36 \times 35.00 \times 1000}$ = 19.523 mm **OK**

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

z = d - 0.416 . X = 244 - 8.121

= 235.879 mm

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

MR = fyd . Ast . z = 434.78 x 565 x 235.88

= 5.8E+07 Nmm = 57.994 kNm/m > 48.59 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 117.701 mm^2/m
Governing Ast	=	150.000 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

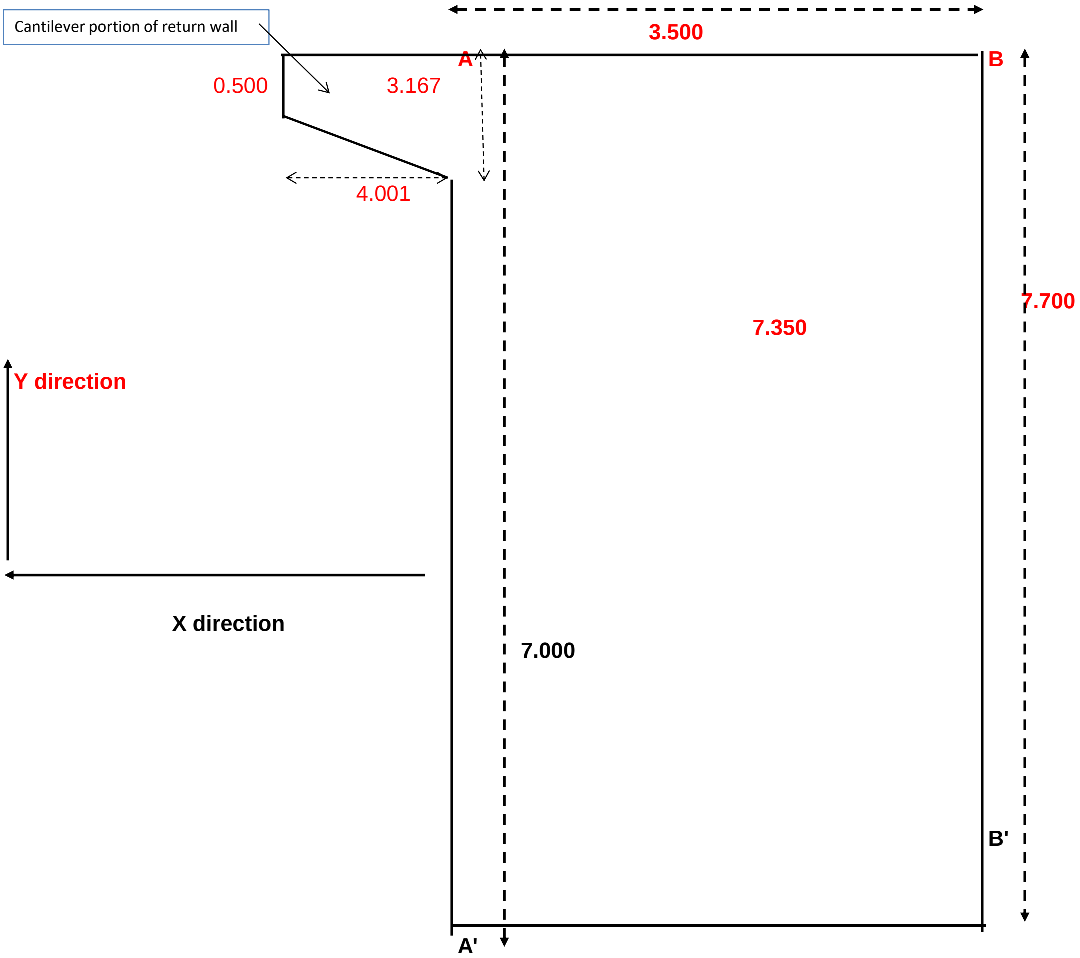
Provide 10 mm dia bar @ 200 mm c/c as vertical steel at earth face.

Provided Ast= 393 mm^2/m) OK

Design of Solid Return wall

FOR MINOR BRIDGE AT CH. 152+790

THICKNESS OF SOLID RETURN WALL	=	0.500	m
THICKNESS OF CANTILEVER RETURN WALL	=	0.500	m



Width of Solid Return	a	=	3.50	m
Avg. Heigth of Solid Return	b	=	7.350	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 = 0.476

a/b	=	0.375	β1 =	0.353	β2 =	0.398
a/b	=	0.5	β1 =	0.631	β2 =	0.632

For uniformly distributed load over entire plate

For, a/b = 0.476 β1 = 0.578

β2 = 0.587

Live Load Surcharge Intensity:

q = 0.2794 x 20.00 x 1.200 = 6.705 kN/m^2

Max. .σ b = $\dfrac{\beta 1 \times q \times b\text{^}2}{(t1)\text{^}2}$

.σ a = $\dfrac{\beta 2 \times q \times b\text{^}2}{(t2)\text{^}2}$

.σ b = $\dfrac{0.578 \times 6.705 \times 54.023}{0.250}$

At bottom edge = 837.550 kN/m2 = **0.838 MPa**

For 1000 mm of width , Z = $\dfrac{1000 \times 250000}{}$

6

= 4.17E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 0.838 x 4.167E+07

= 34897897 Nmm/m = **34.898 kN.m/m**

σ a = (0.587 x 6.705 x 54.023) / 0.250

= 851 = **0.8511 MPa**

For 1000 mm of height , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence, Moment /m height along X direction -

Mx /m height = 0.8511 x 4.167E+07 = 3.546E+07 Nmm/m

= **35.464 kN.m/m**

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.375	β1 =	0.212	β2 =	0.148
a/b =	0.5	β1 =	0.328	β2 =	0.2

For, a/b = 0.476 β1 = **0.306**

β2 = **0.190**

q = 0.279 x 20.00 x 7.35

= 41.069 kN/m^2

Max. σ b = (β1 x q x b^2) / (t1)^2

σ a = (β2 x q x b^2) / (t2)^2

σ b = (0.306 x 41.069 x 54.023) / 0.25

= 2714.81 kN/m^2

= **2.715 MPa**

For 1000 mm of width , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 2.715 x 4.167E+07

= 113117023 Nmm/m = **113.117 kN.m/m**

σ a = (0.190 x 41.069 x 54.023) / 0.25

= 1687.0 kN/m^2 = 1.687 MPa

For 1000 mm of height , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence Moment /m height along X direction -

Mx /m height = 1.687 x 4.167E+07 = 7.029E+07 Nmm/m

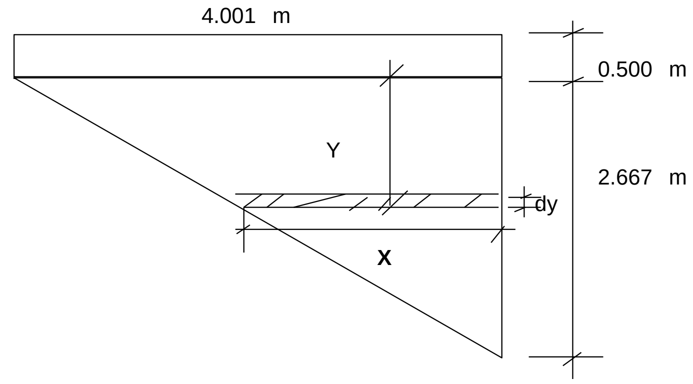
= **70.293 kN.m/m**

Total Moment in Solid Return Wall / m height = 105.757 kN.m/m

Total Moment in Solid Return Wall / m width = 148.015 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 =		148	kN.m/m
Total Moment(My) in Solid Return Wall / m width =		212	kN.m/m



X = 4.001 + -1.500 y

Earth pressure due to LL surchage = 0.279 x 20.000 x 1.200
= 1.20 x 6.705 = **8.046** kN / m^2
load factor

Earth pressure at a depth of 0.500 m from top
= 0.279 x 20 x 0.5
= 1.50 x 2.794 = **4.191** kN / m^2
load factor

Earth pressure at a depth y from top = 0.279 x 20 x y
= 1.50 x 5.588 = **8.382 y**
load factor

A *LL surchage on top* *0.5 m*

Force = 8.046 x 4.00 x 0.5 = 16.095 kN

Moment at face BB' = 16.095 x (4.001 / 2 + 3.500)
(Mx) 1

= **88.524** kN.m

B *LL surchage on triangular portion*

Force for unit strip = 8.046 x **X** x dy = 8.046 X dy

(dMx) 2 = 8.046 **X** dy x { **X** / 2 + 3.50 }

= 8.046 x { **X** ^2 / 2 + 3.5 **X** } dy

= 8.046 x { (4.001 + -1.500 y) ^2 / 2 +
3.50 x (4.001 + -1.500 y) } dy

= 8.046 x { (8.002 + 1.125 y^2 +
-6.001 y) + 14.002 + -5.250 y } dy

= 8.046 x (22.004 + 1.125 y^2 +
-11.251 y) dy

After integrating between limits 0 and 2.667 m

(Mx) 2 = 8.046 x (22.004 x 2.667 + 0.375
x 2.667 ^3 + -5.625 x 2.67 ^2)

= 8.046 x (58.684 + 7.114 + -40.013)

= **207.473** kN.m

C *Earth press. on top* *0.5 m*

Force = 0.5 x 4.191 x 4.001 x 0.5
= 4.191 kN

Moment at face BB' = 4.191 x (4.001 / 2 + 3.500)
(Mx) 3

= **23.053 kN.m**

D *Earth pressure on triangular portion*

Force for unit strip = (4.191 + 8.382 y) **X** x dy

(dMx) 4 = (4.191 + 8.382 y) X x { **X** / 2 + 3.5 }

= (4.191 **X** ^2 + 14.668 **X** + 4.1908 **X** ^2 x y +
(Part 1) *(Part 2)* *(Part 3)*

29.335 **X** x y) dy
(Part 4)

Part 1 :
(dMx) 4.1 = 4.191 **X** ^2 dy

= 4.191 x (4.001 + -1.500 y) ^2 dy

= 4.191 x (16.0040003 + 2.2500 y^2 + -12.002 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.1 = 4.191 x (16.0040003 x 2.667 + 0.7500 x 2.67 ^3 + -6.001 x 2.667 ^2)

= 4.191 x (16.004 x 2.6670 + 0.750 x 18.97 + -6.001 x 7.113)

= 4.191 x (42.683 + 14.228 + -42.6827)

= 4.191 x (14.228)

= **59.624 kN.m**

Part 2 :
(dMx) 4.2 = 14.668 **X** dy

= 14.668 x (4.001 + -1.500 y) dy
= (58.678 + -22.001 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.2 = 58.678 x 2.6670 + -11.001 x 2.6670 ^2

= 156.494 + -78.247 **=** **78.247 kN.m**

Part 3 :

(dMx) 4.3 = 4.191 **X** ^2 x y dy

= 4.191 x (4.001 + -1.500 y) ^2 x y x dy

= 4.191 x (16.0040003 y + 2.250 y ^3 + -12.0015 y^2) x dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.3 = 4.191 x (8.00200013 x 2.6670 ^2 + 0.5625 x 2.66700 ^4 + -4.00050 x 2.66700 ^3)

= 4.191 x (56.917 + 28.459 + -75.890)

= 4.191 x (9.486) **=** **39.754 kN.m**

Part 4 :

(dMx) 4.4	=	29.335 **X** x y dy						
	=	29.335	x (	4.001	+	-1.500 y) x y x dy		
= (		117.356 y	+	-44.003 y^2) dy				
After integrating between limits			0.000 and			2.667 m		
(Mx) 4.4	=	58.678	x	2.667 ^2	+	-14.668	x	2.667 ^3
	=	417.369	+	-278.246	**=**	**139.123 kN.m**		
(Mx) 4	=	59.624	+	78.247	+	39.754	+	139.123
	=	**316.749 kN.m**						
Total Moment at face BB' =		(Mx) 1	+	(Mx) 2	+	(Mx) 3	+	(Mx) 4
		88.524	+	207.473	+	23.053	+	316.749
	=	**635.799 kN.m**						
Horizontal moment per meter =		635.799 /		7.350 =		**86.503 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) =

= **147.997** **+** **86.50**

= **234.50 kN.m / m**

Adorting clear cover on either face	=	75 mm		
Minimum Dia of Reinforcement	=	16 mm		
Maximum Spacing of Steel	=	125 mm		
Thickness of wall	=	0.500 m		
Available effective depth	=	500	-75	-8
	=	417 mm		

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 234.50 kNm/m

Effective Depth of Cap Required (dreq) = SQRT((234.50 x 1000000) / (0.165 x 35.00 x 1000))

Effective Depth of Cap Required (dreq)	=	201.509 mm	
Total Depth Required (Dreq)	=	284.51 mm	
Total Depth Provided (Dprov)	=	500.00 mm	**OK**
R= Mu/(b d^2)	=	1.35	

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d} = \frac{fck\ \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 1355.818 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011

= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1355.818 mm^2/m

Provide 16 mm dia bar @ 125 mm c/c as Horizontal steel at earth face.

Provide Ast= 1608 mm^2/m) OK

Percentage of Steel Provided = 0.386 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \,.\, d}{(0.0035 + fyd/Es)}$

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

= 257.230 mm

Depth of Neutral Axis , X = $\frac{fyd \,.\, Ast}{0.36 \,.\, fck \,.\, b}$

= $\frac{434.78 \times 1608}{0.36 \times 35.00 \times 1000}$

= 55.504 mm OK

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

z = d - 0.416 . X
= 417 - 23.090
= 393.910 mm

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

MR = fyd . Ast . z

= 434.78 x 1608 x 393.910

= 2.75E+08 Nmm

= 275.480 kNm/m > 234.50 kNm/m

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

Provide 12 mm dia bar @ 125 mm c/c as Horizontal steel at non earth face.

Provided Ast = **905 mm^2/m)**

2. Design for Face A'B'

Moment in Solid Return /m width = 211.55 kN.m / m

Adorting clear cover on either face = 75 mm
Minimum Dia of Reinforcement = 16 mm
Maximum Spacing of Steel = 150 mm
Thickness of wall = 0.500 m
Available effective depth = 500 -75 -16 -8
= 401 mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 211.55 kNm/m

Effective Depth of Cap Required (dreq) = SQRT($\frac{211.55 \times 1000000}{0.165 \times 35.00 \times 1000}$)

Effective Depth of Cap Required (dreq) = 191.396 mm

Total Depth Required (Dreq) = 274.40 mm
Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 1.32

Area of Steel Required:

$\frac{pt}{100}$ = $\frac{Ast_{req}}{b\ d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 1270.361 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1270.361 mm^2/m

Provide	**16 mm dia bar @**	**150 mm c/c as vertical steel at earth face.**	

Provide Ast=	**1340**	**mm^2/m)**	**OK**

Percentage of Steel Provided = 0.3343 %

Provide	**12 mm dia bar @**	**150 mm c/c as Vertical steel at non earth face.**

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\dfrac{0.0035\ .\ d}{(0.0035 + fyd/\ Es)}$

$= \dfrac{0.0035 \times 401}{0.0035 + 0.00217}$

= 247.36 mm

Depth of Neutral Axis , X = $\dfrac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$

$= \dfrac{434.78 \times 1340}{0.36 \times 35.00 \times 1000}$

= 46.253 mm OK

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

z = d - 0.416 . X
= 401 - 19.241
= 381.76 mm

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

MR = fyd . Ast . z

= 434.78 x 1340 x 381.759

= 2.22E+08 Nmm

= 222.484 kNm/m > 211.55 kNm/m

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

b) Cantilever Portion of Return Wall

Self-weight of cantilever portion of return wall	=	23	kN/m
Crash Barrier weight	=	10.0	kN/m
Total Load	=	33	kN/m
Moment at Cantilever Face	=	263	kNm
Load Factor	=	1.35	
Design Moment	=	356	kNm
Effective Depth	=	3104.500	mm

R= Mu/(b d^2) = 0.07

Area of Steel Required:

$\dfrac{pt}{100} = \dfrac{Ast_{req}}{b\ d} = \dfrac{fck\ \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.000

Ast_{req} = 263.985 mm^2

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 1862.7 mm^2

Maximum (Ast_{req}, Ast_{min}) = 1862.7000 mm^2

Provide	**25**	**4**	**=**	**1963**	**mm^2**

DESIGN OF RCC SOLID SLAB

SPAN - 10.00m (SKEW 0^{o})

OVERALL DECK WIDTH –11.0 m

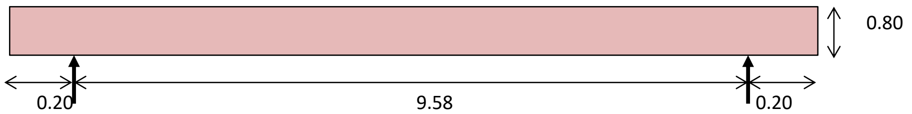
INPUTS FOR DESIGN OF RCC SOLID SLAB

1.0 Input Data :

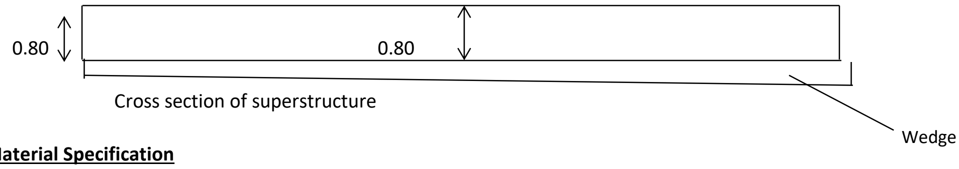
Clear span of the slab	=	9.18	m
Bearing over support	=	0.40	m
Span between Centre to centre of bearing	=	9.58	m
Thickness of filler type exp. Gap	=	0.02	m
Overall span of slab	=	10.00	m
Overall width of slab	=	11.00	m
Carriageway width	=	10.00	m
Width of footpath	=	0.00	m

Camber	=	2.50	%(uni directional)
Depth of Slab at Carriageway edge	=	0.800	m
Depth of Slab at Carriageway centre	=	0.800	m
Wearing Coat thickness	=	0.065	m
Clear cover to steel	=	0.050	m
Diameter of main reinforcement bar	=	0.020	m

Avg. Depth of Solid Slab	=	0.80	m				
Effective Depth of Slab	=	0.80		-	0.05	-	0.010
	=	0.740	m				
Effective Span Min(l+ws , l+d)							
L	=	9.58	m				



Span Arrangment of superstructure



Cross section of superstructure

2.0 Material Specification

Concrete Grade	=	M	30	
Characteristic Compressive Strength of Concrete,fck	=	30.00	Mpa at 28 days	
Design Compressive strength of Concrete,fcd	=	13.40	Mpa at 28 days	(0.67/1.5 * fck)
Tensile strength of concrete , fctm	=	2.50	MPa	
Strain at reaching Characteistic Strength, ec2	=	0.02		
Ultimate Strain, ecu2	=	0.035		
Modulus of Elasticity of Concrete (Ec)	=	2.74E+04	N/mm^2	(5000 x sqrt (fck)
Ecm	=	3.12E+04	N/mm^2	
Steel Grade		Fe	500	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	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	= 14.4	N/mm^2
Maximum tensile stress in steel under rare combination	=	300	N/mm^2	

INPUTS FOR DESIGN OF RCC SOLID SLAB

3.0 Calculation of Load ,Bending Moment & Shear Force

3.1 Self-weight of Deck Slab

Average thickness of deck slab	=	0.80 m			
U.D.L. Due to deck slab weight	=	20.00 kN/m^2			
Bending Moment Deck Slab weight	=	229.44 kNm/m			
Reaction	=	100.00 kN/m			
Shear Force at deff section	=	81.20 kN/m			
Shear Force at mid-span	=	0.00			

3.1 SIDL1 Due to Crash Barrier & Railing

Due to crash barrier	=	2.00	x 8.00	=	16.00 kN/m
U.D.L due to crash barrier	=	16.00	/ 11.00	=	1.45 kN/m^2
Due to RCC Railing	=	2.00	x 0.00	=	0.00 kN/m
	=	0.00	/ 11.00	=	0.00 kN/m^2

Total SIDL1 due to crash barrier & railing	=	1.45 kN/m^2
Bending Moment due to SIDL1	=	16.69 kNm/m
Reaction	=	7.27 kN/m
Shear Force at deff section	=	5.91 kN/m
Shear Force at mid-span	=	0.00

3.2 SIDL2 Due to Wearing Course

due to wearing course and additional ove	=	2.00 kN/m^2
Bending Moment due to SIDL2	=	22.94 kNm/m
Reaction	=	10.00 kN/m
Shear Force at deff section	=	8.12 kN/m
Shear Force at mid-span	=	0.00

3.3 Footpath Load

Refer clause 206.3 of IRC:6-2014

P1	=	0 kg/m^2			
Effective span L	=	9.58 m			
Intensity of FPLL , P	=	P1	-	(40x L - 300)/9	
	=	0	-	9.24 =	-9.24 kg/m^2
P due to both side footpath	=	2	x	-9.24 =	-18.49 kg/m^2
Intensity of FPLL per running meter	=	-0.18	x	0.00 =	0 kN/m
U.D.L. Due to FPLL	=	0.00	/	11.00 =	0.00 kN/m^2
Bending Moment due to FPLL	=	0.00 kNm/m			
Reaction	=	0.00 kN/m			
Shear Force at deff section	=	0.00 kN/m			
Shear Force at mid-span	=	0.00			

3.4 CWLL Bending Moment

			interpolation	
b/L	=	1.15	1.10	2.24
α	=	2.30 For simply supported slab	1.20	2.36

beff = α* a*(1 - a / L) + b_1

A = α* a* (1 - a / L)

b_1 = Breadth of the concentration area of the load

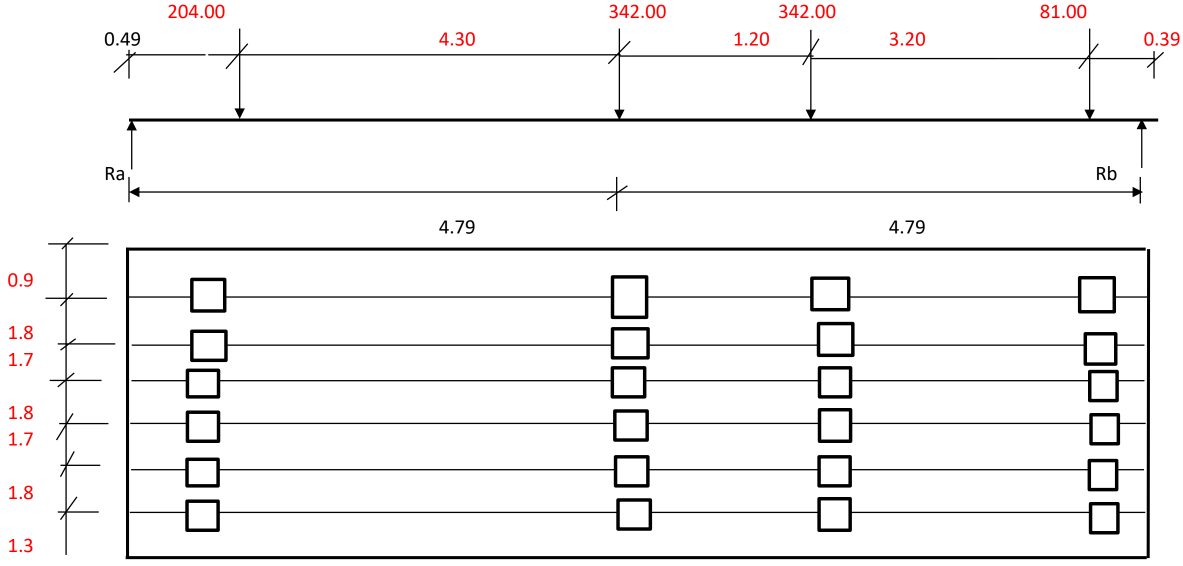
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
114.00	0.50	0.25
68.00	0.38	0.20
27.00	0.20	0.15

b_1	=	0.63	114 kN
b_1	=	0.51	68 kN
b_1	=	0.33	27 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

For Class A - 3 Lanes

I.F = 1.289

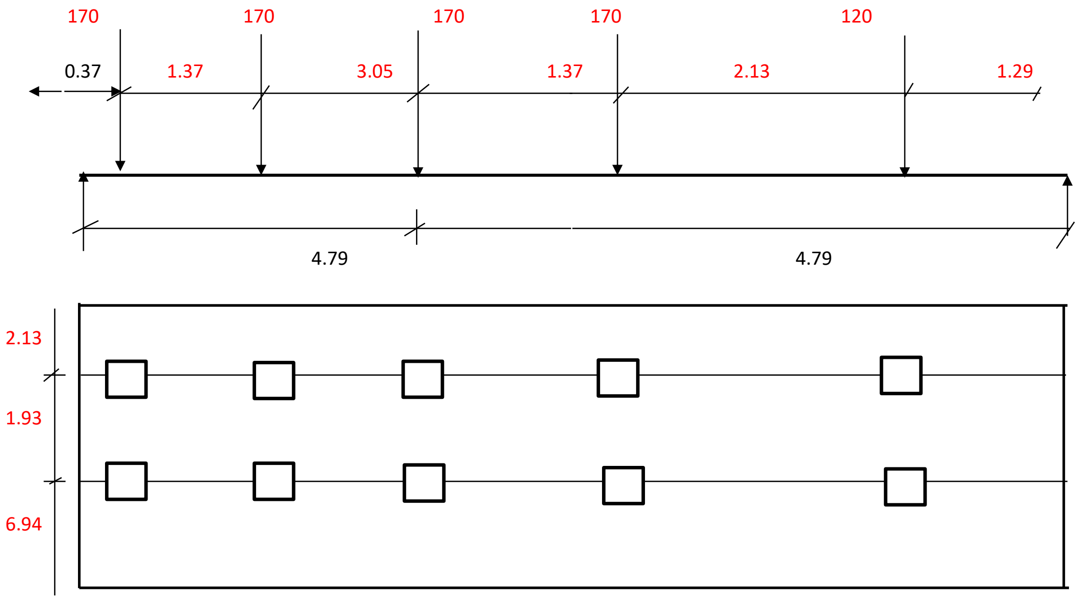


Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Bending Moment (kNm/m)
81.00	0.39	0.86	0.33	1.19	7.14	2.85
342.00	3.59	5.16	0.63	5.79	11.00	71.93
342.00	4.79	5.50	0.63	6.13	11.00	95.97
204.00	0.49	1.07	0.51	1.58	9.47	6.80
						177.55

For Class A - 3 Lanes	B . M =	**177.55** kN-m/m

CLASS 70 RW Loading

Impact Factor = 1.25



INPUTS FOR DESIGN OF RCC SOLID SLAB

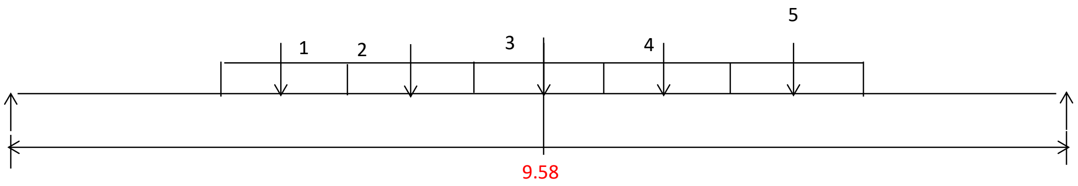
Dimensions of 70RWheel Tyre presssure = 5.273 kg/cm^2 527.30 kN/m^2

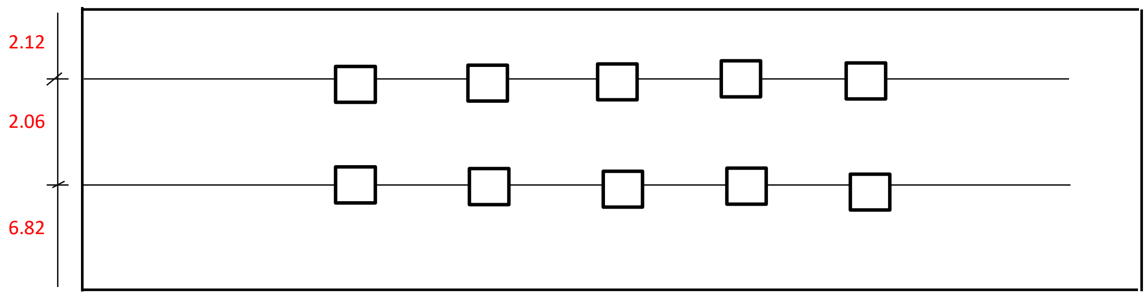
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
170.00	0.86	0.37
120.00	0.86	0.26

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
120	1.29	2.57	0.99	3.56	5.49	17.64
170	3.42	5.05	0.99	6.04	7.08	51.31
170	4.79	5.50	0.99	6.49	7.31	69.65
170	1.74	3.27	0.99	4.26	6.19	29.86
170	0.37	0.82	0.99	1.81	3.61	10.88
					B . M =	**179.34**

CLASS 70 R(T) Loading

Impact Factor = 1.10





Class 70R (T) = 700.00 KN
Length of track load = 4.57 m

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

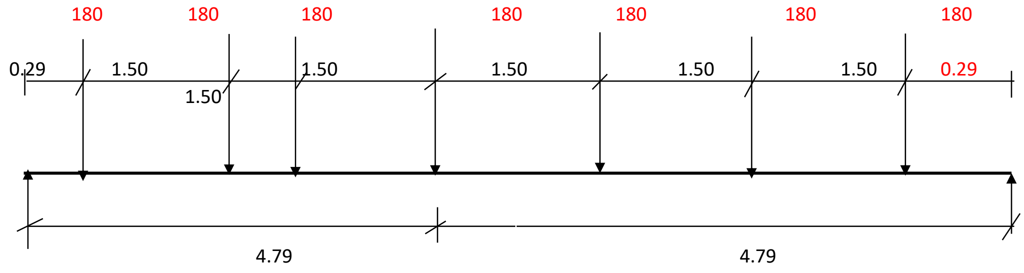
Divide the track load into 5 equal wheel load

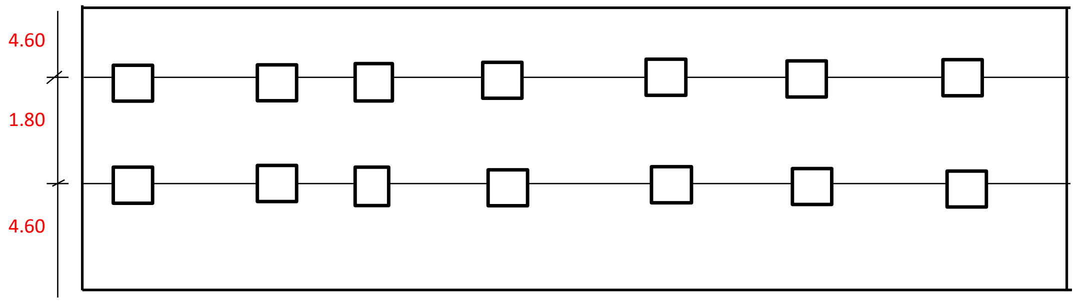
Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	2.96	4.70	0.97	5.67	7.02	32.51
140	3.88	5.30	0.97	6.27	7.32	40.79
140	4.79	5.50	0.97	6.47	7.42	49.73
140	3.88	5.30	0.97	6.27	7.32	40.79
140	2.96	4.70	0.97	5.67	7.02	32.51
					B . M =	**196.33**

INPUTS FOR DESIGN OF RCC SOLID SLAB

CLASS SV Loading

Impact Factor = 1.00





Dimensions of 70RWheel Tyre presssure = 2.250 kg/cm^2 225.00 kN/m^2

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	0.290	0.65	0.58	1.23	2.45	10.64
180	1.79	3.34	0.58	3.92	5.72	28.14
180	3.29	4.96	0.58	5.54	7.34	40.32
180	4.79	5.50	0.58	6.08	6.08	70.86
180	3.29	4.96	0.58	5.54	5.54	53.41
180	1.79	3.34	0.58	3.92	5.72	28.14
180	0.29	0.65	0.58	1.23	2.45	10.64
					B . M =	**242.17**

CWLL Bending Moment For

1.) CLASS A - 3 Lane	=	177.55	kN-m/m
2.) CLASS 70 R Wheel	=	179.34	kN-m/m
3.) CLASS 70 R Track	=	196.33	kN-m/m
4.) SV Loading	=	242.17	kN-m/m

INPUTS FOR DESIGN OF RCC SOLID SLAB

Total Bending Moment at Mid Span

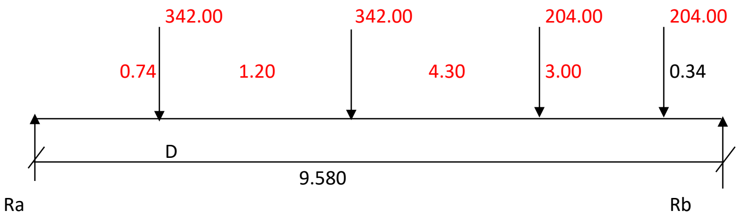
Loads	Unfactored B.M. (kNm/m)	ULS Factor	ULS Moment (kNm/m)	Rare Combination factor	Rare Combination Moment (kNm/m)	Quasi permanent combination Factor	Quasi permanent combination Moment (kNm/m)
DL	229.44	1.35	309.75	1.00	229.44	1.00	229.44
SIDL1	16.69	1.35	22.53	1.00	16.69	1.00	16.69
SIDL2	22.94	1.75	40.15	1.00	22.94	1.00	22.94
FPLL	0.00	1.50	0.00	1.00	0.00	0.00	0.00
CWLL - (Class A /70R)	196.33	1.50	294.49	1.00	196.33	0.00	0.00
CWLL - SV Load	242.17	1.00	242.17	1.00	242.17	0.00	0.00
Total Moment		=	**666.92**		**511.24**		**269.07**

3.5 CWLL Shear Force at a distance of Effective Depth from the centre of support

Thus Shear force at Point D = 73.46 kN (Including Impact Factor)

3- Lane Class A Load

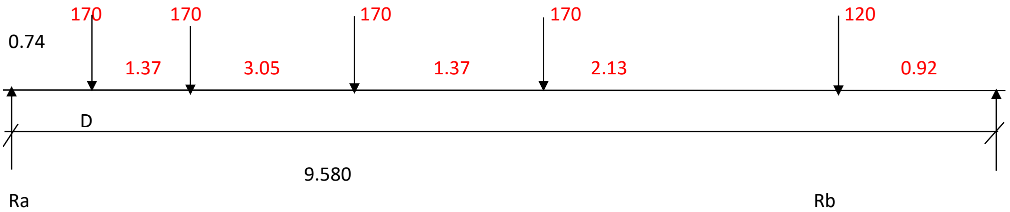
Effective Depth = 0.74 m
Distance from Support = 0.74 m



Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Shear Force (kN/m)
204.00	0.34	0.75	0.51	1.26	7.58	1.23
204.00	3.34	5.00	0.51	5.51	11.00	8.33
342.00	1.94	3.56	0.63	4.19	11.00	31.96
342.00	0.74	1.57	0.63	2.20	10.80	37.66
						79.18

Thus Shear force at Point D = 79.18 kN (Including Impact Factor)

1- 70 R Wheeled Load



Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Shear Force (kN/m)
120	0.92	1.91	0.99	2.90	4.83	2.98
170	3.05	4.78	0.99	5.77	6.94	9.74
170	4.42	5.47	0.99	6.46	7.29	13.45
170	2.11	3.78	0.99	4.77	6.45	25.71
170	0.74	1.57	0.99	2.56	4.49	43.68
						95.56

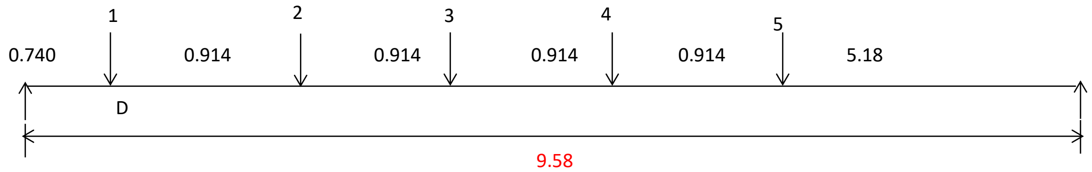
Thus Shear force at Point D = 95.56 kN (70 R Wheeled Load)

Governing Shear Force at Deff = 95.56 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

CLASS 70 R(T) Loading

Impact Factor = 1.10



Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

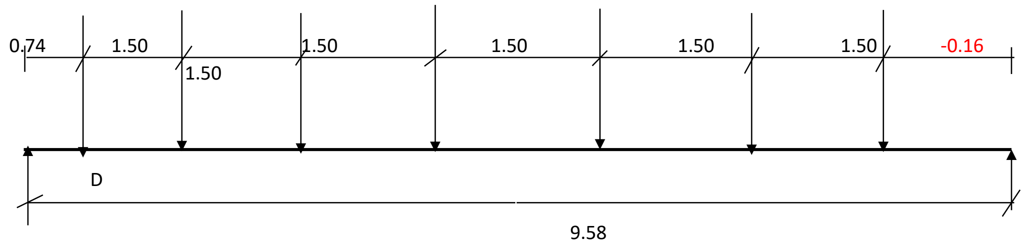
Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	0.74	1.57	0.97	2.54	4.60	18.12
140	1.65	3.14	0.97	4.11	6.17	15.88
140	2.57	4.32	0.97	5.29	6.82	16.52
140	3.48	5.09	0.97	6.06	7.21	17.67
140	4.40	5.47	0.97	6.44	7.40	19.21
					S.F =	**87.39**

Thus Shear force at Point D = 87.39 kN (70 R Track Load)

CLASS SV Loading

Effective Depth = 0.74 m

Distance from Support = 0.74 m



Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	-0.160	-0.37	0.58	0.21	0.41	-7.29
180	1.34	2.65	0.58	3.23	5.03	5.01
180	2.84	4.59	0.58	5.17	6.97	7.65
180	4.34	5.45	0.58	6.03	7.83	10.41
180	3.74	5.24	0.58	5.82	7.62	14.40
180	2.24	3.94	0.58	4.52	6.32	21.81
180	0.74	1.57	0.58	2.15	3.95	42.06
					S.F =	**94.05**

Thus Shear force at Point D = 94.05 kN (SV Load)

INPUTS FOR DESIGN OF RCC SOLID SLAB

CWLL Shear Force for

1.) CLASS A - 3 Lane	=	79.18	kN/m
2.) CLASS 70 R Wheel	=	95.56	kN/m
3.) CLASS 70 R Track	=	87.39	kN/m
4.) SV Loading	=	94.05	kN/m

Summary of Shear Force

Loads	S.F. At deff (kN/m)	ULS Factor	ULS-S.F. (kN/m)
DL	81.20	1.35	109.62
SIDL1	5.91	1.35	7.97
SIDL2	8.12	1.75	14.21
FPLL	0.00	1.50	0.00
CWLL - (Class A	95.56	1.50	143.34
CWLL - SV Load	94.05	1.00	94.05
Total			**275.15**

ULS DESIGN OF SOLID SLAB

DECK SLAB FOR ULS FLEXURAL MOMENT

Min. Thickness of slab	=	**800**	mm
Clear Cover to outer steeel	=	**50**	mm
Maximum Diameter of Reinformcement	=	**20**	mm
Effective Depth Provided (deff)	=	740	mm
Ultimate Design bending moment	=	**666.92**	kNm/m

Mulim = 0.167 x fck x b x d^2 = 666.92 kNm/m (Equation derived based on IRC:112-2011)

Effective Depth of Cap Required (dreq) = SQRT($\frac{666.92 \times 1000000}{0.167 \times 30.00 \times 1000}$)

Effective Depth of Cap Required (dreq)	=	364.852	mm
Total Depth Required (Dreq)	=	424.85	mm
Total Depth Provided (Dprov)	=	800.00	mm

OK

$R = M_{RD}/(b\ d^2)$ = 1.22

Ast Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d} = \frac{fck \{ 1 - \text{sqrt}(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 2178.876 mm^2/m

Minimum Longitudinal Reinforcement :

As. Min = 0.26 x $\frac{fctm}{fyk}$ x b . d - Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011

Whichever is higher OR = 0.0015 x b . d -Refer Clause 16.9 of IRC:112-2011'

b	=	1000.00	mm
d	=	740.00	mm
Ast min	=	**1110.00**	mm^2/m
Governing Reinf. Ast	=	**2178.88**	mm^2/m

Provide **20** mm dia @ **200** mm c/c + **20** mm dia @ **190** mm c/c

Area provided= 3224.27 mm^2/m > 2178.88 mm^2/m OK

Percentage of Steel (pt%) = 0.44 %

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

Smax = 2 h = 1480.00

OR = 250.00 mm whichever is max

Provided Spacing is less than Smax, Hence OK

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035\ .\ d}{(0.0035 + fyd/ Es)} = \frac{0.0035 \times 740.00}{0.0035 + 0.0022}$

= 456.48 mm

Depth of Neutral Axis , X = $\frac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b} = \frac{434.78 \times 3224.27}{0.36 \times 30.00 \times 1000}$

= 129.80 mm **UNDER REINFORCED DESIGN, OK**

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

z = d - 0.416 . X = 740.00 - 54.00

= 686.00 mm

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

MR = fyd . Ast . z

= 434.78 x 3224.27 x 686.00

= 9.62E+08 Nmm / m

= 961.68 kNm / m > 666.92 kNm / m SAFE

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

ULS DESIGN OF SOLID SLAB

Distribution reinforcement:

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$\frac{20.00}{100.00}$ x 3224.27 = 644.853 mm^2/m

Provide	12 dia bar	150 direction in top face. (Providing =	753.98 mm^2)	OK

DESIGN OF SLAB FOR ULS SHEAR

Ultimate Design Shear Force = 275.15 kN/m

Design Shear Strength of Concrete,(t_c) without Shear Reinforcement:

As per Clause 10.3.2 of IRC:112-2011,
Design shear resistance of the member without shear reinforcement is given by:

$$V_{Rd,c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w\, d \quad \textbf{eq.1}$$

Subjected to minimum of

$$V_{Rd,c} = (V_{\min} + 0.15\sigma_{cp})b_w\, d \quad \textbf{eq.2}$$

where,

K = 1 + SQRT(200/d) ≤ 2.0 , where d is depth in mm

K = 1.52

vmin = $0.031\ K^{3/2}\ fck^{1/2}$, fck = 30.00 N/mm^2

Hence **vmin= 0.318 N/mm^2**

σ_{cp} = Concrete compressive stress in concrete at centroidal axis in the direction of axial load or prestressing

σ_{cp} = N_{Ed}/A_c < 0.2 f_{cd} where , f_{cd} = 0.67 fck/1.5

σ_{cp} = 0.00 N/mm^2

Hence,

τ_c = $V_{Rd,c}/(b_w.d)$ = $V_{min} + 0.15\ \sigma_{cp}$ = 0.3182 N/mm^2 **From eq.2**

ρ1 = Steel Ratio = Asl/(bw . d) ≤ 0.02

Hence ρ1 = 0.0044

τ_c = $V_{Rd,c}/(b_w.d)$ = 0.399 N/mm^2 **From eq.1**

Max of eq.1 & eq.2

τ_c = $V_{Rd,c}/(b_w.d)$ = 0.399 N/mm^2	Corresponds to steel ratio = 0.436% & M30 Grade of Concrete

Shear stress(v_{Ed}) = $V_{Ed}/(bw*d)$

v_{Ed} = $\frac{275145.78}{1000.00 \times 740.00}$ = 0.372 N/mm^2 < 0.399 MPa

AsTv is lesser than Tc Hence No Shear Reinforcement is need to be provided.

CHECK FOR PUNCHING AROUND VEHICLE LOAD (as per IRC:112-2011, Clause 10.4.3):

Maximum load on each wheel	=	100 KN	(70 RW Boggie load)
Maximum tyre pressure	=	5.273 kg/cm^3	
Contact width perpendicular to span, L	=	0.86 m	
Contact width parallel to span,B	=	0.190 m	

Basic Equation for Punching shear stress(v_{Ed}) = $\frac{\beta\ V_{Ed,req}}{ui * d}$

Depth of Slab , d		=	740.00 mm		
Length of perimeter, ui		=	pi() * 4 d + L * 2 + B * 2 for Rectangular section	=	11398.40 mm
Loaded area under perimeter		=	pi() * (4 d)^2/4 + L * 2d*2 + B * 2d*2 + L * B =		1.02E+07 mm2
ΔVEd		=	0.00 for deck slab		
$V_{Ed,req}$	= V_{Ed} - ΔV_{Ed}	=	100.00 kN.		
β		=	1.00 for axial load without bending		

v_{Ed} = 0.01 N/mm^2

ULS DESIGN OF SOLID SLAB

Governing Punching Shear Resistance of Concrete VRd,c = As per IRC:112-2011, Clause 10.4.4

$$v_{Rd.c} = \frac{0.18}{\gamma_c} K\left(80\rho_1 f_{ck}\right)^{1/3} + \left(0.1\sigma_{cp}\right) \geq v_{\min} + 0.1\sigma_{cp}$$

where,

K	=	1.52				
ρ1	=	0.0044				
fck	=	30.00	N/mm^2			
σ_{cp}	=	0.00	N/mm^2			
γc	=	1.50				
Vmin	=	$0.031*k^{3/2}*fck^{1/2}$		=	0.318	N/mm^2
Vmin + 0.1 σcp	=	0.318				
$v_{Rd,c}$	**=**	**0.399**	**N/mm^2**	**OK**		

SINCE	**v_{Ed}**	<	**$v_{Rd,c}$**	**HENCE SAFE IN PUNCHING**	

SLS DESIGN OF DECK SLAB

1. Early Age Check (before creep has taken place)
2. Long Term Check (after creep has taken place)

Maximum compressive stress in concrete under rare combination	0.48 . fck	=	14.4	N/mm^2
Maximum tensile stress in steel under rare combination		=	300	N/mm^2
Maximum Tensile stress in concrete	fctm	=	2.50	N/mm^2
Permissible crack width		=	0.3	mm

Section Properties of Uncracked Section:

Width of Section , b	=	1000.00	mm
Depth of Section , D	=	800.00	mm
Gross Cross sectional area , Ag	=	800000	mm^2
Gross Moment of Inertia Igxx	=	4.27E+10	mm^4
Gross Moment of Inertia Igyy	=	6.67E+10	mm^4

Modular Ratio : for Early Age Check

Modulus of elasticity of concrete, Eceff = Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2
Modular Ratio (m) = Es/ Eceff	=	6.41	

Modular Ratio: for Long Term Check

Perimeter of section (u)	=	3600	mm
2 Ac/u	=	444.444	mm
Modulus of elasticity of concrete, Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2

For Moist atmosperic condition

Creep coeff. ø = 1.30

$$Eceff = \frac{Ecm}{(1 + ø)} = 1.35E+04$$

Modular Ratio (m) = Es/ Eceff = **14.77**

STRESS CHECK IN CRACKED SECTION

Depth of neutral axis =

$$dc = \frac{-A_s E_s + \sqrt{(A_s E_s)^2 + 2 b A_s E_s E_{c,eff} d}}{b E_{c,eff}}$$

Cracked Second Moment of Area I_{NA} (in steel units) =

$$I_{NA} = A_s (d - d_c)^2 + \frac{1}{3}\frac{E_{c,eff}}{E_s} b d_c^3$$

		Rare Case		Quasi-Permanent Case	
		For Early Age	For Long Term Check		
	unit	Sagging Moment	Sagging Moment	Sagging Moment	Remarks
SLS Moment	**kNm/m**	**511.237**	**511.237**	**269.072**	
Width of Section, b	mm	1000.00	1000.00	1000.00	
Depth of section , D	mm	800.00	800.00	800.00	
Effective cover , C_{eff}	mm	60.00	60.00	60.00	
Effective depth , d	mm	740.00	740.00	740.00	
Eceff	N/mm^2	31186.57	13537.58	31186.57	
Es	N/mm^2	200000.00	200000.00	200000.00	
Flexural Ast Provided , As	mm^2/m	3224.27	3224.27	3224.27	
dc		155.48	222.12	155.48	
Cracked Second Moment of area , I_{NA}	mm^4	1296976443.04	1112006587.06	1296976443.04	
section modulus, Zt = I_{NA} / dc	mm^3	8341981.95	5006315.33	8341981.95	
section modulus, Zb = I_{NA} / (d-dc)	mm^3	2218858.48	2147231.46	2218858.48	
Maximum compressive stress in concrete= M/Zt x	N/mm2	9.56	6.91	5.03	**< 14.4 SAFE**
Maximum Tensile stress in steel = M/Zb	N/mm2	230.41	238.09	121.27	**< 300 SAFE**

SLS DESIGN OF DECK SLAB

CRACK WIDTH CHECK Refer Clause 12.3.4 of IRC:112-2011

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

Where, Sr max = maximum crack spacing
εsm = mean strain in the reinforcement
εcm = mean strain in concrete between cracks

Spacing between reinf. = 5*(c+ϕ/2)	mm	300	
Spacing provided	mm	100	
Check for spacing criteria		OK	
$S_{r\,max} =$	3.4 c	+	$\frac{0.425\,k1\,k2\,\phi}{\rho_{\rho\,eff}}$
Clear Cover , c	mm	50	
Diameter of Main Bar , ϕ	mm	20	
Coefficient , k_1		0.8	
Coefficient , k_2		0.5	
Width of section , b	mm	1000	
Depth of section , D	mm	800	
Effective Depth of Section , d	mm	740	
Depth of Neutral axis , yt=dc	mm	155.476	
hc eff = Min of 2.5 (D - d) ,D - dc/3 , D/2	mm	150.00	
Ac, eff = b * hc,eff	mm^2	150000	
$\rho_{\rho\,eff} = As/A_{c\,eff}$		0.021	
Sr max	mm	328.176	
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\dfrac{\sigma_{sc} - \dfrac{k_t f_{ct\,eff}(1 + \alpha_e \rho_{\rho\,eff})}{\rho_{\rho\,eff}}}{E_s}$	
Stress in tension steel , σsc (in Quasi-Permanent Case)	**N/mm2**	**121.27**	
Kt		0.500	
Tensile strength of concrete = fct eff = fctm	N/mm2	2.501	
a_e = Es/ Ecm		6.413	
$(\varepsilon_{sm} - \varepsilon_{cm})$		0.000	
Crack Width , Wk	**mm**	**0.09**	
Limited Carck width	**mm**	**0.30**	
Check for Crack width		**< 0.3 mm SAFE**	

DESIGN OF WALL TYPE ABUTMENT WITH OPEN FOUNDATION

Applicable For Following Bridges

1) Minor Bridge at Ch. 159+083
2) Minor Bridge at Ch. 159+297
3) Minor Bridge at Ch. 163+289

Details of Superstructure:

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

Radius of Curvature of Superstructure	=	0	m
Design speed of vehicle	=	100	kmph

		Right Dimensions	Skew Dimensions
Span -c/c of Brg.	=	9.580m	9.580m
Thickness of Expansion Joint	=	0.020m	0.020m
Slab projection Beyond C/L of Bearing (Back Side)	=	0.200m	0.200m
Slab projection Beyond C/L of Bearing (Span Side)	=	0.200m	0.200m
Span -c/c of E.J.	=	10.000m	10.00m
Type of Superstructure	=	RCC SOLID SLAB	
Width of Crash barrier (Both Side)	=	0.500m	
Width of Carriageway	=	8.000m	
Projection beyond crash barrier	=	0.000m	
Thickness of Wearing coat	=	0.065m	
Length of Approach Slab (Right)	=	3.500m	3.500m
Width of Footpath on both side	=	1.500m	
Railing/kerb on footpath edge	=	0.500m	
Total Width of Superstructure	=	11.000m	
Median Width minus 20mm gap	=	0.480m	

Bearings

Type of Bearing	=	Tar Paper Bearing
Coeff. Of Friction for POT-PTFE Bearing	=	0.5

Type of Soil	=	1	Hard or Rocky Strata
NBC of soil -Normal Case	=	250	kN/m^2 (as per geotechnical report with ground improvement)
SBC of soil-Normal Case	=	280	kN/m^2
SBC of soil-Seismic Case	=	350	kN/m^2
Coeff. of friction between concrete and soil	=	0.7	for weathered rock
Permissible FOS against Sliding	=	1	Normal Case
	=	1	Seismic Case
Permissible FOS against Overturning	=	1	Normal Case
	=	1	Seismic Case

Dirt Wall

		Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	=	0.300m	0.300m
Width of Dirt wall at Bottom	=	0.300m	0.300m
Height of Uniform portion	=	0.600m	
Height of Trapering portion	=	0.141m	
Length of Dirt Wall at top (Uniform portion)	=	11.240m	11.240m
Length of Dirt Wall at bottom (Tapering Portion)	=	11.240m	11.240m

Abutment Cap

Width of Abutment cap of Uniform portion	=	0.720m	0.720m
Width of Abutmentcap at bottom of Tapering Portion	=	0.720m	0.720m
Projection of Abutment Cap (Span Side)	=	0.000m	0.000m
Projection of Abutment Cap Back Side	=	0.000m	0.000m
Abutmentcap thickness (Uniform portion)	=	0.300m	
Abutmentcap thickness (Tapering Portion)	=	0.000m	
Length of Abutment Cap at top (Uniform portion)	=	11.240m	11.240m
Length of Abutment Cap at bottom (Tapering Portion)	=	11.240m	11.240m

Abutment- Wall Type

Thickness of Abutment	=	0.720m	
Width of abutment shaft	=	11.240m	11.240m
Thickness of Abutment shaft at Top	=	0.720m	0.720m
Thickness of Abutment shaft at HFL	=	0.811m	0.811m
Thickness of Abutment shaft at Bottom	=	0.900m	0.900m

Solid Return Wall

Length of Return wall	=	3.600m
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	=	0.600m
Height of wall at abutment	=	2.667m
Length of Return wall	=	4.001m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation

Along Traffic Direction:

Total Width of Footing	=	7.000m
abutment pedestal width	=	0.900m
abutment pedestal Height	=	0.000m
Width of Toe Slab	=	2.500m
Width of Heel Slab	=	3.600m
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	=	3.600m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	6.823m
Height of back fill at back fill edge of heel slab	=	6.123m

Across Traffic Direction:

Width of foundation -Uniform portion	=	11.240m (skew dimension)
Width of foundation -Tapering portion	=	11.240m (skew dimension)

Sr. No.	Structure	Chainage	FRL	GRL	FND. LVL	FRL-FND.LVL
1	Minor Bridge	159+083	1822.54	1818.677	1815.677	6.863
2	**Minor Bridge**	**159+297**	**1816.499**	**1812.251**	**1809.251**	**7.248**
3	Minor Bridge	163+289	1758.838	1754.9	1751.9	6.938

Levels

Deck Level at Median Edge=	**1816.499m**	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	**1816.249m**	Height of Superstructure	=	0.800m
Deck level at center line =	1816.499m	Min. Height of Footpath Side Pedestal (1)	=	0.000m
Soffit Level at center of bridge =	1815.634m	Height of Pedestal (2)	=	0.000m
Abutment cap top level =	1815.633m	Height of Pedestal (3)	=	0.000m
Abutment cap bottom lvl (uniform portion ends	1815.333m	Height of Pedestal (4)	=	0.000m
Abutment cap bottom lvl (corbel portion ends)	1815.333m	Distance of nearest girder to c.l. of deck	=	0.000m
Abutment shaft top level =	1815.333m	Height (Avg.) of Dirt Wall	=	0.741m
Ground level/LBL =	1812.251m	Abutment shaft Above G.L	=	3.082m
Abutment shaft bottom level =	1810.251m	Abutment Shaft below G.L	=	2.000m
Foundation level =	1809.251m	Height of abutment shaft	=	5.082m
HFL	1812.751m	MSL	=	1812.251m
		Wedge over girder flange	=	0.0020m

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 Characteistic Strength, εc2	=	0.02		
Ultimate Strain, εcu2	=	0.035		
Ecm	=	32308.250	N/mm^2	
Steel Grade	=	Fe	500D	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00	Mpa	
Dry weight of Concrete	=	25	kN/m^3	
Dry unit weight of soil	=	20	kN/m^3	
Permissible Crack Width	=	0.3	mm - For Moderrate/ severe Exposure Condition	
Maximum compressive stress in concrete under rare combination	=	0.48 fck		
	=	16.8	N/mm^2	
Maximum tensile stress in steel under rare combination	=	300	N/mm^2	

Creep Coefficient

For Abutment Shaft	=	1.2	for 365 days
For Footing	=	1.2	for 365 days

Clear Cover to Reinforcement

Earth Face	=	75	mm
Non-Earth Face	=	50	mm

Seismic Data: NO NEED TO CHECK FOR SEISMIC EFFECT

Seismic Zone		=	5	
Z	=Zone factor	=	0	
I	=Importance factor	=	1.2	
R	=Response Reduction factor	=	3	in Longitudinal direction
		=	1	In Transverse direction

Properties of backfill material :

c	=	0
ϕ	=	30
θ	=	90
β	=	0
δ	=	20.0

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.160m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.120m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

Self weight of Slab	=	0.80	x	10.00	x	11.00	x	25.00
	=	2200.00	KN					
Reaction at one end	=	1100.00	KN					
Transverse Eccentricity	=	0.000	m					

Super Imposed Dead Load Reactions (Excluding Wearing Course)

Weight of Crash barrier	=	2	x	8.00	x	10.00
	=	160.00	KN			
Reaction at one end	=	80.00	KN			
Transverse Eccentricity	=	0.00	m			

Reaction Due to Wearing Course only

Weight due to Wearing Coat	=	2.2	x	10	x	11
	=	242	KN			
Reaction at one end	=	121	KN			
Transverse Eccentricity	=	0.00	m			

Carriageway Live Load Reactions

Reduction Factor = **0.9** (for 3 Lane)

Congestion factor = **1** (As per Table 3 of IRC :112-2014)

MAXIMUM REACTION CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.33	1.94

Min CWLL

Vertical	Transverse ecc
466.27	2.36

SV Loading

Max CWLL

Vertical	Transverse ecc
2561.34	0.30

Min CWLL

Vertical	Transverse ecc
858.66	0.30

MAXIMUM TRASVERSE MOMENT CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.33	1.94

Min CWLL

Vertical	Transverse ecc
466.27	2.36

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for Cl A Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

VOLUME CALCULATION

C.G. Of Footing = 3.500 m
C.G. Of shaft from toe tip = 2.950 m
Distance between c.g. of shaft and footing = 0.550 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	11.24	0.300	0.600	2.023	-0.210	0.340	-3.160	0.000
-Trapering portion	1	11.24	0.300	0.141	0.475	-0.210	0.340	-3.160	0.000
Bracket (Rectangle)	1	11.24	0.300	0.300	1.012	-0.510	0.040	-3.460	0.000
(Corbel) 0.5	1	11.24	0.300	0.300	0.506	-0.460	0.090	-3.410	0.000
Cap (uniform portion)	1	11.24	0.720	0.300	2.428	0.000	0.550	-2.950	0.000
Cap (Corbel Portion)	1	11.24 11.24	0.720 0.720	0.000	0.000	0.000	0.550	-2.950	0.000
Shaft above HFL	1	11.24	0.766	2.582	22.223	0.067	0.617	-2.883	0.000
Shaft below HFL	1	11.24	0.856	2.200	21.160	0.022	0.572	-2.928	0.000
Solid Return Wall	2	3.60	0.500	6.598	23.753	-2.250	-1.700	-5.200	0.000
Cantilever Return wall(Rectangular portion)	2	4.00	0.500	0.600	2.400	-2.450	-1.900	-5.400	0.000
Cantilever Return wall(Traingular portion)	2	4.00	0.500	2.067	4.135	-1.784	-1.234	-4.734	0.000
Footing									
Heel Slab	1	11.24	3.600	0.650	26.302		-1.377	-4.877	0.000
Toe Slab	1	11.24	2.500	0.650	18.265		2.026	-1.474	0.000
Portion between Heel and Toe	1	11.24	0.900	1.000	10.116		0.550	-2.950	0.000
Back filling above HFL over Heel Slab	1	11.24	3.600	3.748	151.659		-1.700	-5.200	0.000
Back filling below HFL over Heel Slab	1	11.24	3.600	2.850	115.322		-1.774	-5.274	0.000
Backfill above Heel slab	1	11.24	3.600	6.473	261.923		-1.732	-5.232	0.000
Front Filling over Toe Slab	1	11.24	2.500	2.350	66.035		2.188	-1.312	0.000
Side filling between heel and toe	1	0.00	0.900	2.350	0.000		0.000	0.000	0.000
Aproach Slab	1	11.240	1.750	0.300	5.901	-0.510	0.040	-3.460	0.000
Back fill above HFL on flared portion of stem	1	11.24	0.091	3.748	3.853		0.170	-3.330	0.000
Back fill below HFL on flared portion of stem	1	11.24	0.089	2.850	2.837		0.130	-3.370	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barri	2	8 kN/m	1.750	28.00kN		-0.210	0.340	-3.160

SECTIONAL PROPERTIES

Width of Footing (B) = 7 m
Length of Footing (L) = 11.240 m

A = 7.000 x 11.240 = 78.680 m^2
ZL = 11.240 x 8.167 = 91.793 m^3
ZT = (IT1 + IT2) / distance of extreme point from centre

IT1 = 7.000 x 118.336 = 828.35 m^4
IT2 (moment of inertia of triangle) = 7.000 x 0.000 + 0.500 x 7.000 x 0.000 x 31.584
from centre of footing = 0.000 m^4

Moment of inertia of two triangle = 0.000 m^4

Total moment of inertia = 828.35 m^4

Distance of extreme point from centre of footing = 5.620 + 0.000 = 5.620 m

Total Section modulus (ZT) = 147.394 m^3

Load Factors (As per IRC:6-2014)

Table 3.1 Partial Safety Factor For Verification of Equilibrium

-Refer Table 3.1 of IRC:6-2014

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Laod,SIDL & Backfill except wearing course	1.050	0.950	1.050	0.950
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.500	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table 3.3 Partial Safety Factor For Verification of Servicibilty Limit State

-Refer Table 3.3 of IRC:6-2014

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Laod+SIDL including wearing course	1.000	1.00	1.00
wearing course	1.200	1.20	1.20
Back Filling Weight	1.000	1.00	1.00
Shrinkage Creep Effect	1.000	1.00	1.00
Earth Pressure due to back filling	1.000	1.000	1.000
CWLL and Associate load and FPLL	1.000	0.750	0.000
Live Load Surcharge	0.800	0.00	0.00

Table 3.4 Partial Safety Factor For Design of Foundation

-Refer Table 3.4 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Possible Load Combination

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

Seismic Coefficient Calculation

(As Per IRC:6-2014 , Clause 219)

Horizontal Seismic Force For Zone 5.0

Feq	=	Seismic forces to be resisted
Feq	=	Ah x (Dead load + Appropriate Live load)
Ah	=	horizontal seismic coefficient

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

Z	=	Zone factor	=	0	
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.

$$= 2.0 \left(\frac{D}{1000F}\right)^{1/2}$$

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

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

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction
= 6.382 m

C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction
= 6.959 m

Abutment Cap Top Level - Foundation Level
= 6.382 m

Dimensions of Abutment Shaft

Length	=	11.24	m
Width	=	0.81	m
Moment of Inertia , $I_{longitudinal}$	=	0.498	m^4
Moment of Inertia , $I_{transverse}$	=	95.852	m^4
Ecm	=	3.231E+07	kN/m^2

Longitudinal Direction				**Transverse Direction**			
Force	=	92.806	KN	Force	=	15735.321	KN
D	=	1301.00	KN	D	=	1580.720	KN
T	=	0.2368	sec	T	=	0.0200	sec

Hard or Rocky Strata

Sa/g	=	2.5	Sa/g	=	2.5

Seismic Coeff. In Longitudinal Direction = 0

Seismic Coeff. In Transverse Direction = 0

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

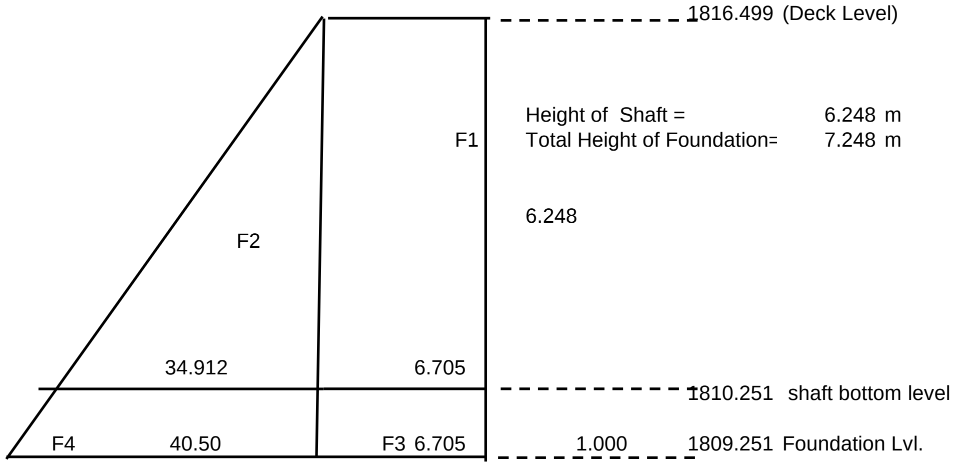
For Design of Foundation (35% increment in Seismic Coeff for Foundation as per IRC:6-2014,Clause No. 219.8)

Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0		
	ϕ	=	30 degree	0.524	radians
	θ	=	90.00 degree	1.571	radians
	$\theta 1$	=	90.00 degree	1.571	radians
	β	=	0	0	radians
	δ	=	20.0 degree	0.349	radians
	Kah	=	0.279 active component		
	Kph	=	3.766 Passive component		
	γ	=	20 kN/m^3		

Equivalent Live Load Surcharge height = 1.2 m
Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1810.251 m (at Shaft Base)
@ RL 1809.251 m (at Foundation Level)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.2	=	6.705	kN/m^2
F1	=	6.705	x	6.248	x	11.240	=	**470.890**	**kN**
M1	=	470.89	x	3.12	=	**1471.061**	**kN.m**	**at Shaft Bottom**	
F3	=	6.705	x	7.248	x	11.240	=	**546.257**	**kN**
M3	=	546.257	x	3.624	=	**1979.634**	**kN.m**	**at Foundation**	

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)

	=	0.279	x	20	x	6.248	=	34.912	kN/m^2
F2	=	0.5	x	34.91	x	6.248	x	11.24	
	=	**1225.884 kN**							

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

M2	=	1225.88	x	2.62	=	**3216.915**	**kN.m**	**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	0.279	x	20	x	7.248	=	40.499	kN/m^2
F4	=	0.5	x	40.50	x	7.248	x	11.24	
	=	**1649.695 kN**							
M4	=	1649.69	x	3.04	=	**5021.935**	**kN.m**	**at Foundation**	

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014 γ fluid = 4.8 kN/m^3

Intensity for triangular portion (At Shaft bottom level)

	=	4.800	x	6.248	=	29.990 kN/m^2			
F	=	0.5	x	29.990	x	6.248	x	11.240	
	=	**1053.076 kN**							

M	=	1053.08	x	2.083	=	**2193.206 kN.m**		**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	4.800	x	7.248	=	34.79 kN/m^2		
F	=	0.5	x	34.790	x	7.25	x	11.240
	=	**1417.144 kN**						
M	=	1417.14	x	2.416	=	**3423.819 kN.m**		**at Foundation**

Intensity of Passive pressure

= 3.766 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 11.24

= **0.000 kN**

Moment due to passive @ Foundation, M

= 0.000 x 0.000 = **0.000 kN.m** **at Foundation**

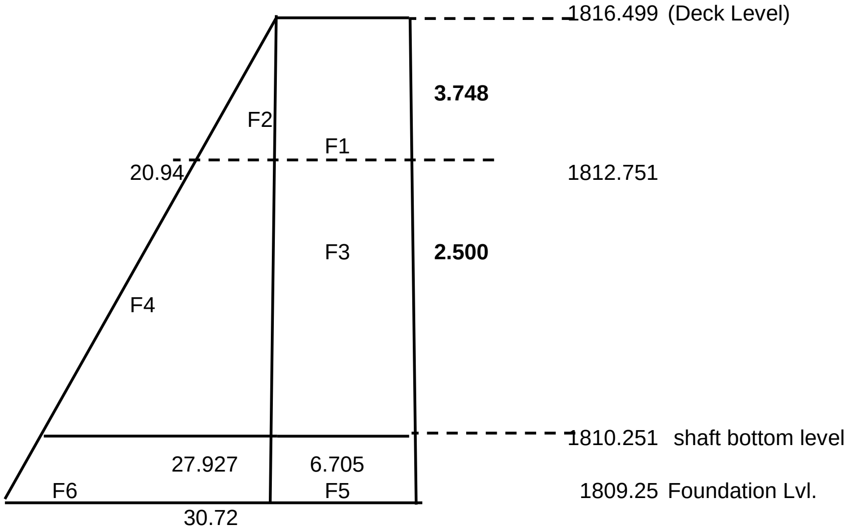
Summary of Moment and Horizontal Force

	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	3216.915	5021.935	1225.884	1649.695
Due to Minimum Fluid Pressure	2193.206	3423.819	1053.076	1417.144
Governing of Two	**3216.915**	**5021.935**	**1225.884**	**1649.695**
Due to Live Load Surcharge	**1471.061**	**1979.634**	**470.890**	**546.257**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0		
	ϕ	=	30 degree	0.524	radians
	θ	=	90.00 degree	1.571	radians
	β	=	0	0	radians
	δ	=	20.0 degree	0.349	radians
	Kah	=	0.279 active component		
	Kph	=	3.766 passive component		
	γd	=	20 kN/m^3		
	γwater	=	10 kN/m^3		
Equivalent Live Load Surcharge height		=	1.2 m		

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1810.3 m (at Shaft Base)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.200	=	6.705 kN/m^2
F1	=	6.705	x	6.248	x	11.240	=	**470.890 kN**
M1	=	470.89	x	3.12	=	**1471.061 kN.m**		**at Shaft Bottom**
F3	=	6.705	x	7.248	x	11.240	=	**546.257 kN**
M3	=	546.26	x	3.62	=	**1979.634 kN.m**		**at Foundation Level**

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.279	x	20	x	3.748	=	20.943 kN/m^2
(At Shaft bottom level) Below HFL	=	0.279	x	10	x	2.500	=	6.985 kN/m^2
F2	=	0.5	x	20.94	x	3.748	x	11.24
	=	**441.130 kN**						

$$F4 = \frac{(20.94 + 27.93)}{2} \times 2.50 \times 11.24$$

= **686.620 kN**

Total Force = 1127.750 kN

M2	=	441.13	x	4.07	=	**1797.234 kN.m**
M4	=	686.62	x	1.19	=	**817.387 kN.m**

Total Mome = 2614.62 kN.m at Shaft Bottom

Intensity for

triangular portion

Upto HFL = 0.279 x 20 x 3.748 = 20.943 kN/m^2

at Foundation level = 0.279 x 10 x 3.500 = 9.778 kN/m^2

F2 = 0.5 x 20.94 x 3.748 x 11.24
= **441.130 kN**

F6 = $\frac{(20.94 + 30.72)}{2}$ x 3.50 x 11.24

= **1016.223 kN**

Total Force = 1457.353 kN

M2 = 441.13 x 5.07 = **2238.363 kN.m**

M6 = 1016.22 x 1.64 = **1666.192 kN.m**

Total Mome = 3904.56 kN.m Foundation Lvl.

Intensity of Passive pressure:
= 3.766 x 10 x 0.00 = 0.000 kN/m^2
Force due to passive @ Foundation, F
= 0.5 x 0.000 x 0.00 x 11.24
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m** **Foundation Lvl.**

Summary of Moment and Horizontal Force

	MOMENTS		**HORIZONTAL FORCE**	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	2614.620	3904.555	1127.750	1457.353
Due to Minimum Fluid Pressure	2193.206	3423.819	1053.076	1417.144
Governing of Two	**2614.620**	**3904.555**	**1127.750**	**1457.353**
Due to Live Load Surcharge	**1471.061**	**1979.634**	**470.890**	**546.257**
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

$$C_a = \frac{\text{Cos}^2(\phi - \lambda - \alpha)\,\text{Cos}\,\delta \qquad (1 \pm \alpha v)}{\text{Cos}^2\alpha\,\text{Cos}(\alpha + \delta + \lambda)\,\text{Cos}\,\lambda\,[1 + \text{sqrt}(\text{Sin}(\phi + \delta)\,\text{Sin}(\phi - \beta - \lambda)\,/\,(\text{Cos}(\alpha + \delta + \lambda)\,\text{Cos}(\alpha - \beta)))]^2}$$

α h	=	0.000	
α v	=	0.000	
φ	=	30.00	0.524
δ	=	20.00	0.349
α	=	0.00	0.000
β	=	0.00	0.000

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

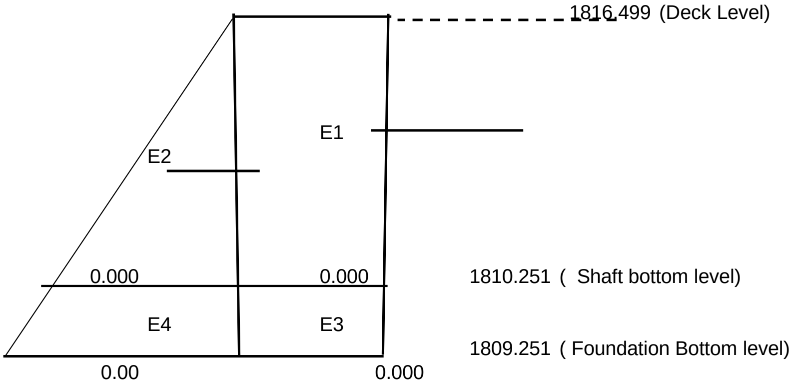
$\lambda = \tan^{-1}\dfrac{\alpha h}{(1 \pm \alpha v)}$ = 0.000, 0.000

		1	2
C_a	=	0.279	0.279

Ca	=	0.279		
Ka	=	**0.279**		
Dynamic Increment	=	0.279	-0.279	**0.000**

3 Earth Pressure : **DRY CASE (Seismic case)**

Equivalent Live Load Surcharge height		=	1.2 m
Assuming	γdry	=	20 kN/m^3
	γwater	=	10.00 kN/m^3



Earth Pressure Diagram for Dynamic Increment

Horizontal Forces and Moments @ RL **1810.3 m (at Shaft Base)**
1809.3 m (at Foundation Bottom Level)

Due to Dynamic Live Load Surcharge

	=	0.000	x	20	x	1.2	=	0.000	kN/m^2
at Shaft Bottom Level									
E1	=	0.000	x	6.248	x	11.240	=	**0.000**	kN
M1	=	0.000	x	4.186			=	**0.000**	kN.m
at Foundation Bottom Level									
E3	=	0.000	x	7.248	x	11.240	=	**0.000**	kN
M3	=	0.000	x	4.856			=	**0.000**	kN.m

Due to Dynamic Active Earth Pressure

(At Shaft bottom level)									
	=	0.000	x	20	x	6.248	=	0.000	kN/m^2
(at Foundation Bottom Level)									
	=	0.000	x	20	x	7.248	=	0.000	kN/m^2
E2	=	0.50	x	0.00	x	6.25	x	11.240	
	=	**0.000**	kN						

E4	=	0.50	x	0.00	x	7.25	x	11.240
	=	**0.000**	kN					
M2	=	0.00	x	3.12	=	**0.000**	kN.m	**(Shaft bottom level)**
M4	=	0.00	x	3.62	=	**0.000**	kN.m	**(Foundation Bottom level)**

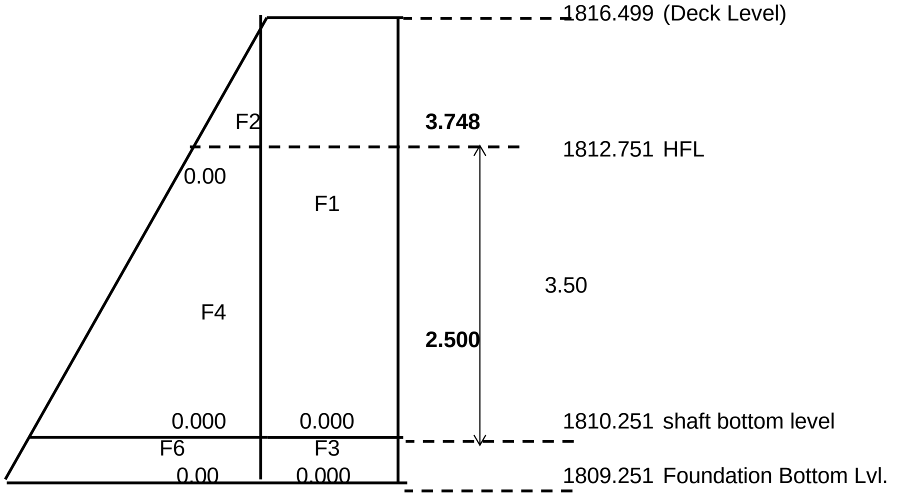
Summary of Moment and Horizontal Force **Dry Seismic Case**

	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)	3216.915	5021.935	1225.884	1649.695
Due to active Earth Pressure (dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	3216.915	5021.935	1225.884	1649.695
Due to Minimum Fluid Pressure	2193.206	3423.819	1053.076	1417.144
Governing of Two	**3216.915**	**5021.935**	**1225.884**	**1649.695**
Due to Live Load Surcharge (Static)	**1471.061**	**1979.634**	**470.890**	**546.257**
Due to Live Load Surcharge(Dynamic)	**0.000**	**0.000**	**0.000**	**0.000**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Dynamic Increment	=	0.000
γd	=	20 kN/m^3
γwater	=	10 kN/m^3
Equivalent Live Load Surcharge height	=	1.2 m

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL **1810.251 m (at Shaft Base)**
1809.251 m (at Foundation Bottom Level)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.000	x	20	x	1.200	=	0.000 kN/m^2
at Shaft Bottom Level								
F1	=	0.000	x	6.248	x	11.240	=	**0.000 kN**
M1	=	0.00	x	4.12	=	**0.000 kN.m**		
at Foundation Bottom Level								
F3	=	0.000	x	7.248	x	11.240	=	**0.000 kN**
M3	=	0.00	x	4.78	=	**0.000 kN.m**		

Due to Dynamic Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.000	x	20	x	3.748	=	0.000 kN/m^2
(At Shaft bottom level) Below HFL	=	0.000	x	10	x	2.500	=	0.000 kN/m^2
(At Foundati bottom level) Below HFL	=	0.000	x	10	x	3.500	=	0.000 kN/m^2

F2 = 0.5 x 0.00 x 3.75 x 11.24
= **0.000 kN**

F4 = $\frac{(0.00 + 0.00)}{2}$ x 2.50 x 11.24
= **0.000 kN**

F6 = $\frac{(0.00 + 0.00)}{2}$ x 3.50 x 11.24
= **0.000 kN**

Total Force (F2 + F4)	=	**0.000 kN**	**at Shaft Bottom Level**
Total Force (F2 + F6)	=	**0.000 kN**	**at Foundation Bottom Level**

M2	=	0.00	x	4.37	=	**0.000 kN.m**
M4	=	0.00	x	0.00	=	**0.000 kN.m**

Total Mome = **0.000 kN.m** **at Shaft Bottom**

M2	=	0.00	x	5.37	=	**0.000**	**kN.m**
M6	=	0.00	x	0.00	=	**0.000**	**kN.m**

Total Mome = **0.000 kN.m** **at Foundation Bottom Level**

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom Lvl kN	At Foundation Bottom Lvl kN
Due to active Earth Pressure(Static	2614.620	3904.555	1127.750	1457.353
Due to active Earth Pressure (Dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	2614.620	3904.555	1127.750	1457.353
Due to Minimum Fluid Pressure	2193.206	3423.819	1053.076	1417.144
Governing of Two	**2614.620**	**3904.555**	**1127.750**	**1457.353**
Due to Live Load Surcharge(Stat	**1471.061**	**1979.634**	**470.890**	**546.257**
Due to Live Load Surcharge (Dynamic Increment)	**0.000**	**0.000**	**0.000**	**0.000**
Due to passive pressure		**0.000**		**0.000**

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1100.00	1.35	1.35	1485.00	1485.00	
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00	
WC	=	121.00	1.75	1.75	211.75	211.75	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmin	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmin	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmax-Transv. Moment Case		0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A

Braking Force = 0.2 x 1000 + 0.05 x 554

= 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1804.75	0	902.375	902.375		
DL+SIDL+LL-Max Reaction case	1804.75	341.55	902.375	902.375	1- 70RW + 2-CLASS A	Dry Case
	1804.75	0	902.375	902.375	SV Loading	
DL+SIDL+LL-Min Reaction case	1804.75	341.55	902.375	902.375	1- 70RW + 2-CLASS A	HFL Case
	1804.75	0	902.375	902.375	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1804.75	341.55	902.375	902.375		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	1804.75	0.00	902.375	902.375	
DL+SIDL+LL-Max Reaction case		1804.75	45.54	902.375	902.375	Dry Case
DL+SIDL+LL-Min Reaction case		1804.75	45.54	902.375	902.375	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1804.75	45.54	902.375	902.375	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	1804.75	0.000	902.375	902.375	
DL+SIDL+LL-Max Reaction case		1804.75	45.540	902.375	902.375	Dry Case
DL+SIDL+LL-Min Reaction case		1804.75	45.540	902.375	902.375	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1804.75	45.540	902.375	902.375	

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1100.00	1.35	1.35	1485.00	1485.00
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00
WC	=	121.00	1.75	1.75	211.75	211.75
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1.5	0.75	0.00	0.00
CWLLmin	=	0.00	1.5	0.75	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1804.75	0.000	902.375	902.375		
DL+SIDL+LL-Max Reaction case	1804.75	341.550	902.375	902.375	1- 70RW + 2-CLASS A	Dry Case
	1804.75	0.000	902.375	902.375	SV Loading	
DL+SIDL+LL-Min Reaction case	1804.75	341.550	902.375	902.375	1- 70RW + 2-CLASS A	HFL Case
	1804.75	0.000	902.375	902.375	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1804.75	341.550	902.375	902.375		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	1804.75	0.00	902.375	902.375	
DL+SIDL+LL-Max Reaction case		1804.75	45.54	902.375	902.375	Dry Case
DL+SIDL+LL-Min Reaction case		1804.75	45.54	902.375	902.375	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1804.75	45.54	902.375	902.375	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	1804.75	0.000	902.375	902.375	
DL+SIDL+LL-Max Reaction case		1804.75	45.540	902.375	902.375	Dry Case
DL+SIDL+LL-Min Reaction case		1804.75	45.540	902.375	902.375	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1804.75	45.540	902.375	902.375	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1100.00	1	1.00	1100.00	1100.00
SIDL except wc	=	80.00	1	1.00	80.00	80.00
WC	=	121.00	1	1.00	121.00	121.00
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1	0.20	0.00	0.00
CWLLmin	=	0.00	1	0.20	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1301.00	0.000	650.500	650.500		
DL+SIDL+LL-Max Reaction case	1301.00	227.700	650.500	650.500	1- 70RW + 2-CLASS A	Dry Case
	1301.00	0.000	650.500	650.500	SV Loading	
DL+SIDL+LL-Min Reaction case	1301.00	227.700	650.500	650.500	1- 70RW + 2-CLASS A	HFL Case
	1301.00	0.000	650.500	650.500	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1301.00	227.700	650.500	650.500		

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	0.000	650.500	650.500	
DL+SIDL+LL-Max Reaction case	1301.00	45.540	650.500	650.500	Dry Case
DL+SIDL+LL-Min Reaction case	1301.00	45.540	650.500	650.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1301.00	45.540	650.500	650.500	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1301.00	0.000	650.500	650.500	
DL+SIDL+LL-Max Reaction case	1301.00	45.540	650.500	650.500	Dry Case
DL+SIDL+LL-Min Reaction case	1301.00	45.540	650.500	650.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1301.00	45.540	650.500	650.500	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1100.00	1.05	1.05	1155.00	1155.00
SIDL except wc	=	80.00	1.05	1.05	84.00	84.00
WC	=	121.00	1.35	1.35	163.35	163.35
FPLL	=	0.00	1.5	0.00	0.00	0.00

CWLLmax-Reaction case	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1.5	0.00	0.00	0.00
CWLLmin	=	0.00	1.5	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	µ R	Max (Fh/2 or µR)		
DL+SIDL	1402.35	0.000	701.175	701.175		
DL+SIDL+LL-Max Reaction case	1402.35	341.550	701.175	701.175	1- 70RW + 2-CLASS A	Dry Case
	1402.35	0.000	701.175	701.175	SV Loading	
DL+SIDL+LL-Min Reaction case	1402.35	341.550	701.175	701.175	1- 70RW + 2-CLASS A	HFL Case
	1402.35	0.000	701.175	701.175	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1402.35	341.550	701.175	701.175		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	µ R	Max (Fh/2 or µR)	
DL+SIDL	1301.00	1402.35	0.00	701.175	701.175	
DL+SIDL+LL-Max Reaction case		1402.35	0.00	701.175	701.175	Dry Case
DL+SIDL+LL-Min Reaction case		1402.35	0.00	701.175	701.175	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1402.35	0.00	701.175	701.175	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	µ R	Max (Fh/2 or µR)	
DL+SIDL	1301.00	1402.35	0.000	701.175	701.175	
DL+SIDL+LL-Max Reaction case		1402.35	0.000	701.175	701.175	Dry Case
DL+SIDL+LL-Min Reaction case		1402.35	0.000	701.175	701.175	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1402.35	0.000	701.175	701.175	

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	=	1100.00	1	1	1	1100.00	1100.00	1100.00
SIDL except wc	=	80.00	1	1	1	80.00	80.00	80.00
WC	=	121.00	1.20	1.20	1.20	145.20	145.20	145.20
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmin	=	0.00	1	0.75	0	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1325.20	0.000	662.600	662.600		
DL+SIDL+LL-Max Reaction case	1325.20	227.700	662.600	662.600	1- 70RW + 2-CLASS A	Dry Case
	1325.20	0.000	662.600	662.600	SV Loading	
DL+SIDL+LL-Min Reaction case	1325.20	227.700	662.600	662.600	1- 70RW + 2-CLASS A	HFL Case
	1325.20	0.000	662.600	662.600	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1325.20	227.700	662.600	662.600		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1325.20	0.000	662.600	662.600	
DL+SIDL+LL-Max Reaction case	1325.20	170.775	662.600	662.600	Dry Case
DL+SIDL+LL-Min Reaction case	1325.20	170.775	662.600	662.600	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1325.20	170.775	662.600	662.600	

Normal Case: Quasi Permenant Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1325.20	0.000	662.600	662.600	
DL+SIDL+LL-Max Reaction case	1325.20	0.000	662.600	662.600	Dry Case
DL+SIDL+LL-Min Reaction case	1325.20	0.000	662.600	662.600	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1325.20	0.000	662.600	662.600	

Centrifugal Force Calculation

As per clause 212 of IRC:6-2014

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

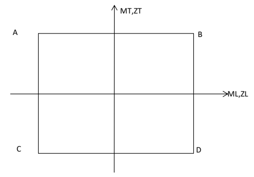
			Normal Case		Seismic Case	
Design Speed	V	=	100.00	kmph	100.00	kmph
Live Load	W	=	932.33	kN	932.33	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1809.251 m

Properties of Footing Base:

A	=	78.680	m^2
ZL	=	91.793	m^3
ZT	=	147.394	m^3



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

Moment along longitudinal axis	ML = ML Cos θ + MT Sin θ
Moment along transverse axis	MT = MT Cos θ - ML Sin θ

Case 1 : DL+SIDL-Normal Dry Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure												
Dead Load	1			1100.000	0.710	781.000	0.000	0.000	0.950	1045.000	-2.790	-2915.550
SIDL except Wearing Course	1			80.000	0.710	56.800	0.000	0.000	0.950	76.000	-2.790	-212.040
Wearing Course	1			121.000	0.710	85.910	0.000	0.000	1.000	121.000	-2.790	-337.590
				1301.000		**923.710**		**0.000**		**1242.000**		**-3465.180**
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	1	25	2.023	50.580	0.340	17.197	0.000	0.000	0.950	48.051	-3.160	-151.841
Dirt Wall-Tapered portion	1	25	0.475	11.886	0.340	4.041	0.000	0.000	0.950	11.292	-3.160	-35.683
Bracket - Uniform portion	1	25	1.012	25.290	0.040	1.012	0.000	0.000	0.950	24.026	-3.460	-83.128
Bracket - Tapered portion	1	25	0.506	12.645	0.090	1.138	0.000	0.000	0.950	12.013	-3.410	-40.963
Cap - (uniform portion)	1	25	2.428	60.696	0.550	33.383	0.000	0.000	0.950	57.661	-2.950	-170.101
Cap - (corbel portion)	1	25	0.000	0.000	0.550	0.000	0.000	0.000	0.950	0.000	-2.950	0.000
Cantilever Return Wall-Rectangle po	1	25	2.400	60.008	-1.900	-114.029	0.000	0.000	0.950	57.007	-5.400	-307.853
Cantilever Return Wall-Traingle porti	1	25	4.135	103.363	-1.234	-127.498	0.000	0.000	0.950	98.195	-4.734	-464.805
RCC Railing or Crash Barrier	1			28.000	0.340	9.520	0.000	0.000	0.950	26.600	-3.160	-84.056
Aproach Slab	1	25	5.901	147.525	0.040	5.901	0.000	0.000	0.950	140.149	-3.460	-484.915
				499.993		**-169.335**		**0.000**		**474.993**		**-1823.344**
Substructure & Foundation -Portion 2												
Solid Return wall	1	25	23.753	593.820	-1.700	-1009.494	0.000	0.000	0.950	564.129	-5.200	-2933.471
Abutment Shaft	1	25	43.383	1084.576	0.572	620.112	0.000	0.000	0.950	1030.347	-2.928	-3017.110
Back filling over heel slab	1	20	261.923	5238.469	-1.732	-9075.347	0.000	0.000	0.950	4976.546	-5.232	-26039.490
Front Filling over toe slab	1	20	66.035	1320.700	2.188	2889.617	0.000	0.000	0.950	1254.665	-1.312	-1646.192
Side filling between heel and toe	1	20	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.000	0.000	0.000
Heel slab	1	25	26.302	657.540	-1.377	-905.382	0.000	0.000	0.950	624.663	-4.877	-3046.433
Toe slab	1	25	18.265	456.625	2.026	924.958	0.000	0.000	0.950	433.794	-1.474	-639.568
portion between heel & toe	1	25	10.116	252.900	0.550	139.095	0.000	0.000	0.950	240.255	-2.950	-708.752
Vertical Components of active earth pressure	1			600.440	-3.500	-2101.540	0.000	0.000	0.950	570.418	-7.000	-3992.925
				10338.855		**-8497.963**		**0.000**		**9821.912**		**-42447.200**
Total				**12139.847**		**-7743.588**		**0.000**		**11538.905**		**-47735.725**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1649.695		5021.935	1649.69	5021.94	0.00	0.00
					2300.195	9174.077	0.000	0.000

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2474.542		7532.903	2474.54	7532.90	0.00	0.00
		3175.717		12008.503	3175.717	12008.503	0.000	0.000

Summary of Forces For SBC

P	12139.847	KN
ML	1430.489	kNm
MT	0.000	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Forces from Case :DL+SIDL		12139.847		-7743.588		0.000		11538.905		-47735.72
CWLL-Max. Reaction case	1	932.327	0.710	661.952	1.943	1811.433	0.000		-2.790	0.00
Vertical Components of LL Surcharge	1	198.821	-3.500	-695.874	0.000	0.000	0.950		-7.000	-1322.16
Total		**13270.995**		**-7777.510**		**1811.433**		**11538.905**		**-49057.89**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1649.695		5021.935	1649.69	5021.94	0.00	0.00
due to Live load surcharge	1	546.257		1979.634	546.257	1979.634	0.000	0.000
					2846.45	**11153.71**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2474.542		7532.903	2474.54	7532.90	0.00	0.00
due to Live load surcharge	1.2	655.508		2375.561	655.51	2375.56	0.00	0.00
		3831.225		**14384.064**	**3831.225**	**14384.064**	**0.000**	**0.000**

Summary of Forces For SBC

P	13270.995	KN
ML	3376.200	kNm
MT	1811.433	kNm

Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Forces from Case :DL+SIDL		12139.847		-7743.588		0.000		11538.90486		-47735.72482
CWLL-Max. Reaction case	1	2561.336	0.710	1818.549	0.300	768.401	0.000		-2.790	0.00
Vertical Components of LL Surcharge	1	198.821	-3.500	-695.874	0.000	0.000	0.950		-7.000	-1322.16
Total		**14900.004**		**-6620.914**		**768.401**		**11538.905**		**-49057.89**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1649.695		5021.935	1649.69	5021.94	0.00	0.00
due to Live load surcharge	1	546.257		1979.634	546.257	1979.634	0.000	0.000
					2846.45	**11153.71**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
					Forces along Long. Axis		Forces along Trans. Axis	
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2474.542		7532.903	2474.54	7532.90	0.00	0.00
due to Live load surcharge	1.2	655.508		2375.561	655.51	2375.56	0.00	0.00
		3831.225		**14384.064**	**3831.225**	**14384.064**	**0.000**	**0.000**

Summary of Forces For SBC

P	14900.004	KN
ML	4532.797	kNm
MT	768.401	kNm

Case 3 : DL+SIDL-Normal HFL Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

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)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Superstructure				**1301.000**		**923.710**		**0.000**		**1242.000**		**-3465.180**
Substructure & Foundation -Portion 1				**499.993**		**-169.335**		**0.000**		**474.993**		**-1823.344**
Substructure & Foundation -Portion 2												

Solid Return wall	1	25	23.753	593.820	-1.700	-1009.494	0.000	0.000	0.950	564.129	-5.200	-2933.471
Shaft above HFL	1	25	22.223	555.566	0.617	342.608	0.000	0.000	0.950	527.788	-2.883	-1521.781
Shaft below HFL	1	15	21.160	317.406	0.572	181.478	0.000	0.000	0.950	301.536	-2.928	-882.970
Back filling above HFL over heel slab	1	20	151.659	3033.181	-1.700	-5156.408	0.000	0.000	0.950	2881.522	-5.200	-14983.916
Back filling below HFL over heel slab	1	10	115.322	1153.224	-1.774	-2045.455	0.000	0.000	0.950	1095.563	-5.274	-5777.652
Front Filling over toe slab	1	10	66.035	660.350	2.188	1444.808	0.000	0.000	0.950	627.333	-1.312	-823.096
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.000	0.000	0.000
Heel slab	1	15	26.302	394.524	-1.377	-543.229	0.000	0.000	0.950	374.798	-4.877	-1827.860
Toe slab	1	15	18.265	273.975	2.026	554.975	0.000	0.000	0.950	260.276	-1.474	-383.741
Portion between Heel & Toe	1	15	10.116	151.740	0.550	83.457	0.000	0.000	0.950	144.153	-2.950	-425.251
Vertical Components of active earth pressure	1			530.433	-3.500	-1856.516	0.000	0.000	0.950	503.912	-7.000	-3527.381
				7769.639		**-7987.024**		**0.000**		**7381.157**		**-33421.721**
Total				**9570.631**		**-7232.650**		**0.000**		**9098.150**		**-38710.246**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1457.353		3904.555	1457.35	3904.56	0.00	0.00
					2107.85	**8056.70**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2186.030		5856.833	2186.03	5856.83	0.00	0.00
		2887.205		**10332.433**	**2887.20**	**10332.43**	**0.00**	**0.00**

Summary of Forces For SBC

P	9570.631	KN
ML	824.047	kNm
MT	0.000	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Forces from Case : DL+SIDL		9570.631		-7232.650		0.000		9098.150		-38710.246
CWLL-Min. Reaction case	1	466.273	0.710	331.054	2.359	1099.889	0.000	0.000	-2.790	0
Vertical Components of LL Surcharge	1	198.821	-3.500	-695.874	0.000	0.000	0.950	188.880	-7.000	-1322.161
Total		**10235.726**		**-7597.470**		**1099.889**		**9287.030**		**-40032.406**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1457.353		3904.555	1457.35	3904.56	0.00	0.00
due to Live load surcharge	1	546.257		1979.634	546.257	1979.634	0.000	0.000
					2654.11	**10036.33**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2186.030		5856.833	2186.03	5856.83	0.00	0.00
due to live load surcharge	1.2	655.508		2375.561	655.508	2375.561	0.000	0.000
		3542.713		**12707.994**	**3542.71**	**12707.99**	**0.00**	**0.00**

Summary of Forces For SBC

P	10235.726	KN
ML	2438.861	kNm
MT	1099.889	kNm

Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case

Vertical Forces For SBC Calculation / **Vertical Forces For Restoring or Resisting Effect**

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
Forces from Case : DL+SIDL		9570.631		-7232.650		0.000		9098.150		-38710.246
CWLL-Min. Reaction case	1	858.664	0.710	609.651	0.300	257.599	0.000	0.000	-2.790	0.00
Vertical Components of LL Surcharge	1	198.821	-3.500	-695.874	0.000	0.000	0.950	188.880	-7.000	-1322.161
Total		**10628.116**		**-7318.872**		**257.599**		**9287.030**		**-40032.406**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure	1	650.500	1815.634	4152.142	650.50	4152.14	0.00	0.00
due to Earth pressure	1	1457.353		3904.555	1457.35	3904.56	0.00	0.00
due to Live load surcharge	1	546.257		1979.634	546.257	1979.634	0.000	0.000
					2654.11	**10036.33**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		701.175	1815.634	4475.600	701.18	4475.60	0.00	0.00
due to Earth pressure	1.5	2186.030		5856.833	2186.03	5856.83	0.00	0.00
due to live load surcharge	1.2	655.508		2375.561	655.508	2375.561	0.000	0.000
		3542.713		**12707.994**	**3542.71**	**12707.99**	**0.00**	**0.00**

Summary of Forces For SBC

P	10628.116	KN
ML	2717.458	kNm
MT	257.599	kNm

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

Seismic Effect Factor = 1

αh= 0.000 In Longitudinal direction
αh= 0.000 In Transverse direction
αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 453.608 KN
Weight of back fill below Ground level = 1618.560 KN

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 = 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			1100.000		0.000	0.000	0.710	781.000	0.000	1816.167		0.000	0.000	0.000
SIDL except Wearing Course	1			80.000		0.000	0.000	0.710	56.800	0.000	1816.948		0.000	0.000	0.000
Wearing Course	1			121.000		0.000	0.000	0.710	85.910	0.000	1816.499		0.000	0.000	0.000
				1301.000		**0.000**	**0.000**		**923.710**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	2.023	50.580	0.000	0.000	0.000	0.340	17.197	0.000	1816.199	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.475	11.886	0.000	0.000	0.000	0.340	4.041	0.000	1815.829	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1	25	1.012	25.290				0.040	1.012						
Bracket - Tapered portion	1	25	0.506	12.645				0.090	1.138						
Cap - (uniform portion)	1	25	2.428	60.696	0.000	0.000	0.000	0.550	33.383	0.000	1815.483	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1815.333	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1	25	2.400	60.008	0.000	0.000	0.000	-1.900	-114.029	0.000	1816.199	0.000	0.000	0.000	0.000
Cantilever Return Wall-Trainqle porti	1	25	4.135	103.363	0.000	0.000	0.000	-1.234	-127.498	0.000	1815.210	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.340	9.520				0.000	0.000	
Aproach Slab	1	25	5.901	147.525				0.040	5.901				0.000	0.000	
				499.993	**0.000**	**0.000**	**0.000**		**-169.335**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	23.753	593.820	0.000	0.000	0.000	-1.700	-1009.494	0.000	1813.263	0.000	0.000	0.000	0.000
Abutment Shaft	1	25	43.383	1084.576	0.000	0.000	0.000	0.572	620.112	0.000	1813.792	0.000	0.000	0.000	0.000
Back filling over heel slab	1	20	261.923	5238.469	0.000	0.000	0.000	-1.732	-9075.347	0.000	1813.263	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1	20	66.035	1320.700				2.188	2889.617				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	26.302	657.540				-1.377	-905.382				0.000	0.000	
Toe slab	1	25	18.265	456.625				2.026	924.958				0.000	0.000	
portion between heel & toe	1	25	10.116	252.900				0.550	139.095				0.000	0.000	
Vertical component of active earth pressure	1			600.440				-3.500	-2101.540				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				10338.855	**0.000**	**0.000**	**0.000**		**-8497.963**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**12139.847**	**0.000**	**0.000**	**0.000**		**-7743.588**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure								
Dead Load	0.95			1045.000	0.000	-2.790	-2915.550	0.000

For Overturning or Sliding Effect

Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL

SIDL except Wearing Course	0.95			76.000	0.000	-2.790	-212.040	0.000				
Wearing Course	1.00			121.000	0.000	-2.790	-337.590	0.000				
				1242.000	**0.000**		**-3465.180**	**0.000**				
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	0.95	25	2.023	48.051	0.000	-3.160	-151.841	0.000	1.0	0.000	1816.199	0.000
Dirt Wall-Tapered portion	0.95	25	0.475	11.292	0.000	-3.160	-35.683	0.000	1.0	0.000	1815.829	0.000
Bracket - Uniform portion	0.95	25	1.012	24.026								
Bracket - Tapered portion	0.95	25	0.506	12.013								
Cap - (uniform portion)	0.95	25	2.428	57.661	0.000	-2.950	-170.101	0.000	1.0	0.000	1815.483	0.000
Cap - (corbel portion)	0.95	25	0.000	0.000	0.000	-2.950	0.000	0.000	1.0	0.000	1815.333	0.000
Cantilever Return Wall-Rectangle po	0.95	25	2.400	57.007	0.000	-5.400	-307.853	0.000	1.0	0.000	1816.199	0.000
Cantilever Return Wall-Traingle porti	0.95	25	4.135	98.195	0.000	-4.734	-464.805	0.000	1.0	0.000	1815.210	0.000
RCC Railing or Crash Barrier	0.95			26.600		-3.160	-84.056					
Aproach Slab	0.95	25	5.901	140.149		-3.460	-484.915					
				474.993	**0.000**		**-1699.253**	**0.000**		**0.000**		**0.000**
Substructure & Foundation -Portion 2												
Abutment Shaft	0.95	25	43.383	1030.347	0.000	-2.928	-3017.110	0.000	1.0	0.000	1813.792	0.000
Solid Return wall	0.95	25	23.753	564.129	0.000	-5.200	-2933.471	0.000	1.0	0.000	1813.263	0.000
Back filling over heel slab	0.95	20	261.923	4976.546	0.000	-5.232	-26039.490	0.000	1.0	0.000	1813.263	0.000
Front Filling over Pile Cap	0.95	20	66.035	1254.665		-1.312	-1646.192					
Side filling between heel and toe	0.95	20	0.000	0.000		0.000	0.000					
Heel slab	0.95	25	26.302	624.663		-4.877	-3046.433					
Toe slab	0.95	25	18.265	433.794		-1.474	-639.568					
portion between heel & toe	0.95	25	10.116	240.255		-2.950	-708.752					
Vertical component of active earth pressure	0.95			570.418		-7.000	-3992.925					
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000					
				9821.912	**0.000**		**-42447.20**	**0.000**		**0.000**		**0.000**
Total =				**11538.905**	**0.000**		**-47611.63**	**0.000**		**0.000**		**0.000**
					0.000			0.000				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1649.695			5021.935	

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
650.50	4152.14	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	0.00
1649.69	5021.94						
2300.19	**9174.08**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.60
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	2474.542		7532.90
due to dynamic Earth pressure	1.5	0.000		0.00
		3175.717		**12008.503**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2474.542	7532.903	0.000	0.000
0.000	0.000	0.000	0.000
3175.72	**12008.50**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	12139.847	12139.847	KN
ML	1430.489	1430.489	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	11538.905	kN
Moment	-47611.633	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 = 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 Superstucture				1301.000		0.000	0.000		923.710	0.000				0.000	0.000
Forces from Substructure				10838.847	0.000	0.000	0.000		-8667.298	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			186.47		0.000	0.000	0.710	132.390	0.000	1817.699		1.943	362.287	0.000
Vertical component of LL Surcharge	0.20			39.76				-3.500	-139.175				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**12366.077**	**0.000**	**0.000**	**0.000**		**-7750.373**	**0.000**		**0.000**		**362.287**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstucture				1242.000	0.000	0.000	-3465.18	0.000
Forces from Substructure				10296.905	0.000	0.000	-44146.45	0.000
CWLL-Max. Reaction case	0.00			0.00	0.000	-2.790	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-7.000	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.00	
Total =				**11538.905**	**0.000**		**-47611.63**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1649.695			5021.935	
due to Live load surcharge	0.20	109.251			395.927	

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
650.50	4152.14	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	0.00
1649.69	5021.94						
109.25	395.93						
2409.45	**9570.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.600
due to Substructure	1.5	0.000		0.000
due to Active Earth pressure	1.5	2474.542		7532.903
due to dynamic Earth pressure	1.5	0.000		0
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		3175.717		**12008.503**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2474.542	7532.903		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3175.72	**12008.50**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	12366.077	12366.077	KN
ML	1819.631	1819.631	kNm
MT	362.287	362.287	kNm

Summary of Restoring Forces

Vertical Load	11538.905	kN
Moment	-47611.633	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 αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1301.000**		**0.000**	**0.000**		**923.710**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**499.993**	**0.000**	**0.000**	**0.000**		**-169.335**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1		23.75	593.82	0.000	0.000	0.000	-1.70	-1009.494	0.000	1813.26	0.000	0.00	0.000	0.000
Shaft above HFL	1	25	22.223	555.566	0.000	0.000	0.000	0.617	342.608	0.000	1814.042	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	21.160	317.406	0.000	0.000	0.000	0.572	181.478	0.000	1812.501	0.000	0.000	0.000	0.000
Back filling above HFL over heel slab	1	20	151.659	3033.181	0.000	0.000	0.000	-1.700	-5156.408	0.000	1814.625	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	115.322	1153.224	0.000	0.000	0.000	-1.774	-2045.455	0.000	1811.251	0.00	0.000	0.000	0.00
Front Filling over Pile Cap	1	10	66.035	660.350				2.188	1444.808				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	26.302	394.524				-1.377	-543.229				0.000	0.000	
Toe slab	1	15	18.265	273.975				2.026	554.975				0.000	0.000	
portion between heel & toe	1	15	10.116	151.740				0.550	83.457				0.000	0.000	
Vertical component of active earth pressure	1			530.433				-3.500	-1856.516				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				7769.639	**0.000**	**0.000**	**0.000**		**-8003.777**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**9570.631**	**0.000**	**0.000**	**0.000**		**-7249.402**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**1242.000**	**0.000**		**-3465.180**	**0.000**
Substructure & Foundation -Portion 1				**474.993**	**0.000**		**-1699.253**	**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	0.95	25	23.7528	564.129	0.000	-5.20	-2933.471	0.000
Shaft above HFL	0.95	25	22.223	527.788	0.000	-2.883	-1521.781	0.000
Shaft below HFL	0.95	15	21.160	301.536	0.000	-2.928	-882.970	0.000
Back filling above HFL over heel slab	0.95	20	151.659	2881.522	0.000	-5.200	-14983.916	0.000
Back filling below HFL over heel slab	0.95	10	115.322	1095.563	0.000	-5.274	-5777.652	0.000
Front Filling over Pile Cap	0.95	10	66.035	627.333		-1.312	-823.096	
Side filling between heel and toe	0.95	10	0.000	0.000		0.000	0.000	
Heel slab	0.95	15	26.302	374.798		-4.877	-1827.860	
Toe slab	0.95	15	18.265	260.276		-1.474	-383.741	
portion between heel & toe	0.95	15	10.116	144.153		-2.950	-425.251	
Vertical component of active earth pressure	0.95			503.912		-7.000	-3527.381	
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000	
				7381.157	**0.000**		**-33421.721**	**0.000**
Total =				**9098.150**	**0.000**		**-38586.15**	**0.000**
					0.000			0.000

For Overturning or Sliding Effect

Loads	Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure				
Substructure & Foundation -Portion 1		**0.000**		**0.000**
Substructure & Foundation -Portion 2				
Solid Return wall	1.0	0.000	1813.263	0.000
Shaft above HFL	1.0	0.000	1814.042	0.000
Shaft below HFL	1.0	0.000	1812.501	0.000
Back filling above HFL over heel slab	1.0	0.000	1814.625	0.000
Back filling below HFL over heel slab	1.0	0.000	1811.251	0.000
		0.000		**0.000**
Total =		**0.000**		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1457.353			3904.555	

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
650.50	4152.14	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	0.00
1457.35	3904.56						
2107.85	**8056.70**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.600
due to Substructure	1.5	0.000		0.000
due to Active Earth pressure	1.5	2186.030		5856.833
due to dynamic Earth pressure	1.5	0.000		0.000
		2887.205		**10332.433**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2186.030	5856.833		
0.000	0.000		
2887.20	**10332.43**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	9570.631	9570.631	KN
ML	807.295	807.295	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	9098.150	kN
Moment	-38586.154	kNm

Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				1301.000		0.000	0.000		923.710	0.000				0.000	0.000
Forces from Substructure				8269.631	0.000	0.000	0.000		-8173.112	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			93.25		0.000	0.000	0.710	66.211	0.000	1817.699		2.359	219.978	0.000
Vertical component of LL Surcharge	0.20			39.76				-3.500	-139.175				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**9703.650**	**0.000**	**0.000**	**0.000**		**-7322.366**	**0.000**		**0.000**		**219.978**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				1242.000	0.000		-3465.180	0.000
Forces from Substructure				7856.150	0.000		-35120.974	0.000
CWLL-Max. Reaction case	0.00			0.00	0.00		-2.79	0.00
Vertical component of LL Surcharge	0.00			0.00		-2.790	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.000	0.000
Total =				**9098.150**	**0.000**		**-38588.94**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1457.353			3904.555	
due to Live load surcharge	0.20	109.251			395.927	

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
650.50	4152.14	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	0.00
1457.35	3904.56						
109.25	395.93						
2217.10	**8452.62**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.600
due to Substructure	1.5	0.000		0
due to Active Earth pressure	1.5	2186.030		5856.833
due to dynamic Earth pressure	1.5	0		0
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		2887.205		**10332.433**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2186.030	5856.833		
0.000	0.000		
0.000	0.000		
0.000	0.000		
2887.20	**10332.43**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	9703.650	9703.650	KN
ML	1130.257	1130.257	kNm
MT	219.978	219.978	kNm

Summary of Restoring Forces

Vertical Load	9098.150	kN
Moment	-38588.944	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 αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1301.000**		**0.000**	**0.000**		**923.710**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**499.993**	**0.000**	**0.000**	**0.000**		**-169.335**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2				**10338.855**	**0.000**	**0.000**	**0.000**		**-8497.963**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**12139.847**	**0.000**	**0.000**	**0.000**		**-7743.588**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure				**1242.000**	**0.000**		**-3465.18**	**0.000**
Substructure & Foundation -Portion 1				**474.993**	**0.000**		**-1699.25**	**0.000**
Substructure & Foundation -Portion 2				**9821.912**	**0.000**		**-42447.20**	**0.000**
Total =				**11538.905**	**0.000**		**-47611.63**	**0.000**
					0.000			0.000

For Overturning or Sliding Effect

Loads	Load Factor	FL = 0.3 x αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure				
Substructure & Foundation -Portion 1		**0.000**		**0.000**
Substructure & Foundation -Portion 2		**0.000**		**0.000**
Total =		**0.000**		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1649.695			5021.935	

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
650.50	4152.14	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	0.00
1649.69	5021.94						
2300.19	**9174.08**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.600
due to Substructure	1.5	0.000		0
due to Active Earth pressure	1.5	2474.542		7532.903126
due to dynamic Earth pressure	1.5	0.000		0
		3175.717		**12008.503**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2474.542	7532.903		
0.000	0.000		
3175.72	**12008.50**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	12139.847	12139.847	KN
ML	1430.489	1430.489	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	11538.905	kN
Moment	-47611.633	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**12366.077**	**0.000**	**-7750.37**	**0.000**	**362.287**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL2	MLs due to Fv
Total =	**11538.905**	**0.000**	**-47611.63**	**0.000**
		0.000		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0.000	1815.634	4152.142	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	1649.695			5021.935	
due to Live load surcharge	0.2	109.251			395.927	

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
650.50	4152.14	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	0.00
1649.69	5021.94						
109.25	395.93						
2409.45	**9570.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.18	1815.634	4475.600
due to Substructure	1.5	0.00		0.000
due to Active Earth pressure	1.5	2474.54		7532.903
due to dynamic Earth pressure	1.5	0.00		0.000
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		3175.72		**12008.503**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2474.542	7532.903		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3175.72	**12008.50**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	12366.077	12366.077	KN
ML	1819.631	1819.631	kNm
MT	362.287	362.287	kNm

Summary of Restoring Forces

Vertical Load	11538.905	kN
Moment	-47611.633	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure		**1301.000**	**0.000**	**923.710**	**0.000**	**0.000**
Substructure & Foundation -Portion 1		**499.993**	**0.000**	**-169.335**	**0.000**	**0.000**
Substructure & Foundation -Portion 2		**7769.639**	**0.000**	**-8003.777**	**0.000**	**0.000**
Total =		**9570.631**	**0.000**	**-7249.402**	**0.000**	**0.000**
			0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure		**1242.000**	**0.000**		**-3465.180**	**0.000**
Substructure & Foundation -		**474.993**	**0.000**		**-1699.253**	**0.000**
Substructure & Foundation -		**7381.157**	**0.000**		**-33421.721**	**0.000**
Total =		**9098.150**	**0.000**		**-38586.154**	**0.000**
				0.000		0.000

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.50	0.00	1815.63	4152.14	0.00
due to Substructure	1	0.00	0.00		0.00	0.00
due to Earth pressure	1	1457.35			3904.56	

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
650.50	4152.14	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	0.00
1457.35	3904.56						
2107.85	**8056.70**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.175	1815.634	4475.60
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	2186.030		5856.83
due to dynamic Earth pressure	1.5	0.000		0.00
		2887.205		**10332.43**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2186.030	5856.833		
0.000	0.000		
2887.20	**10332.43**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	9570.631	9570.631	KN
ML	807.295	807.295	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	9098.150	kN
Moment	-38586.154	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**9703.650**	**0.000**	**-7322.37**	**0.000**	**219.978**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Total =	**9098.150**	**0.000**		**-38588.94**	**0.000**
		0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	650.500	0	1815.634	4152.142	0
due to Substructure	1	0.000	0		0.000	0
due to Earth pressure	1	1457.353			3904.555	
due to Live load surcharge	0.2	109.251			395.927	

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
650.50	4152.14	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	0.00
1457.35	3904.56						
109.25	395.93						
2217.10	**8452.62**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		701.18	1815.634	4475.60
due to Substructure	1.5	0.00		0.00
due to Active Earth pressure	1.5	2186.03		5856.83
due to dynamic Earth pressure	1.5	0.00		0.00
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		2887.20		**10332.43**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
701.18	4475.60	0.00	0.00
0.00	0.00	0.00	0.00
2186.030	5856.833		
0.000	0.000		
0.000	0.000		
0.000	0.000		
2887.20	**10332.43**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	9703.650	9703.650	KN
ML	1130.257	1130.257	kNm
MT	219.978	219.978	kNm

Summary of Restoring Forces

Vertical Load	9098.150	kN
Moment	-38588.944	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 (1817.699 - 1809.251) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.20 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1817.699 - 1809.251) = 0.000 kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

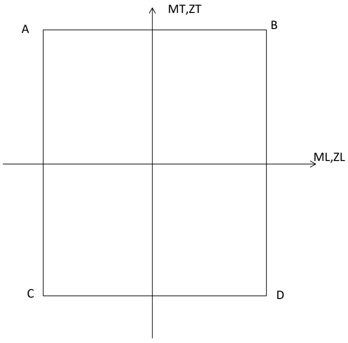
	SAFE BEARING CAPACITY CHECK									SLIDING CHECK			OVERTURNING CHECK		
LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= μP + c.A + Fp	FOS	Overturning moment	Restoring Moment = $\Sigma P.eToe+M_p$	FOS
Normal Dry Case	kN	kNm	kNm	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN/m^2	kN	kN		kNm	kNm	
Case 1 : DL+SIDL-Normal Dry Case	12139.847	1430.489	0.000	138.710	169.878	138.710	169.878	169.878	138.710	3175.717	8077.233	2.54	12008.503	47735.72	3.98
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	13270.995	3376.200	1811.433	144.180	217.741	119.600	193.161	217.741	119.600	3831.225	8077.233	2.11	14384.064	49057.89	3.41
Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case	14900.004	4532.797	768.401	145.208	243.968	134.781	233.542	243.968	134.781	3831.225	8077.233	2.11	14384.064	49057.89	3.41
								SAFE	SAFE			SAFE			SAFE
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	9570.631	824.047	0.000	112.663	130.617	112.663	130.617	130.617	112.663	2887.205	6368.705	2.21	10332.433	38710.25	3.75
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	10235.726	2438.861	1099.889	110.986	164.124	96.062	149.200	164.124	96.062	3542.713	6500.921	1.84	12707.994	40032.41	3.15
Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case	10628.116	2717.458	257.599	107.224	166.432	103.728	162.937	166.432	103.728	3542.713	6500.921	1.84	12707.994	40032.41	3.15
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	12139.847	1430.489	0.000	138.710	169.878	138.710	169.878	169.878	138.710	3175.717	8077.233	2.54	12008.503	47611.63	3.96
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	12366.077	1819.631	362.287	139.804	179.450	134.888	174.534	179.450	134.888	3175.717	8077.233	2.54	12008.503	47611.63	3.96
								SAFE	SAFE			SAFE			SAFE
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	9570.631	807.295	0.000	112.845	130.435	112.845	130.435	130.435	112.845	2887.205	6368.705	2.21	10332.433	38586.15	3.73
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	9703.650	1130.257	219.978	112.510	137.136	109.525	134.151	137.136	109.525	2887.205	6368.705	2.21	10332.433	38588.94	3.73
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	12139.847	1430.489	0.000	138.710	169.878	138.710	169.878	169.878	138.710	3175.717	8077.233	2.54	12008.503	47611.63	3.96
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	12366.077	1819.631	362.287	139.804	179.450	134.888	174.534	179.450	134.888	3175.717	8077.233	2.54	12008.503	47611.63	3.96
								SAFE	SAFE			SAFE			SAFE
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	9570.631	807.295	0.000	112.845	130.435	112.845	130.435	130.435	112.845	2887.205	6368.705	2.21	10332.433	38586.15	3.73
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	9703.650	1130.257	219.978	112.510	137.136	109.525	134.151	137.136	109.525	2887.205	6368.705	2.21	10332.433	38588.94	3.73
								SAFE	SAFE			SAFE			SAFE

DESIGN OF FOUNDATION

Foundation Lvl = 1809.251 m

Properties of Footing Base:

A	=	78.680	m^2
ZL	=	91.793	m^3
ZT	=	147.394	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			1485.000	0.710	1054.350	0.000	0.000
SIDL except Wearing Course	1.35			108.000	0.710	76.680	0.000	0.000
Wearing Course	1.75			211.750	0.710	150.343	0.000	0.000
				1804.750		**1281.373**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.023	68.283	0.340	23.216	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.475	16.047	0.340	5.456	0.000	0.000
Bracket - Uniform portion	1.35	25	1.012	34.142	0.040	1.366	0.000	0.000
Bracket - Tapered portion	1.35	25	0.506	17.071	0.090	1.536	0.000	0.000
Cap - (uniform portion)	1.35	25	2.428	81.940	0.550	45.067	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.550	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1.35	25	2.400	81.010	-1.900	-153.939	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	4.135	139.540	-1.234	-172.123	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.35			37.800	0.340	12.852	0.000	0.000
Aproach Slab	1.35	25	5.901	199.159	0.040	7.966	0.000	0.000
				674.990		**-228.603**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	23.753	801.657	-1.700	-1362.817	0.000	0.000
Abutment Shaft	1.35	25	43.383	1464.178	0.572	837.151	0.000	0.000
Back filling over heel slab	1.35	20	261.923	7071.934	-1.732	-12251.718	0.000	0.000
Front Filling over toe slab	1.35	20	66.035	1782.945	2.188	3900.983	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	26.302	887.679	-1.377	-1222.266	0.000	0.000
Toe slab	1.35	25	18.265	616.444	2.026	1248.694	0.000	0.000
portion between heel & toe	1.35	25	10.116	341.415	0.550	187.778	0.000	0.000
Vertical Components of active earth pressure	1.5			900.660	-3.500	-3152.309	0.000	0.000
				14047.520		**-11787.481**		**0.000**
Total				**16527.260**		**-10734.711**		**0.000**

Summary of Forces About C.G. OF Footing

P	16527.260	KN
ML	2558.052	kNm
MT	0.000	kNm

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	5759.860
due to Earth pressure	1.5	2474.542		7532.903

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	5759.86	0.00	0.00
2474.54	7532.90	0.00	0.00
3376.917	**13292.763**	**0.000**	**0.000**

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		16527.260		-10734.711		0.000
CWLL-Max. Reaction case	1.5	1398.490	0.710	992.928	1.943	2717.150
Vertical Components of LL Surcharge	1.2	238.585	-3.500	-835.049	0.000	0.000
Total		**18164.335**		**-10576.832**		**2717.150**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	5759.860
due to Earth pressure	1.5	2474.542		7532.903
due to Live load surcharge	1.2	655.508		2375.561
		4032.425		**15668.323**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	5759.86	0.00	0.00
2474.54	7532.90		
655.51	2375.56		
4032.425	**15668.323**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	18164.335	KN
ML	5091.491	kNm
MT	2717.150	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				**1804.750**		**1281.373**		**0.000**
Substructure & Foundation -Portion 1				**674.990**		**-228.603**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	23.753	801.657	-1.700	-1362.817	0.000	0.000
Shaft above HFL	1.35	25	22.223	750.015	0.617	462.520	0.000	0.000
Shaft below HFL	1.35	15	21.160	428.498	0.572	244.996	0.000	0.000
Back filling above HFL over heel slab	1.35	20	151.659	4094.795	-1.700	-6961.151	0.000	0.000
Back filling below HFL over heel slab	1.35	10	115.322	1556.852	-1.774	-2761.365	0.000	0.000
Front Filling over toe slab	1.35	10	66.035	891.473	2.188	1950.491	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	26.302	532.607	-1.377	-733.359	0.000	0.000
Toe slab	1.35	15	18.265	369.866	2.026	749.216	0.000	0.000
Portion between Heel & Toe	1.35	15	10.116	204.849	0.550	112.667	0.000	0.000
Vertical Components of active earth pressure	1.5			795.650	-3.500	-2784.774	0.000	0.000
				10568.577		**-11060.960**		**0.000**
Total				**13048.317**		**-10008.191**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	5759.860
due to Earth pressure	1.5	2186.030		5856.833

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	5759.86	0.00	0.00
2186.03	5856.83		
3088.405	**11616.692**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	13048.317	KN
ML	1608.502	kNm
MT	0.000	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		13048.317		-10008.191		0.000
CWLL-Min. Reaction case	1.5	699.410	0.710	496.581	2.359	1649.834
Vertical Components of LL Surcharge	1.2	238.585	-3.500	-835.049	0.000	0.000
Total		**13986.312**		**-10346.658**		**1649.834**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	5759.860
due to Earth pressure	1.5	2186.030		5856.833
due to Live load surcharge	1.2	655.508		2375.561

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	5759.86	0.00	0.00
2186.03	5856.83		
655.51	2375.56		
3743.913	**13992.253**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	13986.312	KN
ML	3645.595	kNm
MT	1649.834	kNm

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

Seismic Effect Factor = 1.50 αh= 0.000 In Longitudinal direction Weight of shaft below Ground level = 453.61 KN
αh= 0.000 In Transverse direction Weight of back fill below Ground level = 1618.56 KN
αv= 0.000 In Vertical direction

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 = 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			1485.000		0.000	0.000	0.710	1054.350	0.000	1816.167		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.000		0.000	0.000	0.710	76.680	0.000	1816.948		0.000	0.000	0.000
Wearing Course	1.75			211.750		0.000	0.000	0.710	150.343	0.000	1816.499		0.000	0.000	0.000
				1804.750		**0.000**	**0.000**		**1281.373**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.023	68.283	0.000	0.000	0.000	0.340	23.216	0.000	1816.199	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.475	16.047	0.000	0.000	0.000	0.340	5.456	0.000	1815.829	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.012	34.142				0.040	1.366						
Bracket - Tapered portion	1.35	25	0.506	17.071				0.090	1.536						
Cap - (uniform portion)	1.35	25	2.428	81.940	0.000	0.000	0.000	0.550	45.067	0.000	1815.483	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1815.333	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1.35	25	2.400	81.010	0.000	0.000	0.000	-1.900	-153.939	0.000	1816.199	0.000	0.000	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	4.135	139.540	0.000	0.000	0.000	-1.234	-172.123	0.000	1815.210	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.340	12.852				0.000	0.000	
Aproach Slab	1.35	25	5.901	199.159				0.040	7.966				0.000	0.000	
				674.990	**0.000**	**0.000**	**0.000**		**-228.603**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	23.753	801.657	0.000	0.000	0.000	-1.700	-1362.817	0.000	1813.263	0.000	0.000	0.000	0.000
Abutment Shaft	1.35	25	43.383	1464.178	0.000	0.000	0.000	0.572	837.151	0.000	1813.792	0.000	0.000	0.000	0.000
Back filling over heel slab	1.35	20	261.923	7071.934	0.000	0.000	0.000	-1.732	-12251.718	0.000	1813.263	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	66.035	1782.945				2.188	3900.983				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	26.302	887.679				-1.377	-1222.266				0.000	0.000	
Toe slab	1.35	25	18.265	616.444				2.026	1248.694				0.000	0.000	
portion between heel & toe	1.35	25	10.116	341.415				0.550	187.778				0.000	0.000	
Vertical component of active earth pressure	1.00			600.440				-3.500	-2101.540						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				13747.300	**0.000**	**0.000**	**0.000**		**-10736.711**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**16227.040**	**0.000**	**0.000**	**0.000**		**-9683.941**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	5759.860	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1649.695			5021.935	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	5759.86	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	0.00
1649.69	5021.94						
0.00	0.00						
2552.07	**10781.80**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	16227.040	16227.040	KN
ML	1097.854	1097.854	kNm
MT	0.000	0.000	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				1804.750		0.000	0.000		1281.373	0.000				0.000	0.000
Forces from Substructure				14422.290	0.000	0.000	0.000		-10965.314	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.75			699.25		0.000	0.000	0.710	496.464	0.000	1817.699		1.943	1358.575	0.000
Vertical component of LL Surcharge	0.20			39.764				-3.500	-139.175						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				**16966.049**	**0.000**	**0.000**	**0.000**		**-9326.652**	**0.000**		**0.000**		**1358.575**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	5759.860	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1	1649.695			5021.935	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	109.251			395.927	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
902.38	5759.86	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	0.00
1649.69	5021.94						
0.00	0.00						
109.25	395.93						
0.00	0.00						
2661.32	**11177.72**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	16966.049	16966.049	KN
ML	1851.070	1851.070	kNm
MT	1358.575	1358.575	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1804.750**		**0.000**	**0.000**		**1281.373**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**674.990**	**0.000**	**0.000**	**0.000**		**-228.603**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	23.753	801.657	0.000	0.000	0.000	-1.700	-1362.817	0.000	1813.263	0.000	0.000	0.000	0.000
Shaft above HFL	1.35	25	22.223	750.015	0.000	0.000	0.000	0.617	462.520	0.000	1814.042	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	15	21.1603948	428.498	0.000	0.000	0.000	0.572	244.996	0.000	1812.501	0.000	0.000	0.000	0.000
Back filling above HFL over heel slab	1.35	20	151.659072	4094.795	0.000	0.000	0.000	-1.700	-6961.151	0.000	1814.625	0.000	0.000	0.000	0.000
Back filling below HFL over heel slab	1.35	10	115.3224	1556.852	0.000	0.000	0.000	-1.774	-2761.365	0.000	1811.251	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	10	66.035	891.473				2.188	1950.491				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	

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL	FT	Fv	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
Heel slab	1.35	15	26.302	532.607				-1.377	-733.359				0.000	0.000	
Toe slab	1.35	15	18.265	369.866				2.026	749.216				0.000	0.000	
portion between heel & toe	1.35	15	10.116	204.849				0.550	112.667				0.000	0.000	
Vertical component of active earth pressure	1.00			530.433				-3.500	-1856.516						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				10303.360	0.000	0.000	0.000		-10132.702	0.000		0.000		0.000	0.000
Total =				12783.101	0.000	0.000	0.000		-9079.933	0.000		0.000		0.000	0.000
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		902.375	0.000	1815.634	5759.860	0.000	902.38	5759.86	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1457.353			3904.555		1457.35	3904.56						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
							2359.73	9664.41	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	12783.101	12783.101	KN
ML	584.482	584.482	kNm
MT	0.000	0.000	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 = 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				1804.750		0.000	0.000		1281.373	0.000				0.000	0.000
Forces from Substructure				10978.351	0.000	0.000	0.000		-10361.305	0.000		0.000		0.000	0.000
CWLL-Min. Reaction case	0.75			349.70		0.000	0.000	0.710	248.291	0.000	1817.699		2.359	824.917	0.000
Vertical component of LL Surcharge	0.20			39.764				-3.500	-139.175						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				13172.570	0.000	0.000	0.000		-8970.817	0.000		0.000		824.917	0.000
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		902.375	0.000	1815.634	5759.860	0.000	902.38	5759.86	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1	1457.353			3904.555		1457.35	3904.56						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
due to Live load surcharge	0.20	109.251			395.927		109.25	395.93						
due to dynamic increment of Surcharge	1.50	0.000			0.000		0.00	0.00						
							2468.98	10060.34	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	13172.570	13172.570	KN
ML	1089.525	1089.525	kNm
MT	824.917	824.917	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = 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				**1804.750**		**0.000**	**0.000**		**1281.373**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**674.990**	**0.000**	**0.000**	**0.000**		**-228.603**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2				**13747.300**	**0.000**	**0.000**	**0.000**		**-10736.711**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				16227.040	0.000	0.000	0.000		-9683.941	0.000		0.000		0.000	0.000
							0.000			0.000					

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		902.375	0.000	1815.634	5759.860	0.000	902.38	5759.86	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure		1649.695			5021.935		1649.69	5021.94						
due to dynamic increment of EP		0.000			0.000		0.00	0.00						
							2552.07	10781.80	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	16227.040	16227.040	KN
ML	1097.854	1097.854	kNm
MT	0.000	0.000	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				16966.049	0.000	-9326.652	0.000	1358.575
					0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		902.375	0.000	1815.634	5759.860	0	902.38	5759.86	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure		1649.695			5021.935		1649.69	5021.94						
due to dynamic increment of EP		0.000			0.000		0.00	0.00						
due to Live load surcharge		109.251			395.927		109.25	395.93						
due to dynamic increment of Surcharge		0.000			0.000		0.00	0.00						
							2661.32	11177.72	0.00	0.00	0.00	0.00	0.00	0.00

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	16966.049	16966.049	KN
ML	1851.070	1851.070	kNm
MT	1358.575	1358.575	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT

Superstructure				1804.750	0.000	1281.373	0.000	0.000
Substructure & Foundation -Portion 1				674.990	0.000	-228.603	0.000	0.000
Substructure & Foundation -Portion 2				10303.360	0.000	-10132.702	0.000	0.000
Total =				**12783.101**	**0.000**	**-9079.933**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	5759.860	0
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1457.353			3904.555	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	5759.86	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	0.00
1457.35	3904.56						
0.00	0.00						
2359.73	**9664.41**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	12783.101	12783.101	KN
ML	584.482	584.482	kNm
MT	0.000	0.000	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**13172.570**	**0.000**	**-8970.817**	**0.000**	**824.917**
		0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	5759.860	0
due to Substructure		0.000	0.000		0.000	0
due to Earth pressure		1457.353			3904.555	
due to dynamic increment of EP		0.000			0.000	
due to Live load surcharge		109.251			395.927	
due to dynamic increment of Surcharge		0.000			0.000	

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
902.38	5759.86	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	0.00
1457.35	3904.56						
0.00	0.00						
109.25	395.93						
0.00	0.00						
2468.98	**10060.34**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	13172.570	13172.570	KN
ML	1089.525	1089.525	kNm
MT	824.917	824.917	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 (1817.699 - 1809.251) = 0.000 kNm

Centrifugal Force : Seismic Case

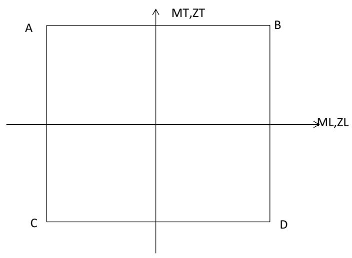
Centrifugal Force (C.F.) = 0.75 x 0.00 = 0.000 KN
Transverse Moment due to C.F. = 0.000 x (1817.699 - 1809.251) = 0.000 kNm

	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

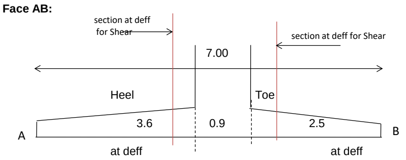
Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT
Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT
Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

Summary of Design Base Pressure

LOAD CASES	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	16527.260	2558.052	0.000	182.189	237.924	182.189	237.924
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	18164.335	5091.491	2717.150	193.831	304.765	156.962	267.896
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	13048.317	1608.502	0.000	148.317	183.363	148.317	183.363
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	13986.312	3645.595	1649.834	149.240	228.671	126.853	206.284
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	16227.040	1097.854	0.000	194.281	218.201	194.281	218.201
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	16966.049	1851.070	1358.575	204.685	245.016	186.251	226.582
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	12783.101	584.482	0.000	156.102	168.837	156.102	168.837
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	13172.570	1089.525	824.917	161.147	184.886	149.954	173.692
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	16227.040	1097.854	0.000	194.281	218.201	194.281	218.201
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	16966.049	1851.070	1358.575	204.685	245.016	186.251	226.582
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	12783.101	584.482	0.000	156.102	168.837	156.102	168.837
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	13172.570	1089.525	824.917	161.147	184.886	149.954	173.692



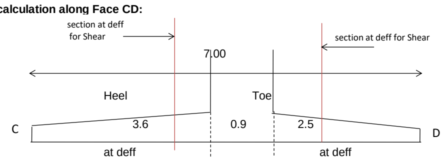
Pressure calculation along Face AB:



	A	at deff			at deff	B
Case : 1	182.189	203.552	210.85	218.019	225.320	237.924
Case : 2	193.831	236.350	250.88	265.146	279.678	304.765
Case : 3	148.317	161.750	166.34	170.847	175.438	183.363
Case : 4	149.240	179.685	190.09	200.303	210.708	228.671
Case : 5	194.281	203.449	206.58	209.658	212.792	218.201
Case : 6	204.685	220.144	225.43	230.612	235.896	245.016
Case : 7	156.102	160.983	162.65	164.289	165.957	168.837
Case : 8	161.147	170.246	173.36	176.407	179.517	184.886
Case : 9	194.281	203.449	206.58	209.658	212.792	218.201
Case : 10	204.685	220.144	225.43	230.612	235.896	245.016
Case : 11	156.102	160.983	162.65	164.289	165.957	168.837
Case : 12	161.147	170.246	173.36	176.407	179.517	184.886

Average MAX Base Pressure for Design of Heel Slab-along Face AB = 222.357 kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face AB = 157.329 kN/m^2
Average MAX Base Pressure for Design of Toe Slab-along Face AB = 284.955 kN/m^2

Pressure calculation along Face CD:



	C	at deff			at deff	D
Case : 1	182.189	203.552	210.85	218.019	225.320	237.924
Case : 2	156.962	199.481	214.01	228.277	242.809	267.896
Case : 3	148.317	161.750	166.34	170.847	175.438	183.363
Case : 4	126.853	157.298	167.70	177.916	188.321	206.284
Case : 5	194.281	203.449	206.58	209.658	212.792	218.201
Case : 6	186.251	201.709	206.99	212.178	217.461	226.582
Case : 7	156.102	160.983	162.65	164.289	165.957	168.837
Case : 8	149.954	159.052	162.16	165.214	168.324	173.692
Case : 9	194.281	203.449	206.58	209.658	212.792	218.201
Case : 10	186.251	201.709	206.99	212.178	217.461	226.582
Case : 11	156.102	160.983	162.65	164.289	165.957	168.837
Case : 12	149.954	159.052	162.16	165.214	168.324	173.692

Average MAX Base Pressure for Design of Heel Slab-along Face CD = 200.432 kN/m^2
Average MIN Base Pressure for Design of Heel Slab-along Face CD = 147.278 kN/m^2
Average Base Pressure for Design of Toe Slab-along Face CD = 248.086 kN/m^2

Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	279.678	kN/m^2	Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	242.809	kN/m^2
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	236.350	kN/m^2	Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	203.552	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	=	222.357 kN/m^2							
Upward moment due to Base pressure	=	1440.873 kNm/m							
		factor							
Downward moment due to backfill	=	1.35 x	261.923	/	11.240	x	20	x	1.800
	=	1132.516 kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	26.302	/	11.240	x	25	x	1.477
	=	116.640 kNm/m							
Net Moment at face of shaft	=	1440.873	-1132.516	-116.640 =	191.717 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	=	147.278 kN/m^2							
Upward moment due to Base pressure	=	954.364 kNm/m							
		factor							
Downward moment due to backfill	=	1.35 x	261.923	/	11.240	x	10	x	1.800
	=	566.258 kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	26.302	/	11.240	x	15	x	1.477
	=	69.984 kNm/m							
Net Moment at face of shaft	=	954.364	-566.258	-69.984 =	318.122 kNm/m		Tension at Bottom of Heel Slab		

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	0.822 m					
effective depth at critical section	=	0.739 m					
Base pressure at deff from face of wall	=	236.350 kN/m^2					
Shear Force due to upward pressure at deff from face of wall	=	220.518 x	2.683 x	11.240	=	6650.140	KN
		factor					
Downward Force due to backfill	=	1.35 x	261.923 x	20	=	7071.934	KN
Downward Force due to self weight of Heel slab	=	1.35 x	26.302 x	25	=	887.679	KN
Net Shear Force	=	6650.140	-7071.934	-887.679	=	-1309.473	KN
Net Shear Force / unit meter	=	-1309.473 /	11.240		=	116.501	KN/m

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	284.955 kN/m^2							
Upward moment due to Base pressure	=	890.485 kNm/m							
Downward moment due to self weight of Toe slab	=	1.35 x	18.265	/	11.240	x	25	x	1.026
	=	56.2500 kNm/m							
Net Moment at face of shaft	=	890.485	-56.250 =		834.235 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	=	0.658 m							
Base pressure at deff from face of wall	=	279.678 kN/m^2							
upward shear force due to base pressure	=	292.222 x	1.583 x		11.240		=	5199.474	KN
C.g. Of base pressure	=	0.679 m							
moment due to upward pressure at critical section	=	3532.551 kNm							
$\tan\beta$	=	0.280							
reduction in shear force (V_{ccd})	=	$\frac{M \tan\beta}{d}$ =	1330.814	KN					
Downward force due to self weight of toe slab	=	1.35 x	0.522	x	1.583	x	11.240	x 25	
	=	313.239 KN							
Net Shear Force at deff	=	5199.474 -	313.239	-	1330.814		=	3555.421	KN
Net Shear Force / unit meter	=	3555.421 /	11.240				=	316.319	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	
fctm	=	2.77 N/mm^2	
Grade of Reinforcement Steel	=	Fe 500D	(HYSD Bars)
fy or fyk	=	500.00 N/mm^2	
fyd	=	434.78 N/mm^2	(fy/1.15)
Es	=	200000.00 N/mm^2	

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	Toe Slab
Ultimate bending moment,Mu (kNm/m)	**=**	**318.12**	**834.24**
Effective depth required (dreq) (mm)	=	233.29	377.79
Effective depth provided (dpro) (mm)	=	917.00	917.00
Check for provided depth	**=**	**SAFE**	**SAFE**
R = Mu/(b d2)	**=**	**0.38**	**0.99**
Total depth provided (mm)	=	1000.00	1000.00
Limiting depth of neutral axis (mm)	=	565.66	565.66
Actual depth of neutral axis (mm)	=	54.20	108.41
Check for Neutral axis depth	**=**	**OK**	**OK**
Lever arm (z) , mm	=	894.45	871.90
Moment of Resistance w.r.to steel	=	610.87	1190.94
Check for Moment Capacity	**=**	**SAFE**	**SAFE**
Ast reqd (mm2 / m)	=	880.83	2360.40
cl. 16.6.1 (2) of IRC :112-2011			
$A_{s,min} = 0.26\, f_{ctm}\, b_t\, d / f_{yk} >= 0.0013\, b_t\, d$	=	1321.45	1321.45
Governing Ast (mm2 / m)	=	1321.45	2360.40
Tension Reinforcement			
Dia (mm)	=	**16.00**	**20.00**
Spacing (mm)	=	**200.00**	**200.00**
+ Dia (mm)	=	**12.00**	**20.00**
Spacing (mm)	=	**200.00**	**200.00**
Ast provided (mm2 / m)	=	1570.80	3141.59
Check for Ast provided	**=**	**OK**	**OK**
As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least			
Secondary Reinforcement (mm2/m)	=	314.16	628.32
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	150.00
Ast provided (mm2 /m)	=	565.49	753.98
Check for Ast provided	**=**	**OK**	**OK**

Shear Reinforcement Calculation:

		Along Traffic Direction		
		Heel Slab	Toe Slab	
Ultimate Shear Force (V_{Ed})	=	**116.501**	**316.319**	kN/m
Ast provided	=	1570.796	3141.59	mm2/m
Depth of slab at critical section	=	**821.694**	**743.240**	mm
Effective depth at critical section	=	738.694	658.240	mm
Percentage of steel provided ($\rho 1$)	=	0.0019	0.0042	
cl. 10.3.1 of IRC :112-2011				
$\rho 1 = A_{sl}/(b_w\, d) <= 0.02$	=	OK	OK	
Actual shear stress=v_{ED} = (V_{Ed}/b*0.9d)	**=**	**0.175**	**0.534**	N/mm^2
Max shear capacity, 0.135 fck(1-fck/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) <= 2.0	=	1.520	1.551	
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010				
$\nu_{min} = 0.031\ K^{3/2}\ fck^{1/2}$	=	0.344	0.354	N/mm^2
$0.12\ K\ (80\ \rho 1\ f_{ck})^{0.33}$	=	0.317	0.421	N/mm^2
$\sigma_{cp} = N_{Ed} / A_c <= 0.2\ f_{cd}$	=	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 ($\nu_{min} + 0.15\ \sigma_{cp}$) b_w d	=	315.264	385.809	kN
Check for Shear Reinforcement		**OK, No shear reinf. Req.**	**OK, No shear reinf. Req.**	
Balance Shear Force= $V_{Rd,s} = V_{Ed} - V_{Rd,c}$	=	0.000	0.000	KN/m
b	=	11.240	11.240	m
Total Shear Force	=	0.000	0.000	kN
θ = 0.5 x sin^-1 [vEd / (0.18 fck (1- fck/250))]		0.927	2.828	
cot θ = (< 1 cotθ <2.5)	=	2.500	2.500	
fywd = 0.8 x fy/1.15	=	347.826	347.826	N/mm^2
Provide Shear Reinforcement				
Legged	=	**0**	**0**	
Dia	=	**0**	**0**	mm
Area of Shear Reinf, Asw	=	0.000	0.000	mm^2
z = 0.9 * d	=	664.825	592.416	mm
Spacing of shear Reinforcement required				
S = Asw*z* fywd* cotθ / V_{Rd}	=	0.000	0.000	mm
As per Clause 10.3.3.5 of IRC:112-2011				
Asw/ (b S) = $\rho_{w,min} = (0.072\ f_{ck}^{0.5}) / f_{yk}$	=	0.001	0.001	
Spacing of shear Reinforcement required	=	0.000	0.000	mm
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011				
S_{max} = 0.75 d	=	554.021	493.680	mm
Governing Spacing of Shear Reinf.	=	**0.000**	**0.000**	mm
Provided Spacing of Shear Reinf.	=	**200**	**150**	mm

SLS CHECK OF FOUNDATION

Foundation Lvl = 1809.251 m

Properties of Footing Base:

A	=	78.680	m^2
ZL	=	91.793	m^3
ZT	=	147.394	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			1100.000	0.710	781.000	0.000	0.000
SIDL except Wearing Course	1.00			80.000	0.710	56.800	0.000	0.000
Wearing Course	1.20			145.200	0.710	103.092	0.000	0.000
				1325.200		**940.892**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	2.023	50.580	0.340	17.197	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.475	11.886	0.340	4.041	0.000	0.000
Bracket - Uniform portion	1.00	25	1.012	25.290	0.040	1.012	0.000	0.000
Bracket - Tapered portion	1.00	25	0.506	12.645	0.090	1.138	0.000	0.000
Cap - (uniform portion)	1.00	25	2.428	60.696	0.550	33.383	0.000	0.000
Cap - (corbel portion)	1.00	25	0.000	0.000	0.550	0.000	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	2.400	60.008	-1.900	-114.029	0.000	0.000
Cantilever Return Wall-Traingle portion	1.00	25	4.135	103.363	-1.234	-127.498	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.340	9.520	0.000	0.000
Aproach Slab	1.00	25	5.901	147.525	0.040	5.901	0.000	0.000
				499.993		**-169.335**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	23.753	593.820	-1.700	-1009.494	0.000	0.000
Abutment Shaft	1.00	25	43.383	1084.576	0.572	620.112	0.000	0.000
Back filling over heel slab	1.00	20	261.923	5238.469	-1.732	-9075.347	0.000	0.000
Front Filling over toe slab	1.00	20	66.035	1320.700	2.188	2889.617	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	26.302	657.540	-1.377	-905.382	0.000	0.000
Toe slab	1.00	25	18.265	456.625	2.026	924.958	0.000	0.000
portion between heel & toe	1.00	25	10.116	252.900	0.550	139.095	0.000	0.000
Vertical Components of active earth pressure	1.00			600.440	-3.500	-2101.540	0.000	0.000
				10338.855		**-8497.963**		**0.000**
Total				**12164.047**		**-7726.406**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		662.600	1815.634	4229.376
due to Earth pressure	1.00	1649.695		5021.935

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
662.60	4229.38	0.00	0.00
1649.69	5021.94		
2312.295	**9251.311**	**0.000**	**0.000**

Summary of Forces

P	12164.047	KN
ML	1524.905	kNm
MT	0.000	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		12164.047		-7726.406		0.000
CWLL-Max. Reaction case	1.00	932.327	0.710	661.952	1.943	1811.433
Vertical Components of LL Surcharge	0.80	159.057	-3.500	-556.699	0.000	0.000
Total		**13255.431**		**-7621.154**		**1811.433**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		662.600	1815.634	4229.376
due to Earth pressure	1.00	1649.695		5021.935
due to Live load surcharge	0.80	437.005		1583.707

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
662.60	4229.38	0.00	0.00
1649.69	5021.94		
437.01	1583.71		
2749.300	**10835.018**	**0.000**	**0.000**

Summary of Forces

P	13255.431	KN
ML	3213.865	kNm
MT	1811.433	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				**1325.200**		**940.892**		**0.000**

Substructure & Foundation -Portion 1				**499.993**		**-169.335**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	23.753	593.820	-1.700	-1009.494	0.000	0.000
Shaft above HFL	1.00	25	22.223	555.566	0.617	342.608	0.000	0.000
Shaft below HFL	1.00	15	21.160	317.406	0.572	181.478	0.000	0.000
Back filling above HFL over heel slab	1.00	20	151.659	3033.181	-1.700	-5156.408	0.000	0.000
Back filling below HFL over heel slab	1.00	10	115.322	1153.224	-1.774	-2045.455	0.000	0.000
Front Filling over toe slab	1.00	10	66.035	660.350	2.188	1444.808	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	26.302	394.524	-1.377	-543.229	0.000	0.000
Toe slab	1.00	15	18.265	273.975	2.026	554.975	0.000	0.000
Portion between Heel & Toe	1.00	15	10.116	151.740	0.550	83.457	0.000	0.000
Vertical Components of active earth pressure	1.00			530.433	-3.500	-1856.516	0.000	0.000
				7769.639		**-7987.024**		**0.000**
Total				**9594.831**		**-7215.468**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		662.600	1815.634	4229.376	662.60	4229.38	0.00	0.00
due to Earth pressure	1.00	1457.353		3904.555	1457.35	3904.56		
					2119.953	**8133.931**	**0.000**	**0.000**

Summary of Forces

P	9594.831	KN
ML	918.463	kNm
MT	0.000	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		9594.831		-7215.468		0.000
CWLL-Min. Reaction case	1.00	466.273	0.710	331.054	2.359	1099.889
Vertical Components of LL Surcharge	0.80	159.057	-3.500	-556.699	0.000	0.000
Total		**10220.161**		**-7441.113**		**1099.889**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		662.600	1815.634	4229.376	662.60	4229.38	0.00	0.00
due to Earth pressure	1.00	1457.353		3904.555	1457.35	3904.56		
due to Live load surcharge	0.80	437.005		1583.707	437.01	1583.71		
					2556.959	**9717.638**	**0.000**	**0.000**

Summary of Forces

P	10220.161	KN
ML	2276.525	kNm
MT	1099.889	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 (1817.699 - 1809.251)

= 0.000 kNm

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

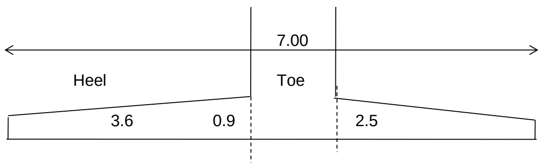
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

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

LOAD CASES	P	ML	MT	Design Base Pressure 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	12164.047	1524.905	0.000	137.989	171.214	137.989	171.214
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	13255.431	3213.865	1811.433	145.750	215.774	121.171	191.195
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	9594.831	918.463	0.000	111.942	131.953	111.942	131.953
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	10220.161	2276.525	1099.889	112.557	162.158	97.632	147.234

Pressure calculation along Face AB:



Case 1 : 137.989 155.08 159.348 171.214

Case 2:	145.750	181.76	190.766	215.774
Case 4:	111.942	122.23	124.806	131.953
Case 5:	112.557	138.07	144.443	162.158

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 163.757 kN/m^2

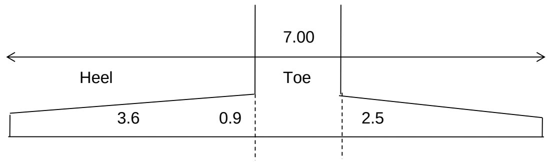
Average Base Pressure for Design of Toe Slab-along Face AB = 203.270 kN/m^2

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 146.533 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face AB = 165.281 kN/m^2

Pressure calculation along Face CD:



Case 1 :	137.989	155.08	159.348	171.214
Case 2:	121.171	157.18	166.186	191.195
Case 4:	111.942	122.23	124.806	131.953
Case 5:	97.632	123.14	129.519	147.234

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 146.533 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 178.691 kN/m^2

For Quasi Permanent Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 146.533 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 165.281 kN/m^2

Moment Calculation

	Rare Combination		Quasi-Parmenant		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	163.76	203.27	146.53	165.28	kN/m^2
Upward moment due to Base pressure	1061.14	635.22	949.53	516.50	kNm/m
Downward moment due to backfill	838.90	0.00	838.90	0.00	kNm/m
Downward moment due to self weight of slab	86.40	41.67	86.40	41.67	kNm/m
Net Moment	**135.84**	**593.55**	**24.23**	**474.84**	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 = 1.2

Ecm = 32308.25 N/mm^2

Es = 200000.00 N/mm^2

Eceff = $\frac{Ecm}{(1+\phi)}$ = 1.47E+04

Modular Ratio (m) = Es/ Eceff = 13.62

		Rare Combination		Quasi Permanent Comb.		
		Heel Slab	**Toe Slab**	**Heel Slab**	**Toe Slab**	
Working bending moment, M	**=**	**135.84**	**593.55**	**24.23**	**474.84**	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)	=	0.815	3.561			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	20.000			mm
Effective Depth deff (dy)	=	917.000	917.000			mm
Ast provided	=	1570.796	3141.593			mm^2/m
Percentage of steel , pt	=	0.0017	0.0034			
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m	=	0.194	0.262			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	177.834	240.584			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	739.166	676.416			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	656.166	591.416			mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2				
Icr	=	1.356E+10	2.422E+10			mm^4
Maximum compressive stress in concrete	=	**1.781**	**5.897**	**0.318**	**4.717**	**< 16.8, SAFE**
Maximum Tensile stress in steel	=	**89.503**	**197.408**	**15.965**	**157.924**	**< 300, SAFE**

Check For Crack Width in Quasi-Permanent Load Combination

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)	=	415.000	425.000	mm
Provided Spacing	=	100.000	100.000	mm
Check for Applicability of Formula	=	OK	OK	
Maximum crack spacing , $S_{r\,max}$	=	3.4 c +	$\frac{0.425\ k1\ k2\ \varphi}{\rho_{\rho\,eff}}$	
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , yc	=	177.834	240.584	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.		
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	207.500	207.500	mm
Ac, eff = Dx * hc,eff	=	207500.000	207500.000	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	0.008	0.015	
Maximum crack spacing , $S_{r\,max}$	=	614.308	479.568	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t\, f_{ct\,eff}\,(1 + \alpha_e\, \rho_{\rho\,eff})}{\rho_{\rho\,eff}}$	/ Es	
tensile stress in steel, ,σsc	=	15.965	157.924	N/mm^2
Kt	=	0.500	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	2.771	N/mm^2
αe = Es/Ecm	=	6.190	6.190	
(εsm - εcm)	=	0.00005	0.0005	
Crack width , Wk=Sr max (εsm - εcm)	**=**	**0.029**	**0.227**	mm
Check	**=**	**SAFE**	**SAFE**	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom M = 1810.251 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			1485.000	0.160	237.600	0.000	0.000
SIDL except Wearing Course	1.35			108.000	0.160	17.280	0.000	0.000
Wearing Course	1.75			211.750	0.160	33.880	0.000	0.000
				1804.750		**288.760**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.023	68.283	-0.210	-14.339	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.475	16.047	-0.210	-3.370	0.000	0.000
Bracket - Uniform portion	1.35	25	1.012	34.142	-0.510	-17.412	0.000	0.000
Bracket - Tapered portion	1.35	25	0.506	17.071	-0.460	-7.853	0.000	0.000
Cap - (uniform portion)	1.35	25	2.428	81.940	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.210	-7.938	0.000	0.000
Aproach Slab	1.35	25	5.901	199.159	-0.510	-101.571	0.000	0.000
				454.440		**-152.483**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1.35	25	43.383	1464.178	0.067	97.634	0.000	0.000
Total				**3723.368**		**233.911**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.38	1815.634	4857.48
due to Earth pressure	1.5	1838.83		4825.37

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	4857.48	0.00	0.00
1838.83	4825.37	0.00	0.00
2741.20	**9682.86**	**0.000**	**0.000**

Summary of Forces

P	3723.37	KN
ML	9916.77	kNm
MT	0.00	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		3723.368		233.911		0.000
CWLL-Max. Reaction case	1.5	1398.490	0.160	223.758	1.943	2717.150
Total		**5121.858**		**457.670**		**2717.150**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	4857.485
due to Earth pressure	1.5	1838.826		4825.373
due to Live load surcharge	1.2	565.068		1765.273

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	4857.48	0.00	0.00
1838.83	4825.37		
565.07	1765.27		
3306.269	**11448.130**	**0.000**	**0.000**

Summary of Forces

P	5121.858	KN
ML	11905.800	kNm
MT	2717.150	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				1804.750		288.760		0.000
Substructure-Portion 1				454.440		-152.483		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	22.223	750.015	0.067	67.517	0.000	0.000
Shaft below HFL	1.35	23.500	21.160	671.314	0.022	19.716	0.000	0.000
Total				**3680.518**		**223.510**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	4857.485
due to Earth pressure	1.5	1691.626		3921.930

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	4857.48	0.00	0.00
1691.63	3921.93	0.00	0.00
2594.001	**8779.415**	**0.000**	**0.000**

Summary of Forces

P	3680.518	KN
ML	9002.925	kNm
MT	0.000	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		3680.518		223.510		0.000
CWLL-Max. Reaction case	1.5	699.410	0.160	111.906	2.359	1649.834
Total		**4379.928**		**335.416**		**1649.834**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		902.375	1815.634	4857.485
due to Earth pressure	1.5	1691.626		3921.930
due to Live load surcharge	1.2	565.068		1765.273

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
902.38	4857.48	0.00	0.00
1691.63	3921.93		
565.07	1765.27		
3159.069	**10544.688**	**0.000**	**0.000**

Summary of Forces

P	4379.928	KN
ML	10880.103	kNm
MT	1649.834	kNm

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

Seismic Effect Factor = 1.50 αh= 0.000 In Longitudinal direction

αh= 0.000 In Transverse direction

αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 453.6 KN

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 = 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			1485.000		0.000	0.000	0.160	237.600	0.000	1816.167		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.000		0.000	0.000	0.160	17.280	0.000	1816.948		0.000	0.000	0.000
Wearing Course	1.75			211.750		0.000	0.000	0.160	33.880	0.000	1816.499		0.000	0.000	0.000

				1804.750		0.000	0.000		288.760	0.000				0.000	0.000
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.023	68.283	0.000	0.000	0.000	-0.210	-14.339	0.000	1816.199	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.475	16.047	0.000	0.000	0.000	-0.210	-3.370	0.000	1815.829	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.012	34.142				-0.510	-17.412						
Bracket - Tapered portion	1.35	25	0.506	17.071				-0.460	-7.853						
Cap - (uniform portion)	1.35	25	2.428	81.940	0.000	0.000	0.000	0.000	0.000	0.000	1815.483	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1815.333	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800				-0.210	-7.938				0.000		
Aproach Slab	1.35	25	5.901	199.159				-0.510	-101.571				0.000		
				454.440	**0.000**	**0.000**	**0.000**		**-152.483**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure-Portion 2															
Abutment Shaft	1.35	25	43.383	**1464.178**	**0.000**	**0.000**	**0.000**	**0.067**	**97.634**	**0.000**	**1813.792**	**0.000**	**0.000**	**0.000**	**0.000**
Total =				**3723.368**	**0.000**	**0.000**	**0.000**		**233.911**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	4857.485	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1225.884			3216.915	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1225.88	3216.92						
0.00	0.00						
2128.26	**8074.40**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3723.368	3723.368	KN
ML	8308.311	8308.311	kNm
MT	0.000	0.000	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				1804.750		0.000	0.000		288.760	0.000				0.000	0.000
Forces from Substructure				1918.618	0.000	0.000	0.000		-54.849	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			186.47		0.000	0.000	0.160	29.834	0.000	1817.699		1.943	362.287	0.000
Total =				**3909.833**	**0.000**	**0.000**	**0.000**		**263.746**	**0.000**		**0.000**		**362.287**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	4857.485	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1225.884			3216.915	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	94.178			294.212	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1225.88	3216.92						
0.00	0.00						
94.18	294.21						
0.00	0.00						
2222.44	**8368.61**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3909.833	3909.833	KN
ML	8632.358	8632.358	kNm
MT	362.287	362.287	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				1804.750		0.000	0.000		288.760	0.000				0.000	0.000
Substructure-Portion 1				454.440	0.000	0.000	0.000		-152.483	0.000		0.000		0.000	0.000
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	22.223	750.015	0.000	0.000	0.000	0.067	50.012	0.000	1814.042	0.000	0.000	0.000	0.000
Shaft below HFL	1.350	23.500	21.160	671.314	0.000	0.000	0.000	0.022	14.605	0.000	1812.501	0.000	0.000	0.000	0.000
				1421.328	**0.000**	**0.000**	**0.000**		**64.617**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**3680.518**			**0.000**		**200.894**	**0.000**				**0.000**	
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	4857.485	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1127.750			2614.620	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1127.75	2614.62						
0.00	0.00						
2030.13	**7472.10**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Downward	Upward	
P	3680.518	3680.52	KN
ML	7672.999	7673.00	kNm
MT	0.000	0.00	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 = 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				1804.750		0.000	0.000		288.760	0.000				0.000	0.000
Forces from Substructure				1875.768	0.000	0.000	0.000		-87.866	0.000		0.000		0.000	0.000
CWLL-Min. Reaction case	0.20			93.25		0.000	0.000	0.160	14.921	0.000	1817.699		2.359	219.978	0.000
Total =				**3773.773**	**0.000**	**0.000**	**0.000**		**215.815**	**0.000**		**0.000**		**219.978**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0.000	1815.634	4857.48	0.000
due to Substructure		0.000	0.000		0.00	0.000
due to Active Earth pressure	1.00	1127.750			2614.62	
due to dynamic increment of EP	1.50	0.000			0.00	
due to Live load surcharge	0.20	94.178			294.21	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
902.38	4857.48	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	0.00
1127.75	2614.62						
0.00	0.00						
94.18	294.21						

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to dynamic increment of Surcharge	1.50	0.000			0.00	

FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
0.00	0.00						
2124.30	**7766.32**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3773.773	3773.773	KN
ML	7982.132	7982.132	kNm
MT	219.978	219.978	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**3723.368**	**0.000**	**233.911**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0	1815.634	4857.485	0
due to Substructure		0.000	0		0.000	0
due to Active Earth pressure	1.00	1225.884			3216.915	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1225.88	3216.92						
0.00	0.00						
2128.26	**8074.40**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3723.368	3723.368	KN
ML	8308.311	8308.311	kNm
MT	0.000	0.000	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**3909.833**	**0.000**	**263.746**	**0.000**	**362.287**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0	1815.634	4857.485	0
due to Substructure		0.000	0		0.000	0.000
due to Earth pressure	1.00	1225.884			3216.915	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	94.178			294.212	
Surcharge	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1225.88	3216.92						
0.00	0.00						
94.18	294.21						
0.00	0.00						
2222.44	**8368.61**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3909.833	3909.833	KN
ML	8632.358	8632.358	kNm
MT	362.287	362.287	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**3680.518**	**0.000**	**200.894**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0	1815.634	4857.485	0.000
due to Substructure		0.000	0		0.000	0.000
due to Active Earth pressure	1.00	1127.750			2614.620	
due to dynamic increment of EP	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1127.75	2614.62						
0.00	0.00						
2030.13	**7472.10**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3680.518	3680.518	KN
ML	7672.999	7672.999	kNm
MT	0.000	0.000	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**3773.773**	**0.000**	**215.815**	**0.000**	**219.978**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		902.375	0	1815.634	4857.485	0
due to Substructure		0.000	0		0.000	0.000
due to Earth pressure	1.00	1127.750			2614.620	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	94.178			294.212	
Surcharge	1.50	0.000			0.000	

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
902.38	4857.48	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	0.00
1127.75	2614.62						
0.00	0.00						
94.18	294.21						
0.00	0.00						
2124.30	**7766.32**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	3773.77	3773.77	KN
ML	7982.13	7982.13	kNm
MT	219.98	219.98	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.50	x	0.00		=	0.000 KN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1817.699 -	1810.251)	=	0.000 kNm	

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

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00		=	0.000 KN	Seismic	0.00	0.00	0.00	0.00
Transverse Moment due to C.F.	=	0.000	x (	1817.699 -	1810.251)	=	0.000 kNm					

Summary of ULS Forces for Design of Abutment Shaft

LOAD CASES		Total forces at bottom of abutment shaft P	ML	MT
Normal Dry Case		kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case		3723.368	9916.769	0.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		5121.858	11905.800	2717.150
Normal HFLCase				
Case 3 : DL+SIDL-Normal HFL Case		3680.518	9002.925	0.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		4379.928	10880.103	1649.834
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	3723.368	8308.311	0.000
	UP	3723.368	8308.311	0.000
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	3909.833	8632.358	362.287
	UP	3909.833	8632.358	362.287
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	3680.518	7672.999	0.000
	UP	3680.518	7672.999	0.000
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	3773.773	7982.132	219.978
	UP	3773.773	7982.132	219.978
Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	3723.368	8308.311	0.000
	UP	3723.368	8308.311	0.000
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	3909.833	8632.358	362.287
	UP	3909.833	8632.358	362.287
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	3680.518	7672.999	0.000
	UP	3680.518	7672.999	0.000
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	3773.773	7982.132	219.978
	UP	3773.773	7982.132	219.978
	MAX =	**5121.86**	**11905.80**	**2717.15**

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)	=	0.900	m
Depth of Wall (D)	=	11.240	m
Area of Concrete (Ac)	=	10.116	m^2
Clear Cover to earth faces	=	75	mm
Clear Cover to non earth faces	=	50	mm
Maximum Dia of Vertical Reinf.	=	25	mm
Dia of Horizontal Reinf.	=	12	mm
Effective cover	=	137	mm

As per Clause 7.6.4.1 of IRC:112-2011

Ultimate axial force (Pu) = **5121.86** kN

0.1 fcd Ac = 0.1 x 15.63 x 10116000
= 15814680 N
= **15814.68** 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)	=	11905.80 kNm	=	**1059.235 kNm/m**

Check For Depth of Wall :

Mult = 0.165 x fck x b x d^2
= 1059.23 kNm/m

b = 1000.00 mm

Effective Depth Required (dreq) = $SQRT\left(\dfrac{1059.23 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

(dreq) = 428.272 mm

Total Depth Required (Dreq)	=	527.77	mm	
Total Depth Provided (Dprov)	=	900.00	mm	OK
Effective depth provided(deff)	=	763.00	mm	

R= Mu/(b d^2) = **1.82**

Minimum Longitudinal Reinforcement in wall on each face

= 0.0012 x b x D

Ast min = **1080.00** mm^2/m

Area of Steel Required:

$$\frac{pt}{100} = \frac{Ast_{req}}{b\,D} = \frac{fck\,\{1 - sqrt(1 - 4.598\,R/fck)\}}{2\,fy}$$

= 0.0045

Ast_{req} = 4021.335 mm^2/m

Ast required = max(Astmin, Astreq) = **4021.34** mm^2/m

Total are of steel required in full length = **45199.81** mm^2

Provide	25	mm dia	@	100.00	mm c/c	=	4908.74	mm^2/m	OK
Provide	0	mm dia	@	90.00	mm c/c	=			

Effective length of shaft = 11008 mm

Calculation of reinforcement in numbers

Provide	25	mm dia	-	110.00	nos	=	53996.12	mm^2	OK
Provide	0	mm dia	-	123.00	nos	=			

Percentage of steel = **0.534** %

Check for Moment of Resistance of Section due to Steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

= $\frac{0.0035 \times 763.00}{0.0035 + 0.0022}$

= 470.66 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 4908.74}{0.36 \times 35.00 \times 1000.00}$

= 169.38 mm **OK**

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

z = d - 0.416 . X
= 763.00 - 70.46
= 692.54 mm

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

MR = fyd . Ast . z

= 434.78 x 4908.74 x 692.54

= 1.48E+09 Nmm /m

= 1478.03 kNm/m > 1059.23 kNm/m

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

= 0.0012 x b x D
Ast min = **1080.00** mm^2/m
= **12139.20** mm^2

Provide	12	mm dia	@	100.00	mm c/c	=	1130.97 mm^2/m	OK

Calculation of reinforcement in numbers

Provide	12	mm dia	@	110.00	nos	=	12440.71 mm^2	OK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

maximum of following = 0.2500 x 6039.71 = 1509.928 As per IRC: 112-2011 , Clause 16.3.2
= 0.001 x 9.00E+05 = 900.000

Maximum Horizontal Reinf. **1510** mm^2 per meter

Min dia of bar = 0.250 x 25 = 6.25 mm
or 8 mm

Maximum Spacing between <= 300 mm c/c

2 Legged	12	dia	@	140 c/c	=	1615.676 mm^2	OK

Confinement Reinforcement

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

Distance between links or ties (ST) = 1/3 x 751 = 250.333
or 200.00 mm

Governing Spacing = 200.00 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)

SL	=		5	x	25 =	125 mm
	or	1/5		x	751 =	150.2 mm
Min	=		100 mm			

2 Legged	12 dia	@	100 c/c	=	2261.947 mm^2	OK
24 Legged	10 dia	@	100 c/c	=	18849.556 mm^2	
40 links	10 dia	@	100 c/c	=	31415.927 mm^2	
					52527.429 mm^2	

Minimum Confinement Reinforrcement:

nk = $\frac{NED}{Ak\ fck}$ = $\frac{5121858.2}{354060000}$ = 0.0145

AC	=	10.116 mm^2				
ACC	=	0.775	x	11.140	=	8.634 mm^2
ρ L	=	0.00535 per meter				
ρ L	=	0.06014				
fyd	=	434.783				
fcd	=	15.633				

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

$\omega_{wd.req}$ = 0.1875

ω_{wd} = max ($\omega_{wd,req}$, 0.12) = 0.1875

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

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

Volumetric ratio,

Asw	=	52527.429 mm^2
SL	=	100.000 mm
b	=	751.000 mm
ρw	=	0.699
$\omega_{wd.c}$	=	19.452

$\omega_{wd,c}$ ≥ ω_{wd} as per equation 17.7 of IRC:112-2011

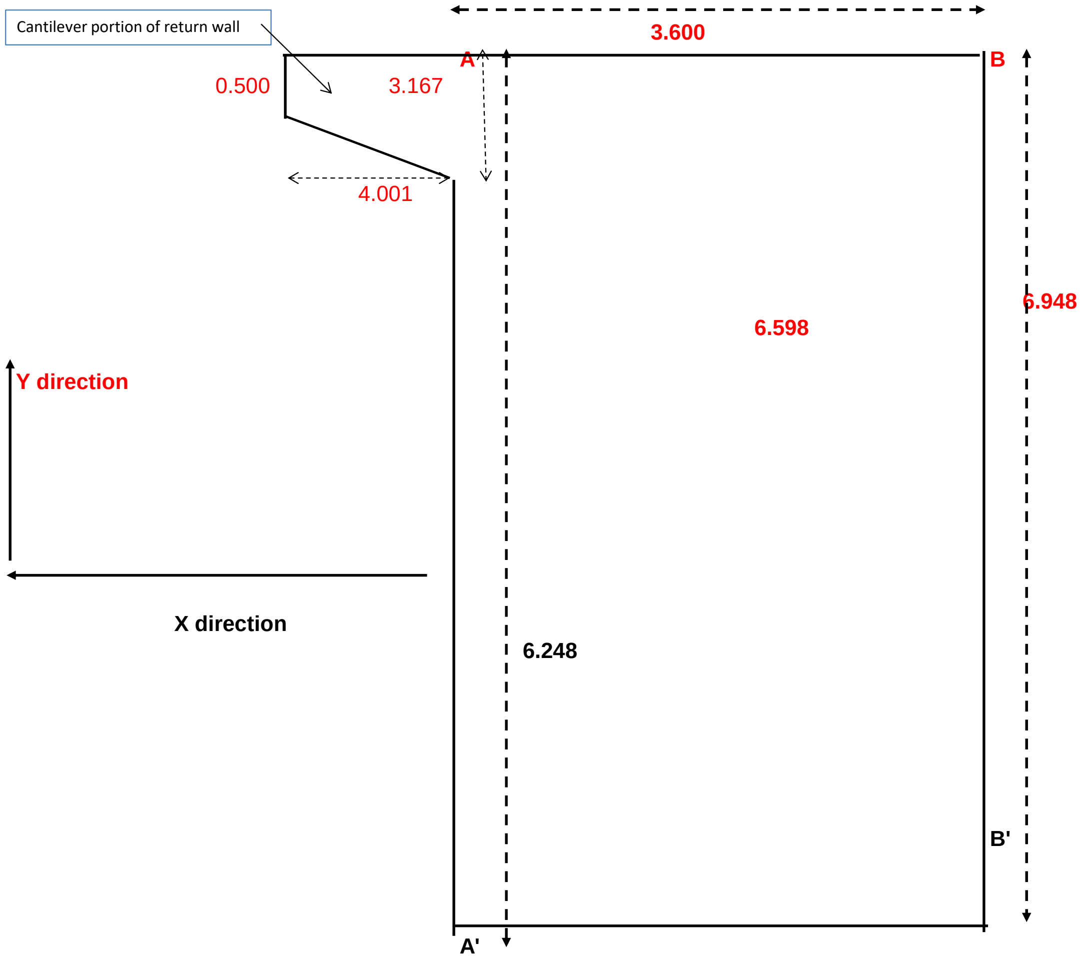
$\omega_{wd,c}$ = 19.45 OK

Length of Potential Plastic Hinges Refer clause 17.2.1.4 of IRC:112-2011

nk = $\frac{NED}{Ak\ fck}$ = 0.0145 < 0.30

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	=	3.60	m
Avg. Heigth of Solid Return	b	=	6.598	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 = 0.546

a/b =	0.5	β1 =	0.631	β2 =	0.632
a/b =	0.75	β1 =	1.246	β2 =	1.186

For uniformly distributed load over entire plate

For, a/b = 0.546 β1 = 0.743

β2 = 0.733

Live Load Surcharge Intensity:

q = 0.2794 x 20.00 x 1.200 = 6.705 kN/m^2

Max. .σ b = $\frac{\beta 1 \times q \times b\text{^}2}{(t1)\text{^}2}$

.σ a = $\frac{\beta 2 \times q \times b\text{^}2}{(t2)\text{^}2}$

.σ b = $\frac{0.743 \times 6.705 \times 43.534}{0.250}$

At bottom edge = 867.795 kN/m2 = **0.868 MPa**

For 1000 mm of width , Z = 1000 x 250000

				6
	=	4.17E+07 mm^3		

Hence Moment /m width along Y direction -

My /m width = 0.868 x 4.167E+07

= 36158120 Nmm/m = **36.158 kN.m/m**

$.\sigma a = \dfrac{0.733 \times 6.705 \times 43.534}{0.250}$

= 856 = **0.8560 MPa**

For 1000 mm of height , Z = $\dfrac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence, Moment /m height along X direction -

Mx /m height = 0.8560 x 4.167E+07 = 3.567E+07 Nmm/m

= **35.665 kN.m/m**

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.500	β1 =	0.328	β2 =	0.200
a/b =	0.75	β1 =	0.537	β2 =	0.276

For, a/b = 0.546 β1 = **0.366**

β2 = **0.214**

q = 0.279 x 20.00 x 6.60

= 36.867 kN/m^2

Max. $.\sigma b = \dfrac{\beta 1 \times q \times b^2}{(t1)^2}$

$.\sigma a = \dfrac{\beta 2 \times q \times b^2}{(t2)^2}$

$\sigma b = \dfrac{0.366 \times 36.867 \times 43.534}{0.25}$

= 2350.57 kN/m^2

= **2.351 MPa**

For 1000 mm of width , Z = $\dfrac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 2.351 x 4.167E+07

= 97940365 Nmm/m = **97.940 kN.m/m**

$.\sigma a = \dfrac{0.214 \times 36.867 \times 43.534}{0.25}$

= 1373.0 kN/m^2 = 1.373 MPa

For 1000 mm of height , Z = $\dfrac{1000 \times 250000}{6}$

= 4.167E+07 mm^3

Hence Moment /m height along X direction -

Mx /m height = 1.373 x 4.167E+07 = 5.721E+07 Nmm/m

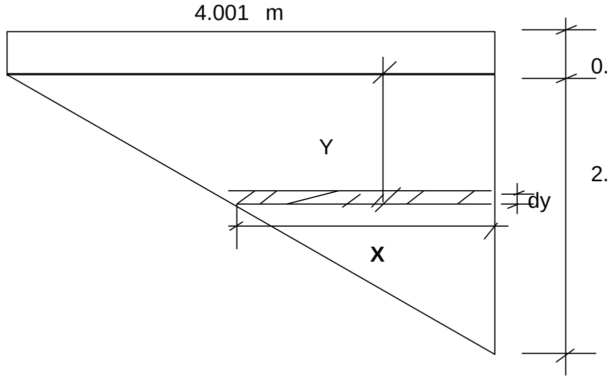
= **57.209 kN.m/m**

Total Moment in Solid Return Wall / m height = 92.874 kN.m/m

Total Moment in Solid Return Wall / m width = 134.098 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 =		129	kN.m/m
Total Moment(My) in Solid Return Wall / m width =		190	kN.m/m



X = 4.001 + -1.500 y

Earth pressure due to LL surchage = 0.279 x 20.000 x 1.200
= 1.20 x 6.705 = **8.046** kN / m^2
load factor

Earth pressure at a depth of 0.500 m from top
= 0.279 x 20 x 0.5
= 1.50 x 2.794 = **4.191** kN / m^2
load factor

Earth pressure at a depth y from top = 0.279 x 20 x y
= 1.50 x 5.588 = **8.382 y**
load factor

A *LL surchage on top* *0.5 m*

Force = 8.046 x 4.00 x 0.5 = 16.095 kN

Moment at face BB' = 16.095 x (4.001 / 2 + 3.600)
(Mx) 1
= **90.133** kN.m

B *LL surchage on triangular portion*

Force for unit strip = 8.046 x **X** x dy = 8.046 X dy

(dMx) 2 = 8.046 **X** dy x { **X** / 2 + 3.60 }

= 8.046 x { **X** ^2 / 2 + 3.6 **X** } dy

= 8.046 x { (4.001 + -1.500 y) ^2 / 2 +
3.60 x (4.001 + -1.500 y) } dy

= 8.046 x { (8.002 + 1.125 y^2 +
-6.001 y) + 14.402 + -5.400 y } dy

= 8.046 x (22.404 + 1.125 y^2 +
-11.401 y) dy

After integrating between limits 0 and 2.667 m

(Mx) 2 = 8.046 x (22.404 x 2.667 + 0.375
x 2.667 ^3 + -5.700 x 2.67 ^2)

= 8.046 x (59.751 + 7.114 + -40.546)

= **211.766** kN.m

C *Earth press. on top* *0.5 m*

Force = 0.5 x 4.191 x 4.001 x 0.5
= 4.191 kN

Moment at face BB' = 4.191 x ($\frac{4.001}{2}$ + 3.600)
(Mx) 3
= $\boxed{\textbf{23.472 kN.m}}$

D *Earth pressure on triangular portion*

Force for unit strip = (4.191 + 8.382 y) **X** x dy

(dMx) 4 = (4.191 + 8.382 y) X x { $\frac{X}{2}$ + 3.6 }

= (4.191 **X** ^2 + 15.087 **X** + 4.1908 **X** ^2 x y +
(*Part 1*) (*Part 2*) (*Part 3*)

30.173 **X** x y) dy
(*Part 4*)

Part 1 :
(dMx) 4.1 = 4.191 **X** ^2 dy

= 4.191 x (4.001 + -1.500 y) ^2 dy

= 4.191 x (16.0040003 + 2.2500 y^2 + -12.002 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.1 = 4.191 x (16.0040003 x 2.667 + 0.7500 x 2.67 ^3 + -6.001 x 2.667 ^2)

= 4.191 x (16.004 x 2.6670 + 0.750 x 18.97 + -6.001 x 7.113)

= 4.191 x (42.683 + 14.228 + -42.6827)

= 4.191 x (14.228)

= **59.624 kN.m**

Part 2 :
(dMx) 4.2 = 15.087 **X** dy

= 15.087 x (4.001 + -1.500 y) dy
= (60.354 + -22.630 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.2 = 60.354 x 2.6670 + -11.315 x 2.6670 ^2

= 160.965 + -80.483 **=** **80.483 kN.m**

Part 3 :

(dMx) 4.3 = 4.191 **X** ^2 x y dy

= 4.191 x (4.001 + -1.500 y) ^2 x y x dy

= 4.191 x (16.0040003 y + 2.250 y ^3 + -12.0015 y^2) x dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.3 = 4.191 x (8.00200013 x 2.6670 ^2 + 0.5625 x 2.66700 ^4 + -4.00050 x 2.66700 ^3)

= 4.191 x (56.917 + 28.459 + -75.890)

= 4.191 x (9.486) **=** **39.754 kN.m**

Part 4 :

(dMx) 4.4	=	30.173 **X** x y dy							
	=	30.173	x (	4.001	+	-1.500 y) x y x dy			
	= (	120.709 y	+	-45.260 y^2) dy					
After integrating between limits			0.000 and		2.667 m				
(Mx) 4.4	=	60.354	x	2.667 ^2	+	-15.087	x	2.667 ^3	
	=	429.294	+	-286.196	**=**	**143.098 kN.m**			
(Mx) 4	=	59.624	+	80.483	+	39.754	+	143.098	
	=	**322.959 kN.m**							
Total Moment at face BB' =		(Mx) 1	+	(Mx) 2	+	(Mx) 3	+	(Mx) 4	
		90.133	+	211.766	+	23.472	+	322.959	
	=	**648.331 kN.m**							
Horizontal moment per meter =		648.331 /		6.598 =		**98.262 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) =

= **128.612 + 98.26**

= **226.87 kN.m / m**

Adorting clear cover on either face	=	75 mm		
Minimum Dia of Reinforcement	=	16 mm		
Maximum Spacing of Steel	=	125 mm		
Thickness of wall	=	0.500 m		
Available effective depth	=	500	-75	-8
	=	417 mm		

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 226.87 kNm/m

Effective Depth of Cap Required (dreq) = SQRT($\frac{226.87 \text{ x } 1000000}{0.165 \text{ x } 35.00 \text{ x } 1000}$)

Effective Depth of Cap Required (dreq)	=	198.205 mm	
Total Depth Required (Dreq)	=	281.21 mm	
Total Depth Provided (Dprov)	=	500.00 mm	**OK**
R= Mu/(b d^2)	=	1.30	

Area of Steel Required:

$\frac{pt}{100}$ = $\frac{Ast_{req}}{b\ d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 1309.545 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011

= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1309.545 mm^2/m

Provide 16 mm dia bar @ 125 mm c/c as Horizontal steel at earth face.

Provide Ast= 1608 mm^2/m) OK

Percentage of Steel Provided = 0.386 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

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

= 257.230 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 1608}{0.36 \times 35.00 \times 1000}$

= 55.504 mm OK

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

z = d - 0.416 . X
= 417 - 23.090
= 393.910 mm

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

MR = fyd . Ast . z

= 434.78 x 1608 x 393.910

= 2.75E+08 Nmm

= 275.480 kNm/m > 226.87 kNm/m

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

Provide 12 mm dia bar @ 125 mm c/c as Horizontal steel at non earth face.

Provided Ast = **905 mm^2/m)**

2. Design for Face A'B'

Moment in Solid Return /m width = 190.30 kN.m / m

Adorting clear cover on either face = 75 mm
Minimum Dia of Reinforcement = 16 mm
Maximum Spacing of Steel = 150 mm
Thickness of wall = 0.500 m
Available effective depth = 500 -75 -16 -8
= 401 mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 190.30 kNm/m

Effective Depth of Cap Required (dreq) = SQRT($\frac{190.30 \times 1000000}{0.165 \times 35.00 \times 1000}$)

Effective Depth of Cap Required (dreq) = 181.528 mm

Total Depth Required (Dreq) = 264.53 mm
Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 1.18

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d} = \frac{fck \{ 1 - \mathrm{sqrt}(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 1137.085 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1137.085 mm^2/m

Provide	**16 mm dia bar @**	**150 mm c/c as vertical steel at earth face.**	

Provide Ast=	**1340**	**mm^2/m)**	**OK**

Percentage of Steel Provided = 0.3343 %

Provide	**12 mm dia bar @**	**150 mm c/c as Vertical steel at non earth face.**

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\dfrac{0.0035\ .\ d}{(0.0035 + fyd/\ Es)}$

$= \dfrac{0.0035 \times 401}{0.0035 + 0.00217}$

= 247.36 mm

Depth of Neutral Axis , X = $\dfrac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$

$= \dfrac{434.78 \times 1340}{0.36 \times 35.00 \times 1000}$

= 46.253 mm OK

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

z = d - 0.416 . X
= 401 - 19.241
= 381.76 mm

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

MR = fyd . Ast . z

= 434.78 x 1340 x 381.759

= 2.22E+08 Nmm

= 222.484 kNm/m > 190.30 kNm/m

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

b) Cantilever Portion of Return Wall

Self-weight of cantilever portion of return wall	=	23	kN/m
Crash Barrier weight	=	10.0	kN/m
Total Load	=	33	kN/m
Moment at Cantilever Face	=	263	kNm
Load Factor	=	1.35	
Design Moment	=	356	kNm
Effective Depth	=	3104.500	mm

R= Mu/(b d^2) = 0.07

Area of Steel Required:

$\dfrac{pt}{100} = \dfrac{Ast_{req}}{b\ d} = \dfrac{fck\ \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.000

Ast_{req} = 263.985 mm^2

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 1862.7 mm^2

Maximum (Ast_{req}, Ast_{min}) = 1862.7000 mm^2

Provide	**25**	**4**	**=**	**1963**	**mm^2**

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1810.251 m

Case 1 : DL+SIDL-Normal Dry Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1100.000	0.160	176.000	0.000	0.000
SIDL except Wearing Course	1			80.000	0.160	12.800	0.000	0.000
Wearing Course	1			121.000	0.160	19.360	0.000	0.000
				1301.000		**208.160**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	2.023	50.580	-0.210	-10.622	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.475	11.886	-0.210	-2.496	0.000	0.000
Bracket - Uniform portion	1	25	1.012	25.290	-0.510	-12.898	0.000	0.000
Bracket - Tapered portion	1	25	0.506	12.645	-0.460	-5.817	0.000	0.000
Cap - (uniform portion)	1	25	2.428	60.696	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.210	-5.880	0.000	0.000
Aproach Slab	1	25	5.901	147.525	-0.510	-75.238	0.000	0.000
				336.622		**-112.950**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1	25	43.383	1084.576	0.067	72.322	0.000	0.000
Total				**2722.199**		**167.531**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		662.600	1815.634	3566.776
due to Earth pressure	1	1225.884		3216.915
				6783.691

Summary of Forces at Bottom of abutment shaft

P	2722.199	KN
ML	6951.222	kNm
MT	0.000	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		2722.199		167.531		0.000
CWLL-Max. Reaction case	1	932.327	0.160	149.172	1.943	1811.433
Total		**3654.525**		**316.704**		**1811.433**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		662.600	1815.634	3566.776
due to Earth pressure	1	1225.884		3216.915
due to Live load surcharge	0.8	376.712		1176.848
				7960.540

Summary of Forces at Bottom of abutment shaft

P	3654.525	KN
ML	8277.243	kNm
MT	1811.433	kNm

Case 3 : DL+SIDL-Normal HFL Case

Forces due to Vertical Load

Loads	Load Factor	Unit Weights (kN/m³)	Volume (m³)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				1301.000		208.160		0.000
Substructure-Portion 1				336.622		-112.950		0.000
Substructure-Portion 2								
Shaft above HFL	1.000	25.000	22.223	555.566	0.07	37.05	0.00	0.00
Shaft below HFL	1.000	23.500	21.160	497.269	0.02	10.82	0.00	0.00
				2690.458		**143.074**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		662.600	1815.634	3566.776
due to Earth pressure	1	1127.750		2614.620
				6181.396

Summary of Forces at Bottom of abutment shaft

P	2690.458	KN
ML	6324.470	kNm
MT	0.000	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		2690.458		143.074		0.000
CWLL-Max. Reaction case	1	466.273	0.160	74.604	2.359	1099.889
Total		**3156.731**		**217.678**		**1099.889**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		662.600	1815.634	3566.776
due to Earth pressure	1	1127.750		2614.620
due to Live load surcharge	0.8	376.712		1176.848
				7358.245

Summary of Forces at Bottom of abutment shaft

P	3156.731	KN
ML	7575.922	kNm
MT	1099.889	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 (	1817.699 - 1810.251)	=	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	2722.199	6951.222	0.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	3654.525	8277.243	1811.433
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	2690.458	6324.470	0.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	3156.731	7575.922	1099.889

IN RARE COMBINATION

Max SLS Moment	=	8277.243	kNm
Max Moment per meter	=	736.410	kNm/m

IN QUASI-PERMANENT

Max SLS Moment	=	6951.222	kNm
Max Moment per meter	=	618.436	kNm/m

Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff = 1.2

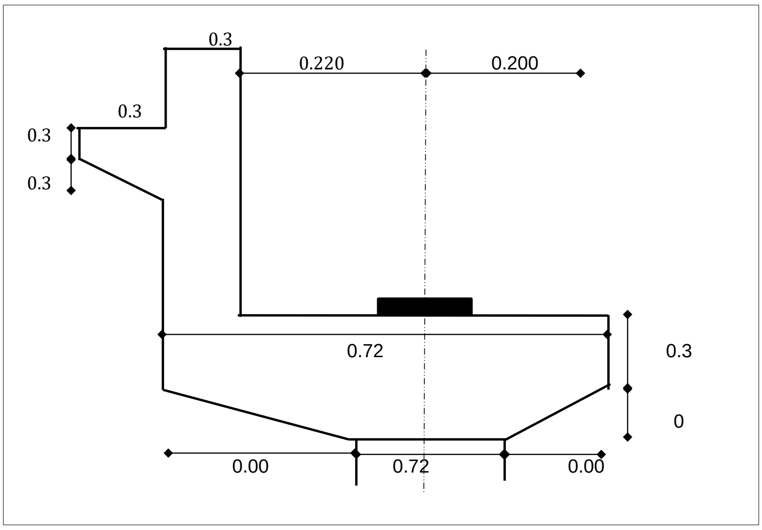
		Rare Combination		Quasi permanent	
		Short term	Long Term		
Working bending moment, M	=	736.41	736.41	618.44	kNm/m
Dx (unit width of shaft)	=	1.00	1.00	1.00	m
Dy (Thickness of shaft)	=	0.90	0.90	0.90	m
Section Modulus (ZL) of uncracke	=	0.14	0.14	0.14	m^3

Bending Stress (M/ZL)	=	5.455	5.455	4.581	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	14685.57	14685.57	N/mm^2
Modular Ratio (m)	=	6.19	13.62	13.62	
Clear Cover, c	=	75.000	75.000	75.00	mm
Maximum dia used, ϕ	=	25.000	25.000	25.00	mm
Effective Depth deff (dy)	=	763.000	763.000	763.00	mm
Ast provided	=	4908.739	4908.739	4908.74	mm^2/m
Percentage of steel , pt	=	0.0053	0.0053	0.0053	
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m		0.226	0.315	0.315	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	172.546	240.703	240.703	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	590.454	522.297	522.297	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	502.954	434.797	434.797	mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2			
Icr	=	1.231E+10	2.289E+10	2.289E+10	mm^4
Maximum compressive stress in concrete	=	**10.325**	**7.745**	**6.505**	**< 16.8, SAFE**
Maximum tensile stress in concrete	=	35.333	16.807	14.114	
Maximum Tensile stress in steel	=	**186.310**	**190.542**	**160.017**	**< 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)	=	437.500	mm
Provided Spacing	=	100.000	mm
Check for Applicability of Formula	=	OK	
Maximum crack spacing , $S_{r\,max}$	=	3.4 c +	0.425 k1 k2 φ / $\rho_{\rho\,eff}$
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	240.703	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.	
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	342.500	mm
Ac, eff = Dx * hc,eff	=	342500.000	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	0.014	
Maximum crack spacing , $S_{r\,max}$	=	551.537	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\dfrac{\sigma_{sc} - k_t f_{ct\,eff} (1 + \alpha_e \rho_{\rho\,eff})}{\rho_{\rho\,eff}}$	/ Es
tensile stress in steel, ,σsc	=	160.017	N/mm^2
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm^2
αe = Es/Ecm	=	13.619	
(εsm - εcm)	=	0.00048	
Crack width , Wk=Sr max (εsm - ε	=	**0.265**	**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.

Assuming a cap thickness of	=	225	mm					
Volume of abutment cap	=	225	x	720	x	11240		
	=	1.82E+09	mm^3					
as per cl. 710.8.7 of IRC : 78- 2014								
Quantity of steel	=	1	% of volume					
	=	$\frac{1}{100}$	x	1.82E+09	=	1.82E+07	mm^3	

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction								
	=	9.10E+06	mm^3					
Clear cover	=	50	mm					
Length of bar	=	11240	-100	=	11140	mm		
Area of steel required in longitudinal direc	=	$\frac{9.10E+06}{11140}$		=	817.2711	mm^2	(top +Bottom)	

Provide	7	Nos. of	12	mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

Provided steel	=	792	mm^2

(b) Transverse steel

Volume of steel to be provided in transverse direction			=	9.10E+06	mm^3
Volume of steel required per meter	=	$\frac{9.10E+06}{11.24}$	=	8.10E+05	mm^3/m

Provide	**2 L**	**12**	**mm dia bar @**	**150**	**mm c/c stirrups**

Length of each stirrups	=	720	-100	-12	=	608	mm
Volume of steel provided per meter	=	9.17E+05	mm^3/m	**OK**			

Thickness of Abutment Cap (uniform)	s	=	0.3	m
Thickness of Abutment Cap (tapered)		=	0	m
C.G. of dirt wall from face of abutment shaft (a)		=	-0.150	m
Overall depth of Abutment cap at face of abutment shaft		=	0.300	m
Clear cover		=	50	mm
Diameter of the main bar		=	12	mm
Effective cover	d'	=	56	mm
Effective depth of cap	d	=	0.244	m

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

Hence	a / d	=	-0.15 / 0.24	
		=	-0.615	**< 1.0**

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

Hence	s / d	=	0.3 / 0.24		
		=	1.23	**> 0.5**	**Proceed with the design**

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	=	5.5575	kN		
Self Weight of Bracket	=	0.135	m^3/m	x	25
	=	3.375	kN		
Total Dead Load	=	8.9325	kN		
Load Factor	=	1.35			
Ultimate Dead Load	=	**12.0589**	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 "	312.06	kN

Horizontal Load (H_u)

H_u = 1.7 x actual horizontal force in working load condition

but not less than

= $0.2 * V_u$

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 $\mathbf{V_u / b\,d <= 0.15 f_c'}$

where;	V_u	=	312.06	kN
		=	312058.88	N
	bd	=	1000 x 244 sqmm	
		=	244000.00	mm^2
	$V_u / b\,d$	=	312058.88 / 244000	
	$V_u / b\,d$	=	**1.28**	N / mm^2
	f_c'	=	28 - day standard cylinder strength of concrete used.	
		=	0.80 times the standard cube strength	
Grade of Concrete			M 35	
	$0.15 f_c'$	=	0.15 x 0.8 x 35	
		=	**4.20**	N / mm^2 **Ensured**

Hence $\mathbf{V_u / b\,d <= 0.15 f_c'}$ is **Ensured**

STEP II

Calculation of Shear Friction Reinforcement " A_{vf} "

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

where;	f_{sy}	=	yeild stress value of the reinforcement used.	
		=	500.00	N/mm^2

	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

Type of Surface (1 / 2 / 3 / 4) ? = **1.00**

m = 1.40

(Note: Only monolithic construction is recommended)

$$A_{vf} = \frac{312.06 \times 1000}{0.87 \times 500 \times 1.4} \text{ mm}^2$$

$$A_{vf} = \mathbf{524.47} \text{ mm}^2$$

STEP III

Calculation for Direct Tension Reinforcement " A_t ".

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

H_u = 68.00 kn

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

= **156.32** mm^2

STEP IV

Calculation for Flexural Tension Reinforcement " A_f "

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

$$= \frac{312.06 \times 1000 \times -150 + 68 \times 1000 (300 - 56)}{0.87 \times 500 \times 244}$$

= **-284.69** mm^2

STEP V

Total Primary Tensile Reinforcement " A_s "

A_s >= $(A_f + A_t)$
>= $(2 / 3 A_{vf} + A_t)$
>= $(0.04 f_c' / f_{sy}) b d$

Provide the largest of these three magnitudes as A_s.

$(A_f + A_t)$ = -284.69 + 156.33 mm^2
= -128.37 mm^2

$(2 / 3 A_{vf} + A_t)$ = 2 / 3 x 524.47 + 156.33 mm^2
= 505.97 mm^2

$(0.04 f_c' / f_{sy}) b d$ = 0.04 x 0.8 x 35 / 500 x 1000 x 244
= 546.56 mm^2

Hence A_s = **546.56** 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 >= 0.25 * A s
>= 0.333 * A_{vf}

Provide the largest of these two magnitudes as Ah.

0.25 * A s = 0.25 x 546.56
= -142.34 mm^2

0.333*A_{vf} = 0.333 x 524.47
= 174.65 mm^2

Hence A_h = **174.65** mm^2

STEP VII

The total steel in vertical stirrups is " A_v "

V c	=	10 bd	in kgs	***where b & d are in cms***
	=	10 x 100 x 24.4		
	=	24400.00	kg	
	=	244.00	kN	
Pitch	=	200	mm	

$$A_v = \frac{0.50 * (V_u - V_c) * p}{f_{sy}\, d}$$

$$= \frac{0.5 \times (312.06 - 244) \times 200 \times 1000}{500 \times 244}$$

A_v = **55.79** mm^2

Reinforcement Details

Total Primary Tensile Reinforcement " A_s "

A_s	=	**546.56**	mm^2			
Diameter of Primary Steel	=	12	mm			
Area of one bar	=	113.10	mm^2			
Spacing of Bar	=	**150**	mm c/c			
$A_{s\ provided}$	=					
	=	753.982	mm^2	>	A_s	R/F is adequate

provide	**150 mm c/c 12 mm diameter bars as main reinforcement**	

Horizontal Steel (Closed Stirrups) " A_h "

A_h	=	**174.65**	mm^2
A_h **/m**	=	**174.65**	mm^2 /m

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

2 / 3 d	=	(2 / 3) x 244				
	=	163.00	mm			
Diameter of Stirrup Bar	=	12.00	mm			
Area of one bar	=	113.10	mm^2			
No. of layers of Stirrups	=	2.00	nos.			
Spacing of Stirrups	=	81.500	mm			
No. of legs of stirrups	=	2.00				
$A_{h\ provided}$	=	113.1 x 2 x 2				
	=	452.389	mm^2	>	A_h	R/F is adequate

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

Vertical Steel (Closed Stirrups) " A_v "

A_v	=	**55.79**	mm^2			
A_v	=	0.25 x As	=	136.64	mm^2	
Av max	=	136.64				
A_v **/m**	=	**136.64**	mm^2 /m			
Diameter of Stirrup Bar	=	10.00	mm			
Area of one bar	=	78.54	mm^2			
Pitch	=	200	mm			
No. of legs of stirrups	=	2.00				
$A_{h\ provided}$	=					
	=	785	mm^2	>	A_v	R/F is adequate

provide	**2 legged. 10 mm diameter bars as vertical stirrups at 200 mm spacing per meter width**	

DESIGN OF DIRT WALL

Dirt wall will be designed as a vertical cantilever.

1.) NORMAL CASE

1a. Dead Load

Self Weight of Dirt Wall	=	2.499	m^3 x	25.00	=	62.466	kN
Self Weight of Dirt Wall/ m	=	62.466 /		11.24	=	5.557	kN

1b. Live Load

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

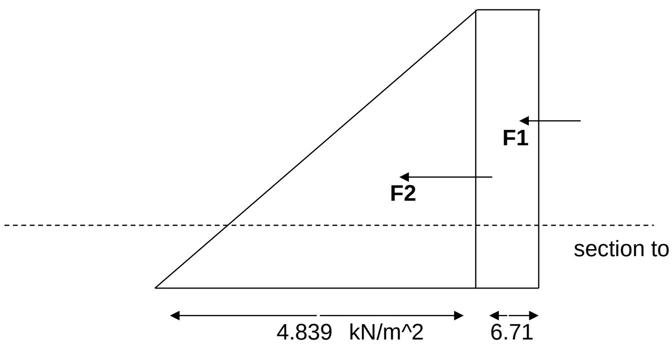
Vertical Load on Dirt Wall = 200 kN

Braking Load

Assuming 20% braking Force i.e. 0.2 *200	=	40.000	kN	acting at 1.2 m above deck
Effective Width	=	2.79	m	

Moment (due to Breaking) = $\frac{40.000 \times 2.066}{2.79}$ = 29.620 kNm/m

1c. EARTH PRESSURE



Height of dirt wall = 0.866 m

Normal Earth Pressure

Earth Pressure Diagram

Intensity for rectangular portion	=	0.279	x 20	x 1.2		=	6.705	kN/m^2
F1	=	6.705	x 0.87	x 1.00		=	5.807	kN/m
Intensity for triangular portion	=	0.2794	x 20	x 0.866		=	4.839	kN/m^2
F2	=	0.50	x 4.84	x 0.866	x 1.00	=	2.095	kN/m

Moment @ RL **1815.63** m (at dirt wall base)

M1	=	5.807	x	0.433	=	2.514	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	=	2.095	x	0.364	=	0.762	kN.m/m

Design Horizontal Forces (Normal Case):

Load Factor For Live Load Surcharge	=	1.2	
Ultimate Moment due to Live Load Surcharge	=	3.017	kN.m/m
Load Factor For Earth Pressure	=	1.5	
Ultimate Moment due to Earth Pressure	=	1.143	kN.m/m
Load Factor For Braking Force	=	1.5	
Ultimate Moment due to Braking Force	=	44.430	kN.m/m
Total Ultimate Moment	**=**	**48.590**	**kN.m/m**

Material Property:

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	N/mm^2
Design Yield Strength of Reinforcement, fyd	=	434.783	N/mm^2
Modulus of Elasticity of Steel (Es)	=	200000	N/mm^2

(a) Vertical steel on earth face As per Clause 16.3.1 of IRC:112-2011

Adorting clear cover on either face	=	50 mm					
Minimum Dia of Reinforcement	=	12 mm					
Maximum Spacing of Steel	=	200 mm					
Thickness of dirtwall	=	0.300 m					
Available effective depth	=	300	-50	-6	=	244	mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 48.59 kNm/m

Effective Depth of Cap Required (dreq) = $SQRT\left(\frac{48.59 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$ = 91.727 mm

Total Depth Required (Dreq) = 147.73 mm

Total Depth Provided (Dprov) = 300.00 mm **OK**

R= Mu/(b d^2) = 0.816

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$ = 0.002

Ast_{req} = 470.803 mm^2/m

As per Clause 16.3.1 of IRC:112-2011

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

Maximum (Ast_{req}, Ast_{min}) = 470.803 mm^2/m

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

Provide Ast = 565 mm^2/m) OK

Percentage of Steel Provided = 0.232 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035\ .\ d}{(0.0035 + fyd/ Es)}$ = $\frac{0.0035 \times 244}{0.0035 + 0.00217}$

= 150.5134 mm

Depth of Neutral Axis , = $\frac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$

= $\frac{435 \times 565}{0.36 \times 35.00 \times 1000}$ = 19.523 mm **OK**

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

z = d - 0.416 . X = 244 - 8.121

= 235.879 mm

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

MR = fyd . Ast . z = 434.78 x 565 x 235.88

= 5.8E+07 Nmm = 57.994 kNm/m > 48.59 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	117.701	mm^2/m
Governing Ast	=	150.000 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 RCC SOLID SLAB

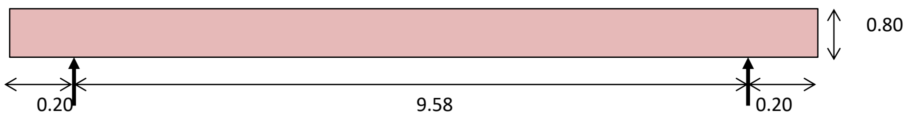
SPAN - 10.00m (SKEW 0^o)

OVERALL DECK WIDTH –11.0 m

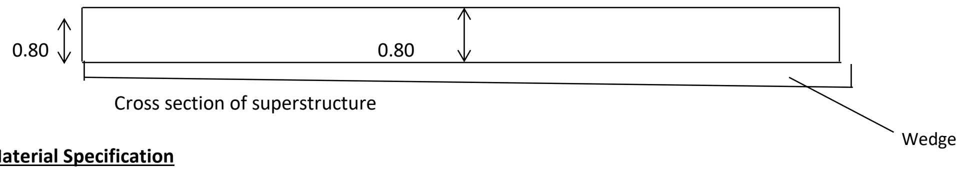
INPUTS FOR DESIGN OF RCC SOLID SLAB

1.0 Input Data :

Clear span of the slab		=	9.18	m			
Bearing over support		=	0.40	m			
Span between Centre to centre of bearin;		=	9.58	m			
Thickness of filler type exp. Gap		=	0.02	m			
Overall span of slab		=	10.00	m			
Overall width of slab		=	11.00	m			
Carriageway width		=	10.00	m			
Width of footpath		=	0.00	m			
Camber		=	2.50	%(uni directional)			
Depth of Slab at Carriageway edge		=	0.800	m			
Depth of Slab at Carriageway centre		=	0.800	m			
Wearing Coat thickness		=	0.065	m			
Clear cover to steel		=	0.050	m			
Diameter of main reinforcement bar		=	0.020	m			
Avg. Depth of Solid Slab		=	0.80	m			
Effective Depth of Slab		=	0.80	-	0.05	-	0.010
		=	0.740	m			
Effective Span	Min(l+ws , l+d)						
	L	=	9.58	m			



Span Arrangment of superstructure



Cross section of superstructure

2.0 Material Specification

Concrete Grade	=	M	30	
Characteristic Compressive Strength of Concrete,fck	=	30.00	Mpa at 28 days	
Design Compressive strength of Concrete,fcd	=	13.40	Mpa at 28 days	(0.67/1.5 * fck)
Tensile strength of concrete , fctm	=	2.50	MPa	
Strain at reaching Characteistic Strength, ec2	=	0.02		
Ultimate Strain, ecu2	=	0.035		
Modulus of Elasticity of Concrete (Ec)	=	2.74E+04	N/mm^2	(5000 x sqrt (fck)
Ecm	=	3.12E+04	N/mm^2	
Steel Grade		Fe	500	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	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	=	14.4 N/mm^2
Maximum tensile stress in steel under rare combination	=	300	N/mm^2	

INPUTS FOR DESIGN OF RCC SOLID SLAB

3.0 Calculation of Load ,Bending Moment & Shear Force

3.1 Self-weight of Deck Slab

Average thickness of deck slab	=	0.80 m			
U.D.L. Due to deck slab weight	=	20.00 kN/m^2			
Bending Moment Deck Slab weight	=	229.44 kNm/m			
Reaction	=	100.00 kN/m			
Shear Force at deff section	=	81.20 kN/m			
Shear Force at mid-span	=	0.00			

3.1 SIDL1 Due to Crash Barrier & Railing

Due to crash barrier	=	2.00	x 8.00	=	16.00 kN/m
U.D.L due to crash barrier	=	16.00	/ 11.00	=	1.45 kN/m^2
Due to RCC Railing	=	2.00	x 0.00	=	0.00 kN/m
	=	0.00	/ 11.00	=	0.00 kN/m^2

Total SIDL1 due to crash barrier & railing	=	1.45 kN/m^2
Bending Moment due to SIDL1	=	16.69 kNm/m
Reaction	=	7.27 kN/m
Shear Force at deff section	=	5.91 kN/m
Shear Force at mid-span	=	0.00

3.2 SIDL2 Due to Wearing Course

due to wearing course and additional ove	=	2.00 kN/m^2
Bending Moment due to SIDL2	=	22.94 kNm/m
Reaction	=	10.00 kN/m
Shear Force at deff section	=	8.12 kN/m
Shear Force at mid-span	=	0.00

3.3 Footpath Load

Refer clause 206.3 of IRC:6-2014

P1	=	0 kg/m^2			
Effective span L	=	9.58 m			
Intensity of FPLL , P	=	P1	-	(40x L - 300)/9	
	=	0	-	9.24 =	-9.24 kg/m^2
P due to both side footpath	=	2	x	-9.24 =	-18.49 kg/m^2
Intensity of FPLL per running meter	=	-0.18	x	0.00 =	0 kN/m
U.D.L. Due to FPLL	=	0.00	/	11.00 =	0.00 kN/m^2
Bending Moment due to FPLL	=	0.00 kNm/m			
Reaction	=	0.00 kN/m			
Shear Force at deff section	=	0.00 kN/m			
Shear Force at mid-span	=	0.00			

3.4 CWLL Bending Moment

			interpolation	
b/L	=	1.15	1.10	2.24
α	=	2.30 For simply supported slab	1.20	2.36

beff = $\alpha * a * (1 - a / L) + b_1$

A = $\alpha * a * (1 - a / L)$

b_1 = Breadth of the concentration area of the load

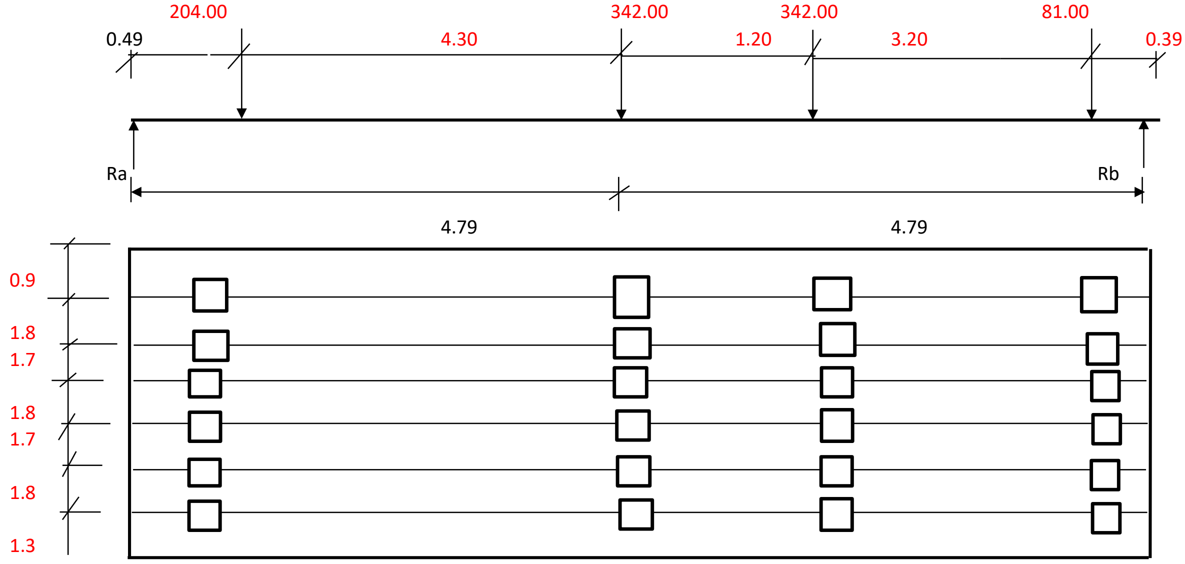
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
114.00	0.50	0.25
68.00	0.38	0.20
27.00	0.20	0.15

b_1	=	0.63	114 kN
b_1	=	0.51	68 kN
b_1	=	0.33	27 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

For Class A - 3 Lanes

I.F = 1.289

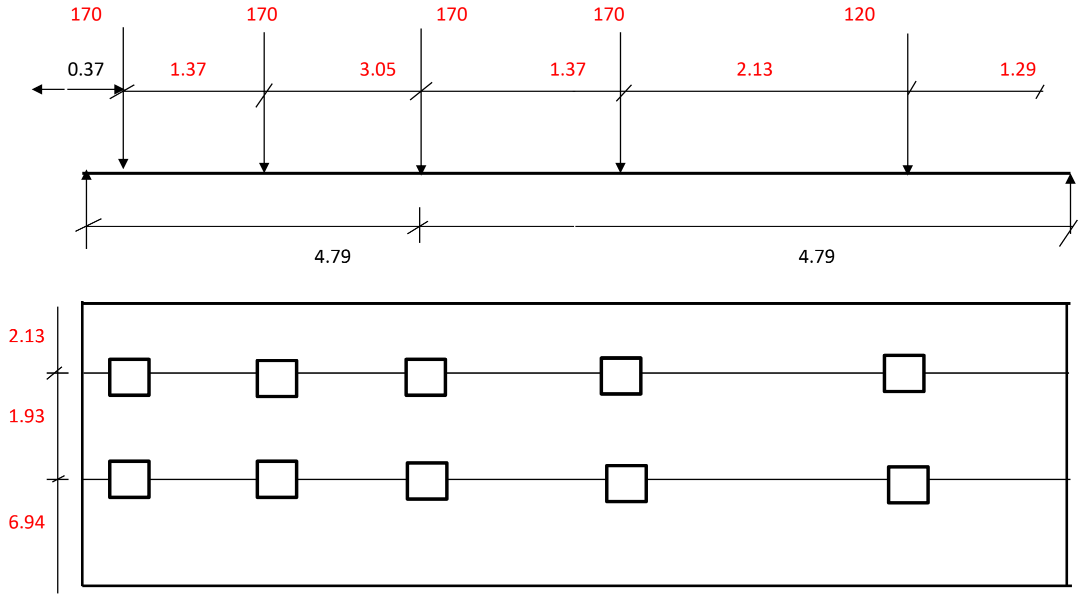


Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Bending Moment (kNm/m)
81.00	0.39	0.86	0.33	1.19	7.14	2.85
342.00	3.59	5.16	0.63	5.79	11.00	71.93
342.00	4.79	5.50	0.63	6.13	11.00	95.97
204.00	0.49	1.07	0.51	1.58	9.47	6.80
						177.55

For Class A - 3 Lanes	B . M =	**177.55** kN-m/m

CLASS 70 RW Loading

Impact Factor = 1.25



INPUTS FOR DESIGN OF RCC SOLID SLAB

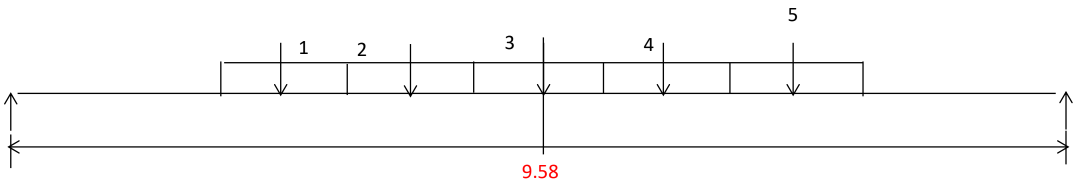
Dimensions of 70RWheel Tyre presssure = 5.273 kg/cm^2 527.30 kN/m^2

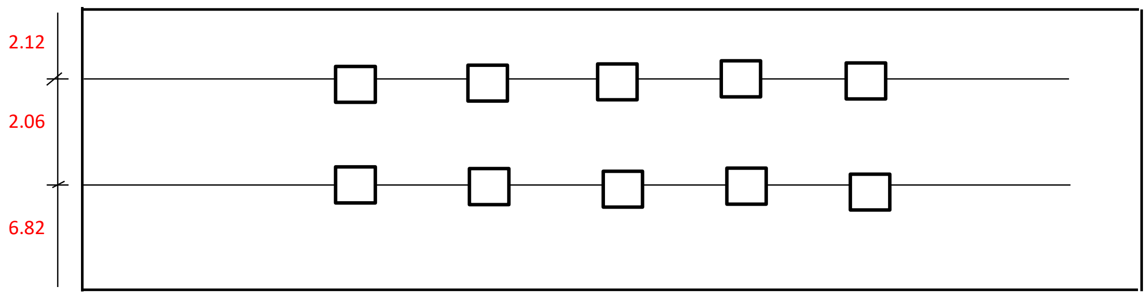
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
170.00	0.86	0.37
120.00	0.86	0.26

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
120	1.29	2.57	0.99	3.56	5.49	17.64
170	3.42	5.05	0.99	6.04	7.08	51.31
170	4.79	5.50	0.99	6.49	7.31	69.65
170	1.74	3.27	0.99	4.26	6.19	29.86
170	0.37	0.82	0.99	1.81	3.61	10.88
					B . M =	**179.34**

CLASS 70 R(T) Loading

Impact Factor = 1.10





Class 70R (T) = 700.00 KN
Length of track load = 4.57 m

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

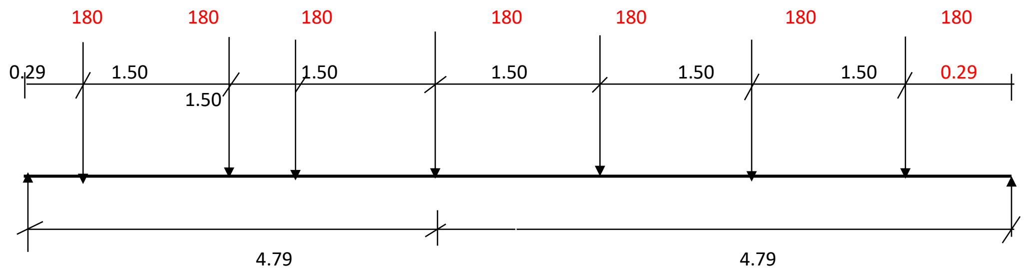
Divide the track load into 5 equal wheel load

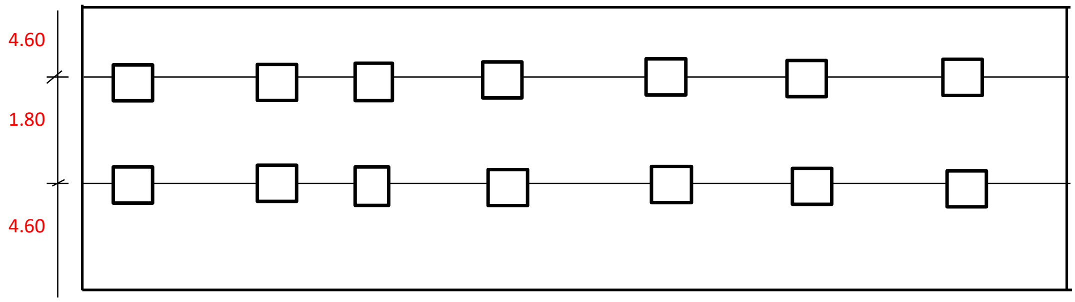
Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	2.96	4.70	0.97	5.67	7.02	32.51
140	3.88	5.30	0.97	6.27	7.32	40.79
140	4.79	5.50	0.97	6.47	7.42	49.73
140	3.88	5.30	0.97	6.27	7.32	40.79
140	2.96	4.70	0.97	5.67	7.02	32.51
					B . M =	**196.33**

INPUTS FOR DESIGN OF RCC SOLID SLAB

CLASS SV Loading

Impact Factor = 1.00





Dimensions of 70RWheel Tyre presssure = 2.250 kg/cm^2 225.00 kN/m^2

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	0.290	0.65	0.58	1.23	2.45	10.64
180	1.79	3.34	0.58	3.92	5.72	28.14
180	3.29	4.96	0.58	5.54	7.34	40.32
180	4.79	5.50	0.58	6.08	6.08	70.86
180	3.29	4.96	0.58	5.54	5.54	53.41
180	1.79	3.34	0.58	3.92	5.72	28.14
180	0.29	0.65	0.58	1.23	2.45	10.64
					B . M =	**242.17**

CWLL Bending Moment For

1.) CLASS A - 3 Lane	=	177.55	kN-m/m
2.) CLASS 70 R Wheel	=	179.34	kN-m/m
3.) CLASS 70 R Track	=	196.33	kN-m/m
4.) SV Loading	=	242.17	kN-m/m

INPUTS FOR DESIGN OF RCC SOLID SLAB

Total Bending Moment at Mid Span

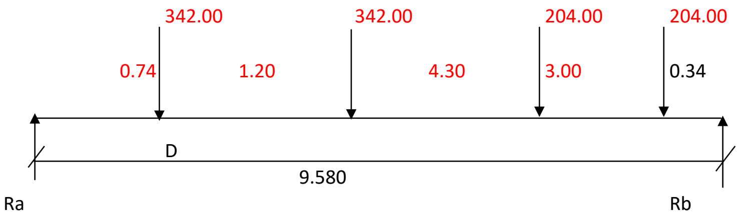
Loads	Unfactored B.M. (kNm/m)	ULS Factor	ULS Moment (kNm/m)	Rare Combination factor	Rare Combination Moment (kNm/m)	Quasi permanent combination Factor	Quasi permanent combination Moment (kNm/m)
DL	229.44	1.35	309.75	1.00	229.44	1.00	229.44
SIDL1	16.69	1.35	22.53	1.00	16.69	1.00	16.69
SIDL2	22.94	1.75	40.15	1.00	22.94	1.00	22.94
FPLL	0.00	1.50	0.00	1.00	0.00	0.00	0.00
CWLL - (Class A /70R)	196.33	1.50	294.49	1.00	196.33	0.00	0.00
CWLL - SV Load	242.17	1.00	242.17	1.00	242.17	0.00	0.00
Total Moment		=	**666.92**		**511.24**		**269.07**

3.5 CWLL Shear Force at a distance of Effective Depth from the centre of support

Thus Shear force at Point D = 73.46 kN (Including Impact Factor)

3- Lane Class A Load

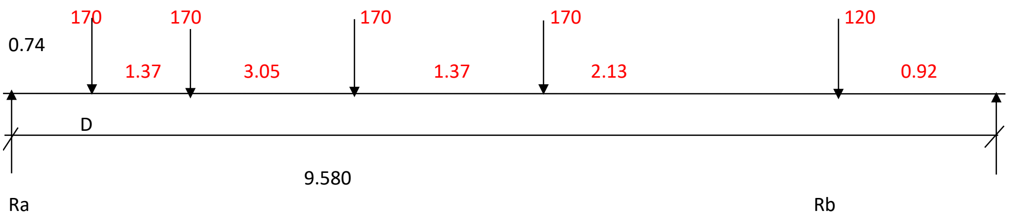
Effective Depth = 0.74 m
Distance from Support = 0.74 m



Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Shear Force (kN/m)
204.00	0.34	0.75	0.51	1.26	7.58	1.23
204.00	3.34	5.00	0.51	5.51	11.00	8.33
342.00	1.94	3.56	0.63	4.19	11.00	31.96
342.00	0.74	1.57	0.63	2.20	10.80	37.66
						79.18

Thus Shear force at Point D = 79.18 kN (Including Impact Factor)

1- 70 R Wheeled Load



Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Shear Force (kN/m)
120	0.92	1.91	0.99	2.90	4.83	2.98
170	3.05	4.78	0.99	5.77	6.94	9.74
170	4.42	5.47	0.99	6.46	7.29	13.45
170	2.11	3.78	0.99	4.77	6.45	25.71
170	0.74	1.57	0.99	2.56	4.49	43.68
						95.56

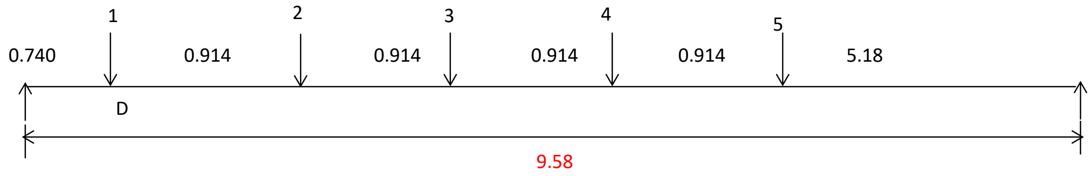
Thus Shear force at Point D = 95.56 kN (70 R Wheeled Load)

Governing Shear Force at Deff = 95.56 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

CLASS 70 R(T) Loading

Impact Factor = 1.10



Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

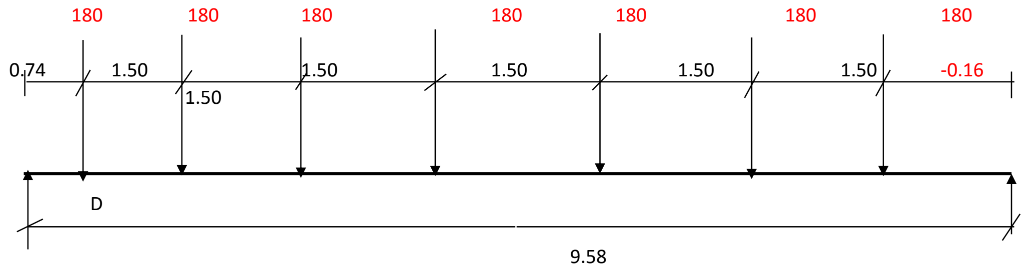
Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	0.74	1.57	0.97	2.54	4.60	18.12
140	1.65	3.14	0.97	4.11	6.17	15.88
140	2.57	4.32	0.97	5.29	6.82	16.52
140	3.48	5.09	0.97	6.06	7.21	17.67
140	4.40	5.47	0.97	6.44	7.40	19.21
					S.F =	**87.39**

Thus Shear force at Point D = 87.39 kN (70 R Track Load)

CLASS SV Loading

Effective Depth = 0.74 m

Distance from Support = 0.74 m



Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	-0.160	-0.37	0.58	0.21	0.41	-7.29
180	1.34	2.65	0.58	3.23	5.03	5.01
180	2.84	4.59	0.58	5.17	6.97	7.65
180	4.34	5.45	0.58	6.03	7.83	10.41
180	3.74	5.24	0.58	5.82	7.62	14.40
180	2.24	3.94	0.58	4.52	6.32	21.81
180	0.74	1.57	0.58	2.15	3.95	42.06
					S.F =	**94.05**

Thus Shear force at Point D = 94.05 kN (SV Load)

INPUTS FOR DESIGN OF RCC SOLID SLAB

CWLL Shear Force for

1.) CLASS A - 3 Lane	=	79.18	kN/m
2.) CLASS 70 R Wheel	=	95.56	kN/m
3.) CLASS 70 R Track	=	87.39	kN/m
4.) SV Loading	=	94.05	kN/m

Summary of Shear Force

Loads	S.F. At deff (kN/m)	ULS Factor	ULS-S.F. (kN/m)
DL	81.20	1.35	109.62
SIDL1	5.91	1.35	7.97
SIDL2	8.12	1.75	14.21
FPLL	0.00	1.50	0.00
CWLL - (Class A	95.56	1.50	143.34
CWLL - SV Load	94.05	1.00	94.05
Total			**275.15**

ULS DESIGN OF SOLID SLAB

DECK SLAB FOR ULS FLEXURAL MOMENT

Min. Thickness of slab	=	800	mm
Clear Cover to outer steeel	=	50	mm
Maximum Diameter of Reinformcement	=	20	mm
Effective Depth Provided (deff)	=	740	mm
Ultimate Design bending moment	=	**666.92**	kNm/m

Mulim = 0.167 x fck x b x d^2 = 666.92 kNm/m (Equation derived based on IRC:112-2011)

Effective Depth of Cap Required (dreq) = SQRT($\frac{666.92 \times 1000000}{0.167 \times 30.00 \times 1000}$)

Effective Depth of Cap Required (dreq)	=	364.852	mm	
Total Depth Required (Dreq)	=	424.85	mm	
Total Depth Provided (Dprov)	=	800.00	mm	**OK**

$R = M_{RD}/(b\ d^2)$ = 1.22

Ast Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d} = \frac{fck \{ 1 - sqrt(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 2178.876 mm^2/m

Minimum Longitudinal Reinforcement :

As. Min = 0.26 x $\frac{fctm}{fyk}$ x b . d - Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011

Whichever is higher OR = 0.0015 x b . d -Refer Clause 16.9 of IRC:112-2011'

	b	=	1000.00	mm
	d	=	740.00	mm
	Ast min	=	**1110.00**	mm^2/m
Governing Reinf. Ast		=	**2178.88**	mm^2/m

Provide 20 mm dia @ 200 mm c/c + 20 mm dia @ 190 mm c/c

Area provided= 3224.27 mm^2/m > 2178.88 mm^2/m OK

Percentage of Steel (pt%) = 0.44 %

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

Smax = 2 h = 1480.00

OR = 250.00 mm whichever is max

Provided Spacing is less than Smax, Hence OK

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035\ .\ d}{(0.0035 + fyd/ Es)}$ = $\frac{0.0035 \times 740.00}{0.0035 + 0.0022}$

= 456.48 mm

Depth of Neutral Axis , X = $\frac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$ = $\frac{434.78 \times 3224.27}{0.36 \times 30.00 \times 1000}$

= 129.80 mm **UNDER REINFORCED DESIGN, OK**

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

z = d - 0.416 . X = 740.00 - 54.00

= 686.00 mm

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

MR = fyd . Ast . z

= 434.78 x 3224.27 x 686.00

= 9.62E+08 Nmm / m

= 961.68 kNm / m > 666.92 kNm / m SAFE

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

ULS DESIGN OF SOLID SLAB

Distribution reinforcement:

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$\frac{20.00}{100.00}$ x 3224.27 = 644.853 mm^2/m

Provide	12	dia bar	150	direction in top face. (Providing =	753.98 mm^2)	OK

DESIGN OF SLAB FOR ULS SHEAR

Ultimate Design Shear Force = 275.15 kN/m

Design Shear Strength of Concrete,(τ_c) without Shear Reinforcement:

As per Clause 10.3.2 of IRC:112-2011,

Design shear resistance of the member without shear reinforcement is given by:

$$V_{Rd,c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right] b_w d \qquad \textbf{eq.1}$$

Subjected to minimum of

$$V_{Rd,c} = (V_{min} + 0.15\sigma_{cp}) b_w d \qquad \textbf{eq.2}$$

where,

K = 1 + SQRT(200/d) ≤ 2.0 , where d is depth in mm

K = 1.52

vmin = $0.031 K^{3/2} fck^{1/2}$, fck = 30.00 N/mm^2

Hence **vmin= 0.318 N/mm^2**

σ_{cp} = Concrete compressive stress in concrete at centroidal axis in the direction of axial load or prestressing

σ_{cp} = N_{Ed}/A_c < 0.2 f_{cd} where , f_{cd} = 0.67 fck/1.5

σ_{cp} = 0.00 N/mm^2

Hence,

τ_c = $V_{Rd,c}/(b_w.d)$ = $V_{min} + 0.15\ \sigma_{cp}$ = 0.3182 N/mm^2 **From eq.2**

ρ1 = Steel Ratio = Asl/(bw . d) ≤ 0.02

Hence ρ1 = 0.0044

τ_c = $V_{Rd,c}/(b_w.d)$ = 0.399 N/mm^2 **From eq.1**

Max of eq.1 & eq.2

τ_c =	**$V_{Rd,c}/(b_w.d)$ =**	**0.399**	**N/mm^2**

Corresponds to steel ratio = 0.436% & M30 Grade of Concrete

Shear stress(v_{Ed}) = V_{Ed}/(bw*d)

v_{Ed}	**=**	**$\frac{275145.78}{1000.00 \times 740.00}$**	**=**	**0.372 N/mm^2**	**<**	**0.399 MPa**

AsTv is lesser than Tc Hence No Shear Reinforcement is need to be provided.

CHECK FOR PUNCHING AROUND VEHICLE LOAD (as per IRC:112-2011, Clause 10.4.3):

Maximum load on each wheel	=	100 KN	(70 RW Boggie load)
Maximum tyre pressure	=	5.273 kg/cm^3	
Contact width perpendicular to span, L	=	0.86 m	
Contact width parallel to span,B	=	0.190 m	

Basic Equation for Punching shear stress(v_{Ed}) = $\frac{\beta\ V_{Ed,req}}{ui * d}$

Depth of Slab , d		=	740.00 mm		
Length of perimeter, ui		=	pi() * 4 d + L * 2 + B * 2 for Rectangular section	=	11398.40 mm
Loaded area under perimeter		=	pi() * (4 d)^2/4 + L * 2d*2 + B * 2d*2 + L * B =		1.02E+07 mm2
ΔVEd		=	0.00 for deck slab		
$V_{Ed,req}$	= V_{Ed} - ΔV_{Ed}	=	100.00 kN.		
β		=	1.00 for axial load without bending		

v_{Ed} = 0.01 N/mm^2

ULS DESIGN OF SOLID SLAB

Governing Punching Shear Resistance of Concrete VRd,c = As per IRC:112-2011, Clause 10.4.4

$$v_{Rd.c} = \frac{0.18}{\gamma_c} K\left(80\rho_1 f_{ck}\right)^{1/3} + \left(0.1\sigma_{cp}\right) \geq v_{\min} + 0.1\sigma_{cp}$$

where,

K	=	1.52			
$\rho 1$	=	0.0044			
fck	=	30.00	N/mm^2		
σ_{cp}	=	0.00	N/mm^2		
γc	=	1.50			
Vmin	=	$0.031 * k^{3/2} * fck^{1/2}$		=	0.318 N/mm^2
Vmin + 0.1 σcp	=	0.318			
$v_{Rd,c}$	**=**	**0.399**	**N/mm^2**	**OK**	

SINCE	**v_{Ed}**	<	**$v_{Rd,c}$**	**HENCE SAFE IN PUNCHING**

SLS DESIGN OF DECK SLAB

1. Early Age Check (before creep has taken place)
2.Long Term Check (after creep has taken place)

Maximum compressive stress in concrete under rare combination	0.48 . fck	=	14.4	N/mm^2
Maximum tensile stress in steel under rare combination		=	300	N/mm^2
Maximum Tensile stress in concrete	fctm	=	2.50	N/mm^2
Permissible crack width		=	0.3	mm

Section Properties of Uncracked Section:

Width of Section , b	=	1000.00	mm
Depth of Section , D	=	800.00	mm
Gross Cross sectional area , Ag	=	800000	mm^2
Gross Moment of Inertia Igxx	=	4.27E+10	mm^4
Gross Moment of Inertia Igyy	=	6.67E+10	mm^4

Modular Ratio : for Early Age Check

Modulus of elasticity of concrete, Eceff = Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2
Modular Ratio (m) = Es/ Eceff	=	6.41	

Modular Ratio: for Long Term Check

Perimeter of section (u)	=	3600	mm
2 Ac/u	=	444.444	mm
Modulus of elasticity of concrete, Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2

For Moist atmosperic condition

Creep coeff. ø = 1.30

$$Eceff = \frac{Ecm}{(1 + ø)} = 1.35E+04$$

Modular Ratio (m) = Es/ Eceff = **14.77**

STRESS CHECK IN CRACKED SECTION

Depth of neutral axis =

$$dc = \frac{-A_sE_s + \sqrt{(A_sE_s)^2 + 2bA_sE_sE_{c,eff}d}}{bE_{c,eff}}$$

Cracked Second Moment of Area **INA** =
(in steel units)

$$A_s(d-d_c)^2 + \frac{1}{3}\frac{E_{c,eff}}{E_s}bd_c^3$$

		Rare Case		Quasi-Permanent Case	
		For Early Age	For Long Term Check		
	unit	Sagging Moment	Sagging Moment	Sagging Moment	Remarks
SLS Moment	**kNm/m**	**511.237**	**511.237**	**269.072**	
Width of Section, b	mm	1000.00	1000.00	1000.00	
Depth of section , D	mm	800.00	800.00	800.00	
Effective cover , C_{eff}	mm	60.00	60.00	60.00	
Effective depth , d	mm	740.00	740.00	740.00	
Eceff	N/mm^2	31186.57	13537.58	31186.57	
Es	N/mm^2	200000.00	200000.00	200000.00	
Flexural Ast Provided , As	mm^2/m	3224.27	3224.27	3224.27	
dc		155.48	222.12	155.48	
Cracked Second Moment of area , I_{NA}	mm^4	1296976443.04	1112006587.06	1296976443.04	
section modulus, Zt = I_{NA} / dc	mm^3	8341981.95	5006315.33	8341981.95	
section modulus, Zb = I_{NA} / (d-dc)	mm^3	2218858.48	2147231.46	2218858.48	
Maximum compressive stress in concrete= M/Zt x	N/mm2	9.56	6.91	5.03	< 14.4 SAFE
Maximum Tensile stress in steel = M/Zb	N/mm2	230.41	238.09	121.27	< 300 SAFE

SLS DESIGN OF DECK SLAB

CRACK WIDTH CHECK Refer Clause 12.3.4 of IRC:112-2011

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

Where, Sr max = maximum crack spacing
εsm = mean strain in the reinforcement
εcm = mean strain in concrete between cracks

Spacing between reinf. = 5*(c+ϕ/2)	mm	300	
Spacing provided	mm	100	
Check for spacing criteria		OK	
$S_{r\,max} =$	3.4 c	+	$\frac{0.425\ k1\ k2\ \phi}{\rho_{\rho\ eff}}$
Clear Cover , c	mm	50	
Diameter of Main Bar , ϕ	mm	20	
Coefficient , k_1		0.8	
Coefficient , k_2		0.5	
Width of section , b	mm	1000	
Depth of section , D	mm	800	
Effective Depth of Section , d	mm	740	
Depth of Neutral axis , yt=dc	mm	155.476	
hc eff = Min of 2.5 (D - d) ,D - dc/3 , D/2	mm	150.00	
Ac, eff = b * hc,eff	mm^2	150000	
$\rho_{\rho\ eff} = As/A_{c\ eff}$		0.021	
Sr max	mm	328.176	
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - \frac{k_t\, f_{ct\ eff}\,(1 + \alpha_e\, \rho_{\rho\ eff})}{\rho_{\rho\ eff}}}{E_s}$	
Stress in tension steel , σsc (in Quasi-Permanent Case)	**N/mm2**	**121.27**	
Kt		0.500	
Tensile strength of concrete = fct eff = fctm	N/mm2	2.501	
a_e = Es/ Ecm		6.413	
$(\varepsilon_{sm} - \varepsilon_{cm})$		0.000	
Crack Width , Wk	**mm**	**0.09**	
Limited Carck width	**mm**	**0.30**	
Check for Crack width		**< 0.3 mm SAFE**	

DESIGN OF WALL TYPE ABUTMENT WITH OPEN FOUNDATION

Applicable For Following Bridges

1) Minor Bridge at Ch. 158+054
2) Minor Bridge at Ch. 163+794
3) Minor Bridge at Ch. 164+119
4) Minor Bridge at Ch. 164+833
5) Minor Bridge at Ch. 170+454

Details of Superstructure:

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

Radius of Curvature of Superstructure	=	0	m
Design speed of vehicle	=	100	kmph

		Right Dimensions	Skew Dimensions
Span -c/c of Brg.	=	9.580m	9.580m
Thickness of Expansion Joint	=	0.020m	0.020m
Slab projection Beyond C/L of Bearing (Back Side)	=	0.200m	0.200m
Slab projection Beyond C/L of Bearing (Span Side)	=	0.200m	0.200m
Span -c/c of E.J.	=	10.000m	10.00m
Type of Superstructure	=	RCC SOLID SLAB	
Width of Crash barrier (Both Side)	=	0.500m	
Width of Carriageway	=	7.500m	
Projection beyond crash barrier	=	0.000m	
Thickness of Wearing coat	=	0.065m	
Length of Approach Slab (Right)	=	3.500m	3.500m
Width of Footpath on both side	=	3.000m	
Railing/kerb on footpath edge	=	1.000m	
Total Width of Superstructure	=	12.500m	
Median Width minus 20mm gap	=	0.480m	

Bearings

Type of Bearing	=	Tar Paper Bearing	
Coeff. Of Friction for POT-PTFE Bearing	=	0.5	
Type of Soil	=	1	Hard or Rocky Strata
NBC of soil -Normal Case	=	250	kN/m^2 (as per geotechnical report with ground improvement)
SBC of soil-Normal Case	=	280	kN/m^2
SBC of soil-Seismic Case	=	350	kN/m^2
Coeff. of friction between concrete and soil	=	0.7	for weathered rock
Permissible FOS against Sliding	=	1	Normal Case
	=	1	Seismic Case
Permissible FOS against Overturning	=	1	Normal Case
	=	1	Seismic Case

Dirt Wall

		Right Dimensions	Skew Dimensions
Width of Dirt wall at Top	=	0.300m	0.300m
Width of Dirt wall at Bottom	=	0.300m	0.300m
Height of Uniform portion	=	0.600m	
Height of Trapering portion	=	0.122m	
Length of Dirt Wall at top (Uniform portion)	=	12.740m	12.740m
Length of Dirt Wall at bottom (Tapering Portion)	=	12.740m	12.740m

Abutment Cap

Width of Abutment cap of Uniform portion	=	0.720m	0.720m
Width of Abutmentcap at bottom of Tapering Portion	=	0.720m	0.720m
Projection of Abutment Cap (Span Side)	=	0.000m	0.000m
Projection of Abutment Cap Back Side	=	0.000m	0.000m
Abutmentcap thickness (Uniform portion)	=	0.300m	
Abutmentcap thickness (Tapering Portion)	=	0.000m	
Length of Abutment Cap at top (Uniform portion)	=	12.740m	12.740m
Length of Abutment Cap at bottom (Tapering Portion)	=	12.740m	12.740m

Abutment- Wall Type

Thickness of Abutment	=	0.720m	
Width of abutment shaft	=	12.740m	12.740m
Thickness of Abutment shaft at Top	=	0.720m	0.720m
Thickness of Abutment shaft at HFL	=	0.818m	0.818m
Thickness of Abutment shaft at Bottom	=	0.900m	0.900m

Solid Return Wall

Length of Return wall	=	3.600m
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	=	0.600m
Height of wall at abutment	=	2.667m
Length of Return wall	=	4.001m
Thickness of Return wall at Top	=	0.500m
Thickness of Return wall at Bottom	=	0.500m

Foundation

Along Traffic Direction:

Total Width of Footing	=	7.000m
abutment pedestal width	=	0.900m
abutment pedestal Height	=	0.000m
Width of Toe Slab	=	2.500m
Width of Heel Slab	=	3.600m
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	=	3.600m
Thickness of heel slab at back fill edge	=	1.000m
Height of back fill at bottom edge of heel slab	=	7.203m
Height of back fill at back fill edge of heel slab	=	6.503m

Across Traffic Direction:

Width of foundation -Uniform portion	=	12.740m (skew dimension)
Width of foundation -Tapering portion	=	12.740m (skew dimension)

Sr. No.	Structure	Chainage	FRL	GRL	FND. LVL	FRL-FND.LVL
1	Minor Bridge	158+054	1842.971	1839.102	1836.102	6.869
2	Minor Bridge	163+794	1757.028	1752.415	1749.415	7.613
3	**Minor Bridge**	**164+119**	**1757.809**	**1753.162**	**1750.162**	**7.647**
4	Minor Bridge	164+833	1756.837	1753.275	1750.275	6.562
5	Minor Bridge	170+454	1664.704	1661.386	1658.386	6.318

Levels

Deck Level at Median Edge=	**1757.809m**	Cross Slope (Bi-directional)	=	2.500%
Deck level at Outer Edge =	**1757.522m**	Height of Superstructure	=	0.800m
Deck level at center line =	1757.809m	Min. Height of Footpath Side Pedestal (1)	=	0.000m
Soffit Level at center of bridge =	1756.944m	Height of Pedestal (2)	=	0.000m
Abutment cap top level =	1756.943m	Height of Pedestal (3)	=	0.000m
Abutment cap bottom lvl (uniform portion ends	1756.643m	Height of Pedestal (4)	=	0.000m
Abutment cap bottom lvl (corbel portion ends)	1756.643m	Distance of nearest girder to c.l. of deck	=	0.000m
Abutment shaft top level =	1756.643m	Height (Avg.) of Dirt Wall	=	0.722m
Ground level/LBL =	1753.162m	Abutment shaft Above G.L	=	3.481m
Abutment shaft bottom level =	1751.162m	Abutment Shaft below G.L	=	2.000m
Foundation level =	1750.162m	Height of abutment shaft	=	5.481m
HFL	1753.662m	MSL	=	1753.162m
		Wedge over girder flange	=	0.0020m

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 Characteistic Strength, $\varepsilon c2$	=	0.02		
Ultimate Strain, $\varepsilon cu2$	=	0.035		
Ecm	=	32308.250	N/mm^2	
Steel Grade	=	Fe	500D	(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	200000.00	Mpa	
Dry weight of Concrete	=	25	kN/m^3	
Dry unit weight of soil	=	20	kN/m^3	
Permissible Crack Width	=	0.3	mm - For Moderrate/ severe Exposure Condition	
Maximum compressive stress in concrete under rare combination	=	0.48 fck		
	=	16.8	N/mm^2	
Maximum tensile stress in steel under rare combination	=	300	N/mm^2	

<u>Creep Coefficient</u>

For Abutment Shaft	=	1.2	for 365 days
For Footing	=	1.2	for 365 days

<u>Clear Cover to Reinforcement</u>

Earth Face	=	75	mm
Non-Earth Face	=	50	mm

<u>Seismic Data:</u> NO NEED TO CHECK FOR SEISMIC EFFECT

Seismic Zone		=	5	
Z	=Zone factor	=	0	
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	=	30
θ	=	90
β	=	0
δ	=	20.0

REACTION FROM SUPERSTRUCTURE (in kN)

Dist between c.g of Bearing and c.g. of abutment shaft	=	0.160m	in longitudinal direction
Dist between c.g of superstructure and c.g. of abutment shaft	=	0.120m	in Transverse direction
C.G. of crash barrier above deck level	=	0.449m	

From Superstructure analysis

Dead Load

Self weight of Slab	=	0.80	x	10.00	x	12.50	x	25.00
	=	2500.00	KN					
Reaction at one end	=	1250.00	KN					
Transverse Eccentricity	=	0.000	m					

Super Imposed Dead Load Reactions (Excluding Wearing Course)

Weight of Crash barrier	=	2	x	8.00	x	10.00
	=	160.00	KN			
Reaction at one end	=	80.00	KN			
Transverse Eccentricity	=	0.00	m			

Reaction Due to Wearing Course only

Weight due to Wearing Coat	=	2.2	x	10	x	12.5
	=	275	KN			
Reaction at one end	=	137.5	KN			
Transverse Eccentricity	=	0.00	m			

Carriageway Live Load Reactions

Reduction Factor = **0.9** (for 3 Lane)

Congestion factor = **1** (As per Table 3 of IRC :112-2014)

MAXIMUM REACTION CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.33	2.41

Min CWLL

Vertical	Transverse ecc
466.27	3.11

SV Loading

Max CWLL

Vertical	Transverse ecc
2561.34	0.30

Min CWLL

Vertical	Transverse ecc
858.66	0.30

MAXIMUM TRASVERSE MOMENT CASE:

1- 70RW + 2-CLASS A

Max CWLL

Vertical	Transverse ecc
932.33	2.41

Min CWLL

Vertical	Transverse ecc
466.27	3.11

Impact Factor for 70R Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

Impact Factor for Cl A Wheeled loading

Impact Factor upto abut. cap	=	1.144
Impact Factor for Abut. Shaft Base	=	1.000

VOLUME CALCULATION

C.G. Of Footing = 3.500 m
C.G. Of shaft from toe tip = 2.950 m
Distance between c.g. of shaft and footing = 0.550 m

Description		No.	LENGTH	WIDTH	HEIGHT	VOLUME	Ecce.(eL) @ abut. Shaft	Ecce.(eL1) @ c.g.of footing	Ecce.(eL2) @ Toe	Trans. Ecc (eT)
			m	m	m	m^3	m	m	m	
Dirt Wall -Uniform portion		1	12.74	0.300	0.600	2.293	-0.210	0.340	-3.160	0.000
-Trapering portion		1	12.74	0.300	0.122	0.467	-0.210	0.340	-3.160	0.000
Bracket (Rectangle)		1	12.74	0.300	0.300	1.147	-0.510	0.040	-3.460	0.000
(Corbel)	0.5	1	12.74	0.300	0.300	0.573	-0.460	0.090	-3.410	0.000
Cap (uniform portion)		1	12.74	0.720	0.300	2.752	0.000	0.550	-2.950	0.000
Cap (Corbel Portion)		1	12.74 12.74	0.720 0.720	0.000	0.000	0.000	0.550	-2.950	0.000
Shaft above HFL		1	12.74	0.769	2.981	29.203	0.065	0.615	-2.885	0.000
Shaft below HFL		1	12.74	0.859	2.200	24.075	0.020	0.570	-2.930	0.000
Solid Return Wall		2	3.60	0.500	6.997	25.189	-2.250	-1.700	-5.200	0.000
Cantilever Return wall(Rectangular portion)		2	4.00	0.500	0.600	2.400	-2.450	-1.900	-5.400	0.000
Cantilever Return wall(Traingular portion)		2	4.00	0.500	2.067	4.135	-1.784	-1.234	-4.734	0.000
Footing										
Heel Slab		1	12.74	3.600	0.650	29.812		-1.377	-4.877	0.000
Toe Slab		1	12.74	2.500	0.650	20.703		2.026	-1.474	0.000
Portion between Heel and Toe		1	12.74	0.900	1.000	11.466		0.550	-2.950	0.000
Back filling above HFL over Heel Slab		1	12.74	3.600	4.147	190.198		-1.700	-5.200	0.000
Back filling below HFL over Heel Slab		1	12.74	3.600	2.850	130.712		-1.774	-5.274	0.000
Backfill above Heel slab		1	12.74	3.600	6.853	314.317		-1.731	-5.231	0.000
Front Filling over Toe Slab		1	12.74	2.500	2.350	74.848		2.188	-1.312	0.000
Side filling between heel and toe		1	0.00	0.900	2.350	0.000		0.000	0.000	0.000
Aproach Slab		1	12.740	1.750	0.300	6.689	-0.510	0.040	-3.460	0.000
Back fill above HFL on flared portion of stem		1	12.74	0.098	4.147	5.172		0.169	-3.331	0.000
Back fill below HFL on flared portion of stem		1	12.74	0.082	2.850	2.981		0.127	-3.373	0.000

			L			eL	eL1	eL2
RCC Railing/Parapet Wall Weight/Crash Barri	2	8 kN/m	1.750	28.00kN		-0.210	0.340	-3.160

SECTIONAL PROPERTIES

Width of Footing (B) = 7 m
Length of Footing (L) = 12.740 m

A = 7.000 x 12.740 = 89.180 m^2
ZL = 12.740 x 8.167 = 104.043 m^3

$$ZT = \frac{IT1 + IT2}{\text{distance of extreme point from centre}}$$

IT1 = 7.000 x 172.317 = 1206.22 m^4
IT2 (moment of inertia of triangle) = 7.000 x 0.000 + 0.500 x 7.000 x 0.000 x 40.577
from centre of footing = 0.000 m^4

Moment of inertia of two triangle = 0.000 m^4

Total moment of inertia = 1206.22 m^4

Distance of extreme point from centre of footing = 6.370 + 0.000 = 6.370 m

Total Section modulus (ZT) = 189.359 m^3

Load Factors (As per IRC:6-2014)

Table 3.1 Partial Safety Factor For Verification of Equilibrium

-Refer Table 3.1 of IRC:6-2014

Loads	Basic Combination		Seismic Combination	
	Overturning or Sliding	Restoring or Resisting	Overturning or Sliding	Restoring or Resisting
Dead Laod,SIDL & Backfill except wearing course	1.050	0.950	1.050	0.950
Wearing Course only	1.350	1.000	1.350	1.000
Earth Pressure due to back filling	1.500	-	1.500	-
Carriageway Live Load	1.500	0.000	0.000	0.000
Live Load Surcharge	1.200	0.000	0.000	0.000
Seismic Effect (During Service)			1.500	0.000
Seismic Effect (During Construction)			0.750	0.000

Table 3.2 Partial Safety Factor For Verification of Structural Strength: Ultimate Limit State

-Refer Table 3.2 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.500	1.00
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL(Service)	1.500	0.20
CWLL and Associate load and FPLL(Construction)	1.350	1.00
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Table 3.3 Partial Safety Factor For Verification of Servicibilty Limit State

-Refer Table 3.3 of IRC:6-2014

Loads	Rare Combination	Frequent Combination	Quasi-Permanent Combination
Dead Laod+SIDL including wearing course	1.000	1.00	1.00
wearing course	1.200	1.20	1.20
Back Filling Weight	1.000	1.00	1.00
Shrinkage Creep Effect	1.000	1.00	1.00
Earth Pressure due to back filling	1.000	1.000	1.000
CWLL and Associate load and FPLL	1.000	0.750	0.000
Live Load Surcharge	0.800	0.00	0.00

Table 3.4 Partial Safety Factor For Design of Foundation

-Refer Table 3.4 of IRC:6-2014

Loads	Basic Combination	Seismic Combination
Dead Laod+SIDL except wearing course	1.350	1.35
Wearing Course only	1.750	1.75
Back Filling Weight	1.350	1.35
Earth Pressure due to back filling	1.500	1.000
CWLL and Associate load and FPLL	1.500	0.75
Live Load Surcharge	1.200	0.20
Seismic Effect (During Service)		1.50
Seismic Effect (During Construction)		0.75

Possible Load Combination

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

Seismic Coefficient Calculation

(As Per IRC:6-2014 , Clause 219)

Horizontal Seismic Force For Zone 5.0

Feq = Seismic forces to be resisted
Feq = Ah x (Dead load + Appropriate Live load)
Ah = horizontal seismic coefficient

$$= \frac{\frac{Z}{2} \quad \frac{Sa}{g}}{\frac{R}{I}}$$

Z	=	Zone factor	=	0	
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.

$$= 2.0 \left(\frac{D}{1000F}\right)^{1/2}$$

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

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

C.g. of Horizontal Force acting at a height from Foundation Level in Longitudinal direction
= 6.781 m
C.g. of Horizontal Force acting at a height from Foundation Level in Tranverse direction
= 7.358 m

Abutment Cap Top Level - Foundation Level
= 6.781 m

Dimensions of Abutment Shaft

Length = 12.74 m
Width = 0.81 m

Moment of Inertia , $I_{longitudinal}$ = 0.564 m^4
Moment of Inertia , $I_{transverse}$ = 139.576 m^4

Ecm = 3.231E+07 kN/m^2

Longitudinal Direction				**Transverse Direction**			
Force	=	87.693	KN	Force	=	19237.036	KN
D	=	1467.50	KN	D	=	1747.220	KN
T	=	0.2587	sec	T	=	0.0191	sec

Hard or Rocky Strata

Sa/g = 2.5 Sa/g = 2.5

Seismic Coeff. In Longitudinal Direction = 0

Seismic Coeff. In Transverse Direction = 0

Summary of Horizontal and Vertical Seismic Coeff.

For Design of Substructure

Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

For Design of Foundation (35% increment in Seismic Coeff for Foundation as per IRC:6-2014,Clause No. 219.8)

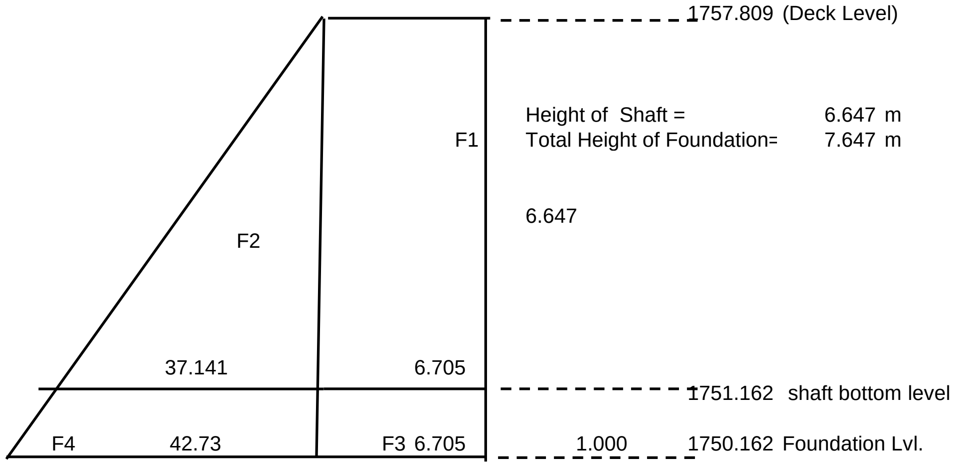
Ah	=	0.000	In Longitudinal direction
Ah	=	0.000	In Transverse direction
Av	=	0.000	In Vertical direction

Earth Pressure : Normal Dry Case

Properties of backfill material :	c	=	0		
	ϕ	=	30 degree	0.524	radians
	θ	=	90.00 degree	1.571	radians
	$\theta 1$	=	90.00 degree	1.571	radians
	β	=	0	0	radians
	δ	=	20.0 degree	0.349	radians
	Kah	=	0.279 active component		
	Kph	=	3.766 Passive component		
	γ	=	20 kN/m^3		

Equivalent Live Load Surcharge height = 1.2 m

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1751.162 m (at Shaft Base)

@ RL 1750.162 m (at Foundation Level)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.2	=	6.705	kN/m^2
F1	=	6.705	x	6.647	x	12.740	=	**567.816**	**kN**
M1	=	567.82	x	3.32	=	**1887.135**	**kN.m**	**at Shaft Bottom**	
F3	=	6.705	x	7.647	x	12.740	=	**653.240**	**kN**
M3	=	653.240	x	3.823	=	**2497.663**	**kN.m**	**at Foundation**	

Due to Active Earth Pressure

Intensity for triangular portion (At Shaft bottom level)

	=	0.279	x	20	x	6.647	=	37.141	kN/m^2
F2	=	0.5	x	37.14	x	6.647	x	12.74	
	=	**1572.613 kN**							

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

M2	=	1572.61	x	2.79	=	**4390.326**	**kN.m**	**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	0.279	x	20	x	7.647	=	42.729	kN/m^2
F4	=	0.5	x	42.73	x	7.647	x	12.74	
	=	**2081.386 kN**							
M4	=	2081.39	x	3.21	=	**6684.870**	**kN.m**	**at Foundation**	

Force Due To Fluid Pressure

As per Cl. 214.1 of IRC :6 -2014 γ fluid = 4.8 kN/m^3

Intensity for triangular portion (At Shaft bottom level)

	=	4.800	x	6.647	=	31.906 kN/m^2		
F	=	0.5	x	31.906	x	6.647	x	12.740
	=	**1350.927 kN**						

M	=	1350.93	x	2.216	=	**2993.205 kN.m**		**at Shaft Bottom**

Intensity for triangular portion (At Foundation level)

	=	4.800	x	7.647	=	36.71 kN/m^2		
F	=	0.5	x	36.706	x	7.65	x	12.740
	=	**1787.981 kN**						
M	=	1787.98	x	2.549	=	**4557.563 kN.m**		**at Foundation**

Intensity of Passive pressure

=	3.766 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 12.74
=	**0.000 kN**			

Moment due to passive @ Foundation, M

=	0.000 x	0.000 =	**0.000 kN.m**	**at Foundation**

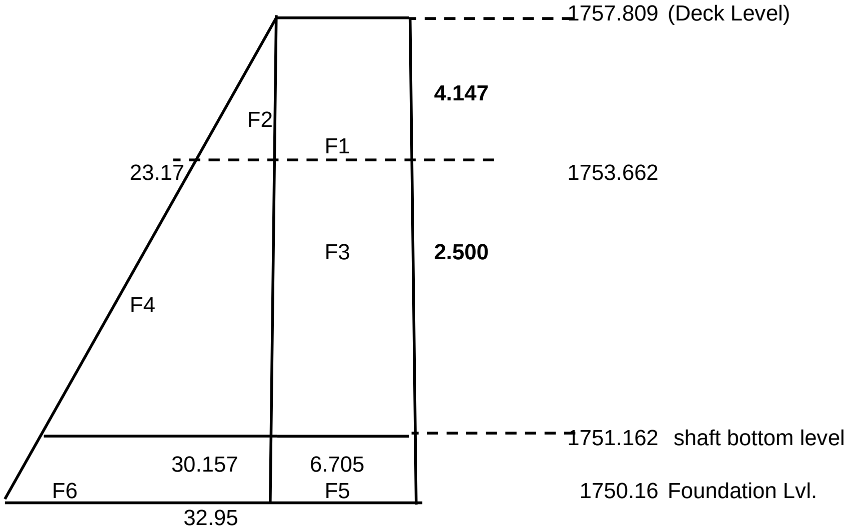
Summary of Moment and Horizontal Force

	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	4390.326	6684.870	1572.613	2081.386
Due to Minimum Fluid Pressure	2993.205	4557.563	1350.927	1787.981
Governing of Two	**4390.326**	**6684.870**	**1572.613**	**2081.386**
Due to Live Load Surcharge	**1887.135**	**2497.663**	**567.816**	**653.240**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Properties of backfill material :	c	=	0	
	ϕ	=	30 degree	0.524 radians
	θ	=	90.00 degree	1.571 radians
	β	=	0	0 radians
	δ	=	20.0 degree	0.349 radians
	Kah	=	0.279 active component	
	Kph	=	3.766 passive component	
	γd	=	20 kN/m^3	
	γwater	=	10 kN/m^3	
Equivalent Live Load Surcharge height		=	1.2 m	

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL 1751.2 m (at Shaft Base)

Due to Live Load Surcharge

Intensity for rectangular portion	=	0.279	x	20	x	1.200	=	6.705 kN/m^2	
F1	=	6.705	x	6.647	x	12.740	=	**567.816 kN**	
M1	=	567.82	x	3.32	=	**1887.135 kN.m**		**at Shaft Bottom**	
F3	=	6.705	x	7.647	x	12.740	=	**653.240 kN**	
M3	=	653.24	x	3.82	=	**2497.663 kN.m**		**at Foundation Level**	

Due to Active Earth Pressure

Intensity for triangular portion

Upto HFL	=	0.279	x	20	x	4.147	=	23.172 kN/m^2	
(At Shaft bottom level) Below HFL	=	0.279	x	10	x	2.500	=	6.985 kN/m^2	
F2	=	0.5	x	23.17	x	4.147	x	12.74	
	=	**612.123 kN**							

$$F4 = \frac{(23.17 + 30.16)}{2} \times 2.50 \times 12.74$$

= **849.260 kN**

Total Force = 1461.383 kN

M2	=	612.12	x	4.24	=	**2596.465 kN.m**
M4	=	849.26	x	1.20	=	**1015.230 kN.m**

Total Mome = 3611.69 kN.m at Shaft Bottom

Intensity for

triangular portion

Upto HFL	=	0.279	x	20	x	4.147	=	23.172	kN/m^2
at Foundation level	=	0.279	x	10	x	3.500	=	9.778	kN/m^2

F2 = 0.5 x 23.17 x 4.147 x 12.74
= **612.123 kN**

F6 = (23.17 + 32.95) / 2 x 3.50 x 12.74
= **1251.253 kN**

Total Force = 1863.376 kN

M2 = 612.12 x 5.24 = **3208.588 kN.m**

M6 = 1251.25 x 1.65 = **2062.520 kN.m**

Total Mome = 5271.11 kN.m Foundation Lvl.

Intensity of Passive pressure:
= 3.766 x 10 x 0.00 = 0.000 kN/m^2
Force due to passive @ Foundation, F
= 0.5 x 0.000 x 0.00 x 12.74
= **0.000 kN**

Moment due to passive @ Foundation, M
= 0.000 x 0.000 = **0.000 kN.m** **Foundation Lvl.**

Summary of Moment and Horizontal Force

	MOMENTS		**HORIZONTAL FORCE**	
	At Shaft Bottom kN-m	At Foundation kN-m	At Shaft Bottom Lvl kN	at Foundatio kN
Due to active Earth Pressure	3611.695	5271.108	1461.383	1863.376
Due to Minimum Fluid Pressure	2993.205	4557.563	1350.927	1787.981
Governing of Two	**3611.695**	**5271.108**	**1461.383**	**1863.376**
Due to Live Load Surcharge	**1887.135**	**2497.663**	**567.816**	**653.240**
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

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

α h	=	0.000	
α v	=	0.000	
ϕ	=	30.00	0.524
δ	=	20.00	0.349
α	=	0.00	0.000
β	=	0.00	0.000

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

$\lambda = \tan^{-1} \dfrac{\alpha\ h}{(1 \pm \alpha\ v)}$ = 0.000 / 0.000

		1	2
C_a	=	0.279	0.279

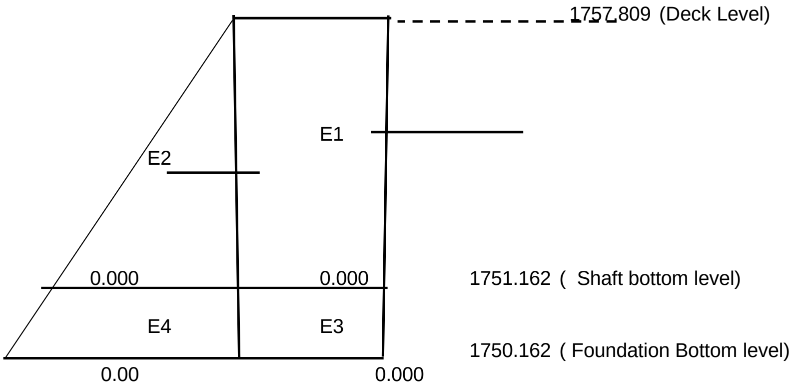
Ca	=	0.279		
Ka	=	**0.279**		
Dynamic Increment	=	0.279	-0.279	**0.000**

3 Earth Pressure : **DRY CASE (Seismic case)**

Equivalent Live Load Surcharge height = 1.2 m

Assuming γdry = 20 kN/m^3

γwater = 10.00 kN/m^3



Earth Pressure Diagram for Dynamic Increment

Horizontal Forces and Moments @ RL **1751.2 m (at Shaft Base)**
1750.2 m (at Foundation Bottom Level)

Due to Dynamic Live Load Surcharge

	=	0.000	x	20	x	1.2	=	0.000	kN/m^2
at Shaft Bottom Level									
E1	=	0.000	x	6.647	x	12.740	=	**0.000**	kN
M1	=	0.000	x	4.453			=	**0.000**	kN.m
at Foundation Bottom Level									
E3	=	0.000	x	7.647	x	12.740	=	**0.000**	kN
M3	=	0.000	x	5.123			=	**0.000**	kN.m

Due to Dynamic Active Earth Pressure

(At Shaft bottom level)									
	=	0.000	x	20	x	6.647	=	0.000	kN/m^2
(at Foundation Bottom Level)									
	=	0.000	x	20	x	7.647	=	0.000	kN/m^2
E2	=	0.50	x	0.00	x	6.65	x	12.740	
	=	**0.000**	kN						

E4	=	0.50	x	0.00	x	7.65	x	12.740	
	=	**0.000**	kN						
M2	=	0.00	x	3.32	=	**0.000**	kN.m	**(Shaft bottom level)**	
M4	=	0.00	x	3.82	=	**0.000**	kN.m	**(Foundation Bottom level)**	

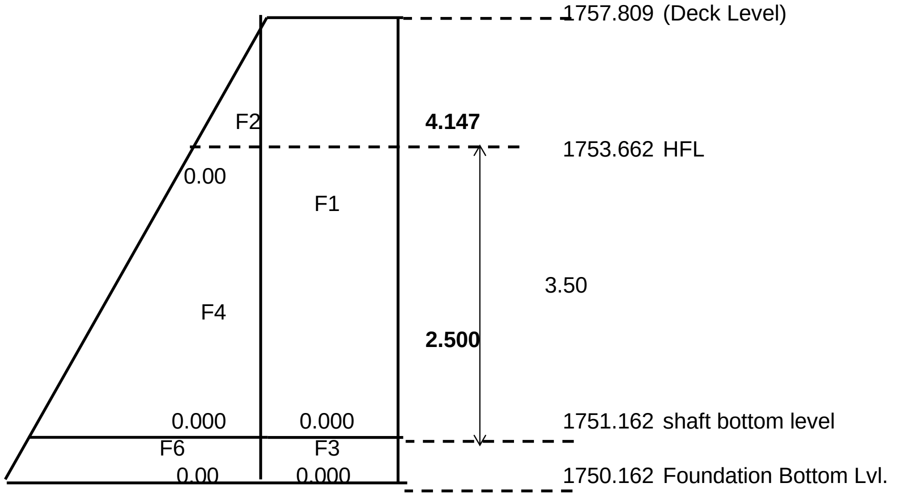
Summary of Moment and Horizontal Force **Dry Seismic Case**

	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)	4390.326	6684.870	1572.613	2081.386
Due to active Earth Pressure (dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	4390.326	6684.870	1572.613	2081.386
Due to Minimum Fluid Pressure	2993.205	4557.563	1350.927	1787.981
Governing of Two	**4390.326**	**6684.870**	**1572.613**	**2081.386**
Due to Live Load Surcharge (Static)	**1887.135**	**2497.663**	**567.816**	**653.240**
Due to Live Load Surcharge(Dynamic)	**0.000**	**0.000**	**0.000**	**0.000**
Due to Passive pressure		**0.000**		**0.000**

Earth Pressure : Normal HFL Case

Dynamic Increment	=	0.000
γd	=	20 kN/m^3
$\gamma water$	=	10 kN/m^3
Equivalent Live Load Surcharge height	=	1.2 m

Assuming



Earth Pressure Diagram

Horizontal Forces and Moments @ RL **1751.162 m (at Shaft Base)**
1750.162 m (at Foundation Bottom Level)

Due to Live Load Surcharge

Intensity for rectangular portion = 0.000 x 20 x 1.200 = 0.000 kN/m^2

at Shaft Bottom Level

F1 = 0.000 x 6.647 x 12.740 = **0.000 kN**

M1 = 0.00 x 4.39 = **0.000 kN.m**

at Foundation Bottom Level

F3 = 0.000 x 7.647 x 12.740 = **0.000 kN**

M3 = 0.00 x 5.05 = **0.000 kN.m**

Due to Dynamic Active Earth Pressure

Intensity for triangular portion

Upto HFL = 0.000 x 20 x 4.147 = 0.000 kN/m^2

(At Shaft bottom level) Below HFL = 0.000 x 10 x 2.500 = 0.000 kN/m^2

(At Foundati bottom level) Below HFL = 0.000 x 10 x 3.500 = 0.000 kN/m^2

F2 = 0.5 x 0.00 x 4.15 x 12.74
= **0.000 kN**

F4 = $\frac{(0.00 + 0.00)}{2}$ x 2.50 x 12.74
= **0.000 kN**

F6 = $\frac{(0.00 + 0.00)}{2}$ x 3.50 x 12.74
= **0.000 kN**

Total Force (F2 + F4)	**=**	**0.000 kN**	**at Shaft Bottom Level**
Total Force (F2 + F6)	**=**	**0.000 kN**	**at Foundation Bottom Level**

M2 = 0.00 x 4.57 = **0.000 kN.m**

M4 = 0.00 x 0.00 = **0.000 kN.m**

Total Mome = **0.000 kN.m** **at Shaft Bottom**

M2 = 0.00 x 5.57 = **0.000 kN.m**

M6 = 0.00 x 0.00 = **0.000 kN.m**

Total Mome = **0.000 kN.m** **at Foundation Bottom Level**

Summary of Moment and Horizontal Force

	MOMENTS		HORIZONTAL FORCE	
	At Shaft Bottom kN-m	At Foundation Bottom kN-m	At Shaft Bottom Lvl kN	At Foundation Bottom Lvl kN
Due to active Earth Pressure(Static	3611.695	5271.108	1461.383	1863.376
Due to active Earth Pressure (Dynamic Increment)	0.000	0.000	0.000	0.000
Total Earth Pressure	3611.695	5271.108	1461.383	1863.376
Due to Minimum Fluid Pressure	2993.205	4557.563	1350.927	1787.981
Governing of Two	**3611.695**	**5271.108**	**1461.383**	**1863.376**
Due to Live Load Surcharge(Stat	**1887.135**	**2497.663**	**567.816**	**653.240**
Due to Live Load Surcharge (Dynamic Increment)	**0.000**	**0.000**	**0.000**	**0.000**
Due to passive pressure		**0.000**		**0.000**

Horizontal Force AT Bearings (HL) IN ULTIMATE LIMIT STATE

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)	
DL	=	1250.00	1.35	1.35	1687.50	1687.50	
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00	
WC	=	137.50	1.75	1.75	240.63	240.63	
FPLL	=	0.00	1.5	0.20	0.00	0.00	
CWLLmax-Reaction case	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmin	=	0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A
CWLLmin	=	0.00	1	0.20	0.00	0.00	SV Loading
CWLLmax-Transv. Moment Case		0.00	1.5	0.20	0.00	0.00	1- 70RW + 2-CLASS A

Braking Force = 0.2 x 1000 + 0.05 x 554

= 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	2036.13	0	1018.063	1018.063		
DL+SIDL+LL-Max Reaction case	2036.13	341.55	1018.063	1018.063	1- 70RW + 2-CLASS A	Dry Case
	2036.13	0	1018.063	1018.063	SV Loading	
DL+SIDL+LL-Min Reaction case	2036.13	341.55	1018.063	1018.063	1- 70RW + 2-CLASS A	HFL Case
	2036.13	0	1018.063	1018.063	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	2036.13	341.55	1018.063	1018.063		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	0.00	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	45.54	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	45.54	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	45.54	1018.063	1018.063	

Transverse Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Factored Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	0.000	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	45.540	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	45.540	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	45.540	1018.063	1018.063	

Horizontal Force AT Bearings (HL) For Foundation Design

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1.35	1.35	1687.50	1687.50
SIDL except wc	=	80.00	1.35	1.35	108.00	108.00
WC	=	137.50	1.75	1.75	240.63	240.63
FPLL	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Reaction case	=	0.00	1.5	0.75	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1.5	0.75	0.00	0.00
CWLLmin	=	0.00	1.5	0.75	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	2036.13	0.000	1018.063	1018.063		
DL+SIDL+LL-Max Reaction case	2036.13	341.550	1018.063	1018.063	1- 70RW + 2-CLASS A	Dry Case
	2036.13	0.000	1018.063	1018.063	SV Loading	
DL+SIDL+LL-Min Reaction case	2036.13	341.550	1018.063	1018.063	1- 70RW + 2-CLASS A	HFL Case
	2036.13	0.000	1018.063	1018.063	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	2036.13	341.550	1018.063	1018.063		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	0.00	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	45.54	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	45.54	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	45.54	1018.063	1018.063	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	2036.13	0.000	1018.063	1018.063	
DL+SIDL+LL-Max Reaction case		2036.13	45.540	1018.063	1018.063	Dry Case
DL+SIDL+LL-Min Reaction case		2036.13	45.540	1018.063	1018.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		2036.13	45.540	1018.063	1018.063	

Horizontal Force AT Bearings (HL) For Base Pressure Calculation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1	1.00	1250.00	1250.00
SIDL except wc	=	80.00	1	1.00	80.00	80.00
WC	=	137.50	1	1.00	137.50	137.50
FPLL	=	0.00	1	1.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.20	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1	0.20	0.00	0.00
CWLLmin	=	0.00	1	0.20	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1467.50	0.000	733.750	733.750		
DL+SIDL+LL-Max Reaction case	1467.50	227.700	733.750	733.750	1- 70RW + 2-CLASS A	Dry Case
	1467.50	0.000	733.750	733.750	SV Loading	
DL+SIDL+LL-Min Reaction case	1467.50	227.700	733.750	733.750	1- 70RW + 2-CLASS A	HFL Case
	1467.50	0.000	733.750	733.750	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1467.50	227.700	733.750	733.750		

Longitudinal Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	0.000	733.750	733.750	
DL+SIDL+LL-Max Reaction case	1467.50	45.540	733.750	733.750	Dry Case
DL+SIDL+LL-Min Reaction case	1467.50	45.540	733.750	733.750	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1467.50	45.540	733.750	733.750	

Transverse Seismic Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	0.000	733.750	733.750	
DL+SIDL+LL-Max Reaction case	1467.50	45.540	733.750	733.750	Dry Case
DL+SIDL+LL-Min Reaction case	1467.50	45.540	733.750	733.750	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1467.50	45.540	733.750	733.750	

Horizontal Force AT Bearings (HL) For Stability of Foundation

(Refer Clause 211.5.1.1 of IRC:6-2014)

Type of bearing - Tar Paper Bearing

Loads		Unfactored Load	Basic Comb	Seismic Comb	Load (Basic Comb)	Load (Seismic Comb)
DL	=	1250.00	1.05	1.05	1312.50	1312.50
SIDL except wc	=	80.00	1.05	1.05	84.00	84.00
WC	=	137.50	1.35	1.35	185.63	185.63
FPLL	=	0.00	1.5	0.00	0.00	0.00

CWLLmax-Reaction case	=	0.00	1.5	0.00	0.00	0.00
CWLLmax-Transv. Moment Case		0.00	1.5	0.00	0.00	0.00
CWLLmin	=	0.00	1.5	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case:

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1582.13	0.000	791.063	791.063		
DL+SIDL+LL-Max Reaction case	1582.13	341.550	791.063	791.063	1- 70RW + 2-CLASS A	Dry Case
	1582.13	0.000	791.063	791.063	SV Loading	
DL+SIDL+LL-Min Reaction case	1582.13	341.550	791.063	791.063	1- 70RW + 2-CLASS A	HFL Case
	1582.13	0.000	791.063	791.063	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1582.13	341.550	791.063	791.063		

Longitudinal Seismic Case: Seismic effect = 1.50

	Unfactored Vertical Force	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	1582.13	0.00	791.063	791.063	
DL+SIDL+LL-Max Reaction case		1582.13	0.00	791.063	791.063	Dry Case
DL+SIDL+LL-Min Reaction case		1582.13	0.00	791.063	791.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1582.13	0.00	791.063	791.063	

Transverse Seismic Case:

	Unfactored Vertical Force	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1467.50	1582.13	0.000	791.063	791.063	
DL+SIDL+LL-Max Reaction case		1582.13	0.000	791.063	791.063	Dry Case
DL+SIDL+LL-Min Reaction case		1582.13	0.000	791.063	791.063	HFL Case
DL+SIDL+LL-Max Transv. Moment case		1582.13	0.000	791.063	791.063	

Horizontal Force At Bearings (HL) IN SLS CASE

Loads		Unfactored Load	Rare Comb	Frequent Comb	Quasi-Permanent Comb	Load (Rare Comb)	Load (Frequent Comb)	Load (Quasi-Permanent Comb)
DL	=	1250.00	1	1	1	1250.00	1250.00	1250.00
SIDL except wc	=	80.00	1	1	1	80.00	80.00	80.00
WC	=	137.50	1.20	1.20	1.20	165.00	165.00	165.00
FPLL	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Reaction case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmax-Transv. Moment Case	=	0.00	1	0.75	0	0.00	0.00	0.00
CWLLmin	=	0.00	1	0.75	0	0.00	0.00	0.00

Braking Force = 227.7 KN

Normal Case: Rare Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)		
DL+SIDL	1495.00	0.000	747.500	747.500		
DL+SIDL+LL-Max Reaction case	1495.00	227.700	747.500	747.500	1- 70RW + 2-CLASS A	Dry Case
	1495.00	0.000	747.500	747.500	SV Loading	
DL+SIDL+LL-Min Reaction case	1495.00	227.700	747.500	747.500	1- 70RW + 2-CLASS A	HFL Case
	1495.00	0.000	747.500	747.500	SV Loading	
DL+SIDL+LL-Max Transv. Moment case	1495.00	227.700	747.500	747.500		

Normal Case: Frequent Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1495.00	0.000	747.500	747.500	
DL+SIDL+LL-Max Reaction case	1495.00	170.775	747.500	747.500	Dry Case
DL+SIDL+LL-Min Reaction case	1495.00	170.775	747.500	747.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1495.00	170.775	747.500	747.500	

Normal Case: Quasi Permenant Combination

	Vertical Force (R)	Fh	μ R	Max (Fh/2 or μR)	
DL+SIDL	1495.00	0.000	747.500	747.500	
DL+SIDL+LL-Max Reaction case	1495.00	0.000	747.500	747.500	Dry Case
DL+SIDL+LL-Min Reaction case	1495.00	0.000	747.500	747.500	HFL Case
DL+SIDL+LL-Max Transv. Moment case	1495.00	0.000	747.500	747.500	

Centrifugal Force Calculation

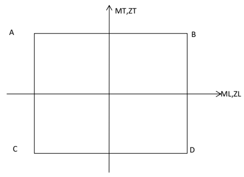
As per clause 212 of IRC:6-2014

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

			Normal Case		Seismic Case	
Design Speed	V	=	100.00	kmph	100.00	kmph
Live Load	W	=	932.33	kN	932.33	kN
Radius of Curvature	R	=	0.00	m	0.00	m
CENTRIFUGAL FORCE	C	=	0.00	kN	0.00	kN

SBC AND STABILITY CHECK OF FOUNDATION

Foundation Lvl = 1750.162 m



Properties of Footing Base:

A	=	89.180	m^2
ZL	=	104.043	m^3
ZT	=	189.359	m^3

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

Moment along longitudinal axis	**ML = ML Cos θ + MT Sin θ**
Moment along transverse axis	**MT = MT Cos θ - ML Sin θ**

Case 1 : DL+SIDL-Normal Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure								
Dead Load	1			1250.000	0.710	887.500	0.000	0.000
SIDL except Wearing Course	1			80.000	0.710	56.800	0.000	0.000
Wearing Course	1			137.500	0.710	97.625	0.000	0.000
				1467.500		**1041.925**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1	25	2.293	57.330	0.340	19.492	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.467	11.681	0.340	3.972	0.000	0.000
Bracket - Uniform portion	1	25	1.147	28.665	0.040	1.147	0.000	0.000
Bracket - Tapered portion	1	25	0.573	14.333	0.090	1.290	0.000	0.000
Cap - (uniform portion)	1	25	2.752	68.796	0.550	37.838	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.550	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1	25	2.400	60.008	-1.900	-114.029	0.000	0.000
Cantilever Return Wall-Traingle porti	1	25	4.135	103.363	-1.234	-127.498	0.000	0.000
RCC Railing or Crash Barrier	1			28.000	0.340	9.520	0.000	0.000
Aproach Slab	1	25	6.689	167.213	0.040	6.688	0.000	0.000
				539.387		**-161.581**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1	25	25.189	629.730	-1.700	-1070.541	0.000	0.000
Abutment Shaft	1	25	53.278	1331.943	0.570	759.472	0.000	0.000
Back filling over heel slab	1	20	314.317	6286.349	-1.731	-10879.422	0.000	0.000
Front Filling over toe slab	1	20	74.848	1496.950	2.188	3275.242	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	29.812	745.290	-1.377	-1026.207	0.000	0.000
Toe slab	1	25	20.703	517.563	2.026	1048.396	0.000	0.000
portion between heel & toe	1	25	11.466	286.650	0.550	157.658	0.000	0.000
Vertical Components of active earth pressure	1			757.562	-3.500	-2651.469	0.000	0.000
				12215.103		**-10362.743**		**0.000**
Total				**14221.990**		**-9482.399**		**0.000**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
0.950	1187.500	-2.790	-3313.125
0.950	76.000	-2.790	-212.040
1.000	137.500	-2.790	-383.625
	1401.000		**-3908.790**
0.950	54.464	-3.160	-172.105
0.950	11.097	-3.160	-35.066
0.950	27.232	-3.460	-94.222
0.950	13.616	-3.410	-46.430
0.950	65.356	-2.950	-192.801
0.950	0.000	-2.950	0.000
0.950	57.007	-5.400	-307.853
0.950	98.195	-4.734	-464.805
0.950	26.600	-3.160	-84.056
0.950	158.852	-3.460	-549.627
	512.418		**-1946.965**
0.950	598.244	-5.200	-3110.866
0.950	1265.346	-2.930	-3707.213
0.950	5972.032	-5.231	-31237.562
0.950	1422.103	-1.312	-1865.879
0.950	0.000	0.000	0.000
0.950	708.026	-4.877	-3452.986
0.950	491.684	-1.474	-724.919
0.950	272.318	-2.950	-803.337
0.950	719.684	-7.000	-5037.790
	11604.348		**-50456.631**
	13517.766		**-56312.386**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	2081.386		6684.870

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
2081.39	6684.87	0.00	0.00
2815.136	11661.162	0.000	0.000

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	3122.079		10027.305
		3913.141		15392.291

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
3122.08	10027.30	0.00	0.00
3913.141	15392.291	0.000	0.000

Summary of Forces For SBC		
P	14221.990	KN
ML	2178.763	kNm
MT	0.000	kNm

Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		14221.990		-9482.399		0.000
CWLL-Max. Reaction case	1	932.327	0.710	661.952	2.411	2248.220
Vertical Components of LL Surcharge	1	237.760	-3.500	-832.160	0.000	0.000
Total		**15392.077**		**-9652.607**		**2248.220**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	13517.766		-56312.39
0.000		-2.790	0.00
0.950		-7.000	-1581.10
	13517.766		**-57893.49**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	2081.386		6684.870
due to Live load surcharge	1	653.240		2497.663

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
2081.39	6684.87	0.00	0.00
653.240	2497.663	0.000	0.000
3468.38	**14158.83**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	3122.079		10027.305
due to Live load surcharge	1.2	783.888		2997.195
		4697.029		**18389.486**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
3122.08	10027.30	0.00	0.00
783.89	2997.20	0.00	0.00
4697.029	**18389.486**	**0.000**	**0.000**

Summary of Forces For SBC		
P	15392.077	KN
ML	4506.219	kNm
MT	2248.220	kNm

Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case :DL+SIDL		14221.990		-9482.399		0.000
CWLL-Max. Reaction case	1	2561.336	0.710	1818.549	0.300	768.401
Vertical Components of LL Surcharge	1	237.760	-3.500	-832.160	0.000	0.000
Total		**17021.086**		**-8496.010**		**768.401**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	13517.76558		-56312.38581
0.000		-2.790	0.00
0.950		-7.000	-1581.10
	13517.766		**-57893.49**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	2081.386		6684.870
due to Live load surcharge	1	653.240		2497.663

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
2081.39	6684.87	0.00	0.00
653.240	2497.663	0.000	0.000
3468.38	**14158.83**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	3122.079		10027.305
due to Live load surcharge	1.2	783.888		2997.195
		4697.029		**18389.486**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
3122.08	10027.30	0.00	0.00
783.89	2997.20	0.00	0.00
4697.029	**18389.486**	**0.000**	**0.000**

Summary of Forces For SBC		
P	17021.086	KN
ML	5662.815	kNm
MT	768.401	kNm

Case 3 : DL+SIDL-Normal HFL Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Superstructure				**1467.500**		**1041.925**		**0.000**
Substructure & Foundation -Portion 1				**539.387**		**-161.581**		**0.000**
Substructure & Foundation -Portion 2								

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	1401.000		**-3908.790**
	512.418		**-1946.965**

Solid Return wall	1	25	25.189	629.730	-1.700	-1070.541	0.000	0.000	0.950	598.244	-5.200	-3110.866
Shaft above HFL	1	25	29.203	730.078	0.615	449.002	0.000	0.000	0.950	693.574	-2.885	-2000.956
Shaft below HFL	1	15	24.075	361.119	0.570	205.910	0.000	0.000	0.950	343.063	-2.930	-1005.108
Back filling above HFL over heel slab	1	20	190.198	3803.960	-1.700	-6466.732	0.000	0.000	0.950	3613.762	-5.200	-18791.563
Back filling below HFL over heel slab	1	10	130.712	1307.124	-1.774	-2318.425	0.000	0.000	0.950	1241.768	-5.274	-6548.691
Front Filling over toe slab	1	10	74.848	748.475	2.188	1637.621	0.000	0.000	0.950	711.051	-1.312	-932.940
Side filling between heel and toe	1	10	0.000	0.000	0.000	0.000	0.000	0.000	0.950	0.000	0.000	0.000
Heel slab	1	15	29.812	447.174	-1.377	-615.724	0.000	0.000	0.950	424.815	-4.877	-2071.792
Toe slab	1	15	20.703	310.538	2.026	629.038	0.000	0.000	0.950	295.011	-1.474	-434.952
Portion between Heel & Toe	1	15	11.466	171.990	0.550	94.595	0.000	0.000	0.950	163.391	-2.950	-482.002
Vertical Components of active earth pressure	1			678.213	-3.500	-2373.747	0.000	0.000	0.950	644.303	-7.000	-4510.118
				9321.656		-9807.770		0.000		8855.573		-40311.887
Total				**11328.543**		**-8927.426**		**0.000**		**10768.991**		**-46167.642**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	1863.376		5271.108

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
1863.38	5271.11	0.00	0.00
2597.13	**10247.40**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	2795.064		7906.662
		3586.126		**13271.648**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
2795.06	7906.66	0.00	0.00
3586.13	**13271.65**	**0.00**	**0.00**

Summary of Forces For SBC		
P	11328.543	KN
ML	1319.975	kNm
MT	0.000	kNm

Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		11328.543		-8927.426		0.000
CWLL-Min. Reaction case	1	466.273	0.710	331.054	3.109	1449.594
Vertical Components of LL Surcharge	1	237.760	-3.500	-832.160	0.000	0.000
Total		**12032.576**		**-9428.531**		**1449.594**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	10768.991		-46167.642
0.000	0.000	-2.790	0
0.950	225.872	-7.000	-1581.103
	10994.863		**-47748.745**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	1863.376		5271.108
due to Live load surcharge	1	653.240		2497.663

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
1863.38	5271.11	0.00	0.00
653.240	2497.663	0.000	0.000
3250.37	**12745.06**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	2795.064		7906.662
due to live load surcharge	1.2	783.888		2997.195
		4370.014		**16268.844**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
2795.06	7906.66	0.00	0.00
783.888	2997.195	0.000	0.000
4370.01	**16268.84**	**0.00**	**0.00**

Summary of Forces For SBC		
P	12032.576	KN
ML	3316.532	kNm
MT	1449.594	kNm

Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL1) (m)	ML = PxeL1 (kNm)	Trans. Ecc (eT) (m)	MT = PxeT (kNm)
Forces from Case : DL+SIDL		11328.543		-8927.426		0.000
CWLL-Min. Reaction case	1	858.664	0.710	609.651	0.300	257.599
Vertical Components of LL Surcharge	1	237.760	-3.500	-832.160	0.000	0.000
Total		**12424.967**		**-9149.934**		**257.599**

Vertical Forces For Restoring or Resisting Effect

Load Factor	Vertical Load(P) kN.	Long. Ecc. (eL2) @ Toe (m)	ML@toe = PxeL2 (kNm)
	10768.991		-46167.642
0.000	0.000	-2.790	0.00
0.950	225.872	-7.000	-1581.103
	10994.863		**-47748.745**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure	1	733.750	1756.944	4976.292
due to Earth pressure	1	1863.376		5271.108
due to Live load surcharge	1	653.240		2497.663

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
733.75	4976.29	0.00	0.00
1863.38	5271.11	0.00	0.00
653.240	2497.663	0.000	0.000
3250.37	**12745.06**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Earth pressure	1.5	2795.064		7906.662
due to live load surcharge	1.2	783.888		2997.195
		4370.014		**16268.844**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
2795.06	7906.66	0.00	0.00
783.888	2997.195	0.000	0.000
4370.01	**16268.84**	**0.00**	**0.00**

Summary of Forces For SBC		
P	12424.967	KN
ML	3595.130	kNm
MT	257.599	kNm

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

Seismic Effect Factor = 1

αh= 0.000 In Longitudinal direction
αh= 0.000 In Transverse direction
αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 514.165 KN
Weight of back fill below Ground level = 1834.560 KN

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 = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure															
Dead Load	1			1250.000		0.000	0.000	0.710	887.500	0.000	1757.477		0.000	0.000	0.000
SIDL except Wearing Course	1			80.000		0.000	0.000	0.710	56.800	0.000	1758.258		0.000	0.000	0.000
Wearing Course	1			137.500		0.000	0.000	0.710	97.625	0.000	1757.809		0.000	0.000	0.000
				1467.500		**0.000**	**0.000**		**1041.925**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1	25	2.293	57.330	0.000	0.000	0.000	0.340	19.492	0.000	1757.509	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.467	11.681	0.000	0.000	0.000	0.340	3.972	0.000	1757.148	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1	25	1.147	28.665				0.040	1.147						
Bracket - Tapered portion	1	25	0.573	14.333				0.090	1.290						
Cap - (uniform portion)	1	25	2.752	68.796	0.000	0.000	0.000	0.550	37.838	0.000	1756.793	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1756.643	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1	25	2.400	60.008	0.000	0.000	0.000	-1.900	-114.029	0.000	1757.509	0.000	0.000	0.000	0.000
Cantilever Return Wall-Trainqle porti	1	25	4.135	103.363	0.000	0.000	0.000	-1.234	-127.498	0.000	1756.520	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1			28.000				0.340	9.520				0.000	0.000	
Aproach Slab	1	25	6.689	167.213				0.040	6.688				0.000	0.000	
				539.387	**0.000**	**0.000**	**0.000**		**-161.581**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1	25	25.189	629.730	0.000	0.000	0.000	-1.700	-1070.541	0.000	1754.382	0.000	0.000	0.000	0.000
Abutment Shaft	1	25	53.278	1331.943	0.000	0.000	0.000	0.570	759.472	0.000	1754.903	0.000	0.000	0.000	0.000
Back filling over heel slab	1	20	314.317	6286.349	0.000	0.000	0.000	-1.731	-10879.422	0.000	1754.382	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1	20	74.848	1496.950				2.188	3275.242				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	29.812	745.290				-1.377	-1026.207				0.000	0.000	
Toe slab	1	25	20.703	517.563				2.026	1048.396				0.000	0.000	
portion between heel & toe	1	25	11.466	286.650				0.550	157.658				0.000	0.000	
Vertical component of active earth pressure	1			757.562				-3.500	-2651.469				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				12215.103	**0.000**	**0.000**	**0.000**		**-10362.743**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**14221.990**	**0.000**	**0.000**	**0.000**		**-9482.399**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure								
Dead Load	0.95			1187.500	0.000	-2.790	-3313.125	0.000

For Overturning or Sliding Effect

Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL

SIDL except Wearing Course	0.95			76.000	0.000	-2.790	-212.040	0.000				
Wearing Course	1.00			137.500	0.000	-2.790	-383.625	0.000				
				1401.000	**0.000**		**-3908.790**	**0.000**				
Substructure & Foundation -Portion 1												
Dirt Wall-Uniform portion	0.95	25	2.293	54.464	0.000	-3.160	-172.105	0.000	1.0	0.000	1757.509	0.000
Dirt Wall-Tapered portion	0.95	25	0.467	11.097	0.000	-3.160	-35.066	0.000	1.0	0.000	1757.148	0.000
Bracket - Uniform portion	0.95	25	1.147	27.232								
Bracket - Tapered portion	0.95	25	0.573	13.616								
Cap - (uniform portion)	0.95	25	2.752	65.356	0.000	-2.950	-192.801	0.000	1.0	0.000	1756.793	0.000
Cap - (corbel portion)	0.95	25	0.000	0.000	0.000	-2.950	0.000	0.000	1.0	0.000	1756.643	0.000
Cantilever Return Wall-Rectangle po	0.95	25	2.400	57.007	0.000	-5.400	-307.853	0.000	1.0	0.000	1757.509	0.000
Cantilever Return Wall-Traingle porti	0.95	25	4.135	98.195	0.000	-4.734	-464.805	0.000	1.0	0.000	1756.520	0.000
RCC Railing or Crash Barrier	0.95			26.600		-3.160	-84.056					
Aproach Slab	0.95	25	6.689	158.852		-3.460	-549.627					
				512.418	**0.000**		**-1806.313**	**0.000**		**0.000**		**0.000**
Substructure & Foundation -Portion 2												
Abutment Shaft	0.95	25	53.278	1265.346	0.000	-2.930	-3707.213	0.000	1.0	0.000	1754.903	0.000
Solid Return wall	0.95	25	25.189	598.244	0.000	-5.200	-3110.866	0.000	1.0	0.000	1754.382	0.000
Back filling over heel slab	0.95	20	314.317	5972.032	0.000	-5.231	-31237.562	0.000	1.0	0.000	1754.382	0.000
Front Filling over Pile Cap	0.95	20	74.848	1422.103		-1.312	-1865.879					
Side filling between heel and toe	0.95	20	0.000	0.000		0.000	0.000					
Heel slab	0.95	25	29.812	708.026		-4.877	-3452.986					
Toe slab	0.95	25	20.703	491.684		-1.474	-724.919					
portion between heel & toe	0.95	25	11.466	272.318		-2.950	-803.337					
Vertical component of active earth pressure	0.95			719.684		-7.000	-5037.790					
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000					
				11604.348	**0.000**		**-50456.63**	**0.000**		**0.000**		**0.000**
Total =				**13517.766**	**0.000**		**-56171.73**	**0.000**		**0.000**		**0.000**
					0.000			0.000				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2081.386			6684.870	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4976.29	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	0.00
2081.39	6684.87						
2815.14	**11661.16**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.99
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	3122.079		10027.30
due to dynamic Earth pressure	1.5	0.000		0.00
		3913.141		**15392.291**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
3122.079	10027.305	0.000	0.000
0.000	0.000	0.000	0.000
3913.14	**15392.29**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	14221.990	14221.990	KN
ML	2178.763	2178.763	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	13517.766	kN
Moment	-56171.734	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 = 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 Superstucture				1467.500		0.000	0.000		1041.925	0.000				0.000	0.000
Forces from Substructure				12754.490	0.000	0.000	0.000		-10524.324	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			186.47		0.000	0.000	0.710	132.390	0.000	1759.009		2.411	449.644	0.000
Vertical component of LL Surcharge	0.20			47.55				-3.500	-166.432				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**14456.007**	**0.000**	**0.000**	**0.000**		**-9516.441**	**0.000**		**0.000**		**449.644**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstucture				1401.000	0.000	0.000	-3908.79	0.000
Forces from Substructure				12116.766	0.000	0.000	-52262.94	0.000
CWLL-Max. Reaction case	0.00			0.00	0.000	-2.790	0.00	0.00
Vertical component of LL Surcharge	0.00			0.00		-7.000	0.00	
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.00	
Total =				**13517.766**	**0.000**		**-56171.73**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2081.386			6684.870	
due to Live load surcharge	0.20	130.648			499.533	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4976.29	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	0.00
2081.39	6684.87						
130.65	499.53						
2945.78	**12160.69**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Substructure	1.5	0.000		0.000
due to Active Earth pressure	1.5	3122.079		10027.305
due to dynamic Earth pressure	1.5	0.000		0
due to Live load surcharge	0	0		0
due to dynamic increment of live load surcharge	0	0		0
		3913.141		**15392.291**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
3122.079	10027.305		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3913.14	**15392.29**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	14456.007	14456.007	KN
ML	2644.254	2644.254	kNm
MT	449.644	449.644	kNm

Summary of Restoring Forces

Vertical Load	13517.766	kN
Moment	-56171.734	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 αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1467.500**		**0.000**	**0.000**		**1041.925**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**539.387**	**0.000**	**0.000**	**0.000**		**-161.581**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1		25.19	629.73	0.000	0.000	0.000	-1.70	-1070.541	0.000	1754.38	0.000	0.00	0.000	0.000
Shaft above HFL	1	25	29.203	730.078	0.000	0.000	0.000	0.615	449.002	0.000	1755.153	0.000	0.000	0.000	0.000
Shaft below HFL	1	15	24.075	361.119	0.000	0.000	0.000	0.570	205.910	0.000	1753.412	0.000	0.000	0.000	0.000
Back filling above HFL over heel slab	1	20	190.198	3803.960	0.000	0.000	0.000	-1.700	-6466.732	0.000	1755.736	0.00	0.000	0.000	0.00
Back filling below HFL over heel slab	1	10	130.712	1307.124	0.000	0.000	0.000	-1.774	-2318.425	0.000	1752.162	0.00	0.000	0.000	0.00
Front Filling over Pile Cap	1	10	74.848	748.475				2.188	1637.621				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	29.812	447.174				-1.377	-615.724				0.000	0.000	
Toe slab	1	15	20.703	310.538				2.026	629.038				0.000	0.000	
portion between heel & toe	1	15	11.466	171.990				0.550	94.595				0.000	0.000	
Vertical component of active earth pressure	1			678.213				-3.500	-2373.747				0.000	0.000	
Vertical component of dynamic increment of earth pressure	1			0.000				-3.500	0.000				0.000	0.000	
				9321.656	**0.000**	**0.000**	**0.000**		**-9829.004**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**11328.543**	**0.000**	**0.000**	**0.000**		**-8948.660**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect / **For Overturning or Sliding Effect**

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv	Load Factor	FL = αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure				**1401.000**	**0.000**		**-3908.790**	**0.000**				
Substructure & Foundation -Portion 1				**512.418**	**0.000**		**-1806.313**	**0.000**		**0.000**		**0.000**
Substructure & Foundation -Portion 2												
Solid Return wall	0.95	25	25.1892	598.244	0.000	-5.20	-3110.866	0.000	1.0	0.000	1754.382	0.000
Shaft above HFL	0.95	25	29.203	693.574	0.000	-2.885	-2000.956	0.000	1.0	0.000	1755.153	0.000
Shaft below HFL	0.95	15	24.075	343.063	0.000	-2.930	-1005.108	0.000	1.0	0.000	1753.412	0.000
Back filling above HFL over heel slab	0.95	20	190.198	3613.762	0.000	-5.200	-18791.563	0.000	1.0	0.000	1755.736	0.000
Back filling below HFL over heel slab	0.95	10	130.712	1241.768	0.000	-5.274	-6548.691	0.000	1.0	0.000	1752.162	0.000
Front Filling over Pile Cap	0.95	10	74.848	711.051		-1.312	-932.940					
Side filling between heel and toe	0.95	10	0.000	0.000		0.000	0.000					
Heel slab	0.95	15	29.812	424.815		-4.877	-2071.792					
Toe slab	0.95	15	20.703	295.011		-1.474	-434.952					
portion between heel & toe	0.95	15	11.466	163.391		-2.950	-482.002					
Vertical component of active earth pressure	0.95			644.303		-7.000	-4510.118					
Vertical component of dynamic increment of earth pressure	0.95			0.000		-7.000	0.000					
				8855.573	**0.000**		**-40311.887**	**0.000**		**0.000**		**0.000**
Total =				**10768.991**	**0.000**		**-46026.99**	**0.000**		**0.000**		**0.000**
					0.000			0.000				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	Forces along Trans. Axis FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000	733.75	4976.29	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure	1	0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure	1	1863.376			5271.108		1863.38	5271.11						
							2597.13	**10247.40**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		791.063	1756.944	5364.986	791.06	5364.99	0.00	0.00
due to Substructure	1.5	0.000		0.000	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.5	2795.064		7906.662	2795.064	7906.662		
due to dynamic Earth pressure	1.5	0.000		0.000	0.000	0.000		
		3586.126		**13271.648**	**3586.13**	**13271.65**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	11328.543	11328.543	KN
ML	1298.740	1298.740	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	10768.991	kN
Moment	-46026.990	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				1467.500		0.000	0.000		1041.925	0.000				0.000	0.000
Forces from Substructure				9861.043	0.000	0.000	0.000		-9990.585	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			93.25		0.000	0.000	0.710	66.211	0.000	1759.009		3.109	289.919	0.000
Vertical component of LL Surcharge	0.20			47.55				-3.500	-166.432				0.000	0.000	
Vertical component of dynamic increment LL Surcharge	0.20			0.00				-3.500	0.000				0.000	0.000	
Total =				**11469.350**	**0.000**	**0.000**	**0.000**		**-9048.881**	**0.000**		**0.000**		**289.919**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Forces from Superstructure				1401.000	0.000		-3908.790	0.000
Forces from Substructure				9367.991	0.000		-42118.200	0.000
CWLL-Max. Reaction case	0.00			0.00	0.00		-2.79	0.00
Vertical component of LL Surcharge	0.00			0.00		-2.790	0.00	0.00
Vertical component of dynamic increment LL Surcharge	0.00			0.000		-7.000	0.000	0.000
Total =				**10768.991**	**0.000**		**-46029.78**	**0.000**
					0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	Forces along Trans. Axis FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000	733.75	4976.29	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure	1	0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure	1	1863.376			5271.108		1863.38	5271.11						
due to Live load surcharge	0.20	130.648			499.533		130.65	499.53						
							2727.77	**10746.93**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	Forces along Trans. Axis FL Sinθ	ML Sin θ
due to Superstructure		791.063	1756.944	5364.986	791.06	5364.99	0.00	0.00
due to Substructure	1.5	0.000		0	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.5	2795.064		7906.662	2795.064	7906.662		
due to dynamic Earth pressure	1.5	0		0	0.000	0.000		
due to Live load surcharge	0	0		0	0.000	0.000		
due to dynamic increment of live load surcharge	0	0		0	0.000	0.000		
		3586.126		**13271.648**	**3586.13**	**13271.65**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	11469.350	11469.350	KN
ML	1698.052	1698.052	kNm
MT	289.919	289.919	kNm

Summary of Restoring Forces

Vertical Load	10768.991	kN
Moment	-46029.780	kNm

Case 9 : DL+SIDL-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	FL = 0.3 x αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**1467.500**		**0.000**	**0.000**		**1041.925**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**539.387**	**0.000**	**0.000**	**0.000**		**-161.581**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2				**12215.103**	**0.000**	**0.000**	**0.000**		**-10362.743**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**14221.990**	**0.000**	**0.000**	**0.000**		**-9482.399**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Vertical Forces For Restoring or Resisting Effect / **For Overturning or Sliding Effect**

Loads	Load Factor	Unit Weights (kN/m^3)	Volume (m^3)	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv	Load Factor	FL = 0.3 x αh x P (kN)	C.g. of Force (m)	MLs due to FL
Superstructure				**1401.000**	**0.000**		**-3908.79**	**0.000**				
Substructure & Foundation -Portion 1				**512.418**	**0.000**		**-1806.31**	**0.000**		**0.000**		**0.000**
Substructure & Foundation -Portion 2				**11604.348**	**0.000**		**-50456.63**	**0.000**		**0.000**		**0.000**
Total =				**13517.766**	**0.000**		**-56171.73**	**0.000**		**0.000**		**0.000**
					0.000			0.000				

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	Forces along Long. Axis FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	Forces along Trans. Axis FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000	733.75	4976.29	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure	1	0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure	1	2081.386			6684.870		2081.39	6684.87						
							2815.14	**11661.16**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

Forces along Long. Axis	Forces along Trans. Axis

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.986
due to Substructure	1.5	0.000		0
due to Active Earth pressure	1.5	3122.079		10027.30478
due to dynamic Earth pressure	1.5	0.000		0
		3913.141		**15392.291**

FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
3122.079	10027.305		
0.000	0.000		
3913.14	**15392.29**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	14221.990	14221.990	KN
ML	2178.763	2178.763	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	13517.766	kN
Moment	-56171.734	kNm

Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**14456.007**	**0.000**	**-9516.44**	**0.000**	**449.644**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL2	MLs due to Fv
Total =	**13517.766**	**0.000**	**-56171.73**	**0.000**
		0.000		**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	0.000	1756.944	4976.292	0.000
due to Substructure	1	0.000	0.000		0.000	0.000
due to Earth pressure	1	2081.386			6684.870	
due to Live load surcharge	0.2	130.648			499.533	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4976.29	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	0.00
2081.39	6684.87						
130.65	499.53						
2945.78	**12160.69**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.06	1756.944	5364.986
due to Substructure	1.5	0.00		0.000
due to Active Earth pressure	1.5	3122.08		10027.305
due to dynamic Earth pressure	1.5	0.00		0.000
due to Live load surcharge	0	0.00		0.000
due to dynamic increment of live load surcharge	0	0.00		0.000
		3913.14		**15392.291**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
3122.079	10027.305		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3913.14	**15392.29**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	14456.007	14456.007	KN
ML	2644.254	2644.254	kNm
MT	449.644	449.644	kNm

Summary of Restoring Forces

Vertical Load	13517.766	kN
Moment	-56171.734	kNm

Case 11 : DL+SIDL-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Superstructure		**1467.500**	**0.000**	**1041.925**	**0.000**	**0.000**
Substructure & Foundation -Portion 1		**539.387**	**0.000**	**-161.581**	**0.000**	**0.000**
Substructure & Foundation -Portion 2		**9321.656**	**0.000**	**-9829.004**	**0.000**	**0.000**
Total =		**11328.543**	**0.000**	**-8948.660**	**0.000**	**0.000**
			0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads		Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Superstructure		**1401.000**	**0.000**		**-3908.790**	**0.000**
Substructure & Foundation -Po		**512.418**	**0.000**		**-1806.313**	**0.000**
Substructure & Foundation -Po		**8855.573**	**0.000**		**-40311.887**	**0.000**
Total =		**10768.991**	**0.000**		**-46026.990**	**0.000**
			0.000			0.000

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.75	0.00	1756.94	4976.29	0.00
due to Substructure	1	0.00	0.00		0.00	0.00
due to Earth pressure	1	1863.38			5271.11	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4976.29	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	0.00
1863.38	5271.11						
2597.13	**10247.40**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.063	1756.944	5364.99
due to Substructure	1.5	0.000		0.00
due to Active Earth pressure	1.5	2795.064		7906.66
due to dynamic Earth pressure	1.5	0.000		0.00
		3586.126		**13271.65**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
2795.064	7906.662		
0.000	0.000		
3586.13	**13271.65**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	11328.543	11328.543	KN
ML	1298.740	1298.740	kNm
MT	0.000	0.000	kNm

Summary of Restoring Forces

Vertical Load	10768.991	kN
Moment	-46026.990	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Vertical Forces For SBC Calculation

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**11469.350**	**0.000**	**-9048.88**	**0.000**	**289.919**
		0.000		0.000	

Vertical Forces For Restoring or Resisting Effect

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL2) @ Toe (m)	ML = PxeL2	MLs due to Fv
Total =	**10768.991**	**0.000**		**-46029.78**	**0.000**
		0.000			**0.000**

Horizontal Forces For SBC Calculation

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure	1	733.750	0	1756.944	4976.292	0
due to Substructure	1	0.000	0		0.000	0
due to Earth pressure	1	1863.376			5271.108	
due to Live load surcharge	0.2	130.648			499.533	

Forces along Long. Axis				Forces along Trans. Axis			
FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
733.75	4976.29	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	0.00
1863.38	5271.11						
130.65	499.53						
2727.77	**10746.93**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Horizontal Forces For Overturning or Sliding Effect

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		791.06	1756.944	5364.99
due to Substructure	1.5	0.00		0.00
due to Active Earth pressure	1.5	2795.06		7906.66
due to dynamic Earth pressure	1.5	0.00		0.00
due to Live load surcharge	0	0.00		0.00
due to dynamic increment of live load surcharge	0	0.00		0.00
		3586.13		**13271.65**

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
791.06	5364.99	0.00	0.00
0.00	0.00	0.00	0.00
2795.064	7906.662		
0.000	0.000		
0.000	0.000		
0.000	0.000		
3586.13	**13271.65**	**0.00**	**0.00**

Summary of Forces For SBC

	Downward	Upward	
P	11469.350	11469.350	KN
ML	1698.052	1698.052	kNm
MT	289.919	289.919	kNm

Summary of Restoring Forces

Vertical Load	10768.991	kN
Moment	-46029.780	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 (1759.009 - 1750.162) = 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 (1759.009 - 1750.162) = 0.000 kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

SAFE BEARING CAPACITY CHECK	SLIDING CHECK	OVERTURNING CHECK

LOAD CASES	P	ML	MT	σ_A	σ_B	σ_C	σ_D	Max. Base Pressure	Min. Base Pressure	Sliding Force	Restoring Force= μP + c.A + Fp	FOS	Overturning moment	Restoring Moment = ΣP.eToe+Mp	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	14221.990	2178.763	0.000	138.534	180.416	138.534	180.416	180.416	138.534	3913.141	9462.436	2.42	15392.291	56312.39	3.66
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	15392.077	4506.219	2248.220	141.157	227.779	117.412	204.034	227.779	117.412	4697.029	9462.436	2.01	18389.486	57893.49	3.15
Case 2A : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case-SV Load Case	17021.086	5662.815	768.401	140.493	249.348	132.377	241.232	249.348	132.377	4697.029	9462.436	2.01	18389.486	57893.49	3.15
								SAFE	**SAFE**			**SAFE**			**SAFE**
Normal HFLCase															
Case 3 : DL+SIDL-Normal HFL Case	11328.543	1319.975	0.000	114.343	139.717	114.343	139.717	139.717	114.343	3586.126	7538.294	2.10	13271.648	46167.64	3.48
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	12032.576	3316.532	1449.594	110.703	174.456	95.393	159.146	174.456	95.393	4370.014	7696.404	1.76	16268.844	47748.75	2.93
Case 4A : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case-SV Load Case	12424.967	3595.130	257.599	106.131	175.239	103.410	172.518	175.239	103.410	4370.014	7696.404	1.76	16268.844	47748.75	2.93
								SAFE	**SAFE**			**SAFE**			**SAFE**
Longitudinal Seismic Dry Case															
Case 5 : DL+SIDL-Long. Seismic Dry Case	14221.990	2178.763	0.000	138.534	180.416	138.534	180.416	180.416	138.534	3913.141	9462.436	2.42	15392.291	56171.73	3.65
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	14456.007	2644.254	449.644	139.059	189.889	134.310	185.140	189.889	134.310	3913.141	9462.436	2.42	15392.291	56171.73	3.65
								SAFE	**SAFE**			**SAFE**			**SAFE**
Longitudinal Seismic HFL Case															
Case 7 : DL+SIDL-Long. Seismic HFL Case	11328.543	1298.740	0.000	114.547	139.513	114.547	139.513	139.513	114.547	3586.126	7538.294	2.10	13271.648	46026.99	3.47
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	11469.350	1698.052	289.919	113.819	146.461	110.757	143.399	146.461	110.757	3586.126	7538.294	2.10	13271.648	46029.78	3.47
								SAFE	**SAFE**			**SAFE**			**SAFE**
Transverse Seismic Dry Case															
Case 9 : DL+SIDL-Trans. Seismic Dry Case	14221.990	2178.763	0.000	138.534	180.416	138.534	180.416	180.416	138.534	3913.141	9462.436	2.42	15392.291	56171.73	3.65
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	14456.007	2644.254	449.644	139.059	189.889	134.310	185.140	189.889	134.310	3913.141	9462.436	2.42	15392.291	56171.73	3.65
								SAFE	**SAFE**			**SAFE**			**SAFE**
Transverse Seismic HFL Case															
Case 11 : DL+SIDL-Trans. Seismic HFL Case	11328.543	1298.740	0.000	114.547	139.513	114.547	139.513	139.513	114.547	3586.126	7538.294	2.10	13271.648	46026.99	3.47
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	11469.350	1698.052	289.919	113.819	146.461	110.757	143.399	146.461	110.757	3586.126	7538.294	2.10	13271.648	46029.78	3.47
								SAFE	**SAFE**			**SAFE**			**SAFE**

DESIGN OF FOUNDATION

Foundation Lvl = 1750.162 m

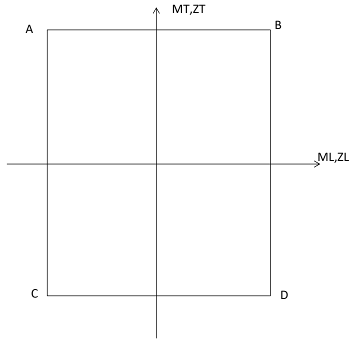
Properties of Footing Base:

A	=	89.180	m^2
ZL	=	104.043	m^3
ZT	=	189.359	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			1687.500	0.710	1198.125	0.000	0.000
SIDL except Wearing Course	1.35			108.000	0.710	76.680	0.000	0.000
Wearing Course	1.75			240.625	0.710	170.844	0.000	0.000
				2036.125		**1445.649**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.293	77.396	0.340	26.314	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.467	15.769	0.340	5.362	0.000	0.000
Bracket - Uniform portion	1.35	25	1.147	38.698	0.040	1.548	0.000	0.000
Bracket - Tapered portion	1.35	25	0.573	19.349	0.090	1.741	0.000	0.000
Cap - (uniform portion)	1.35	25	2.752	92.875	0.550	51.081	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.550	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1.35	25	2.400	81.010	-1.900	-153.939	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	4.135	139.540	-1.234	-172.123	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.35			37.800	0.340	12.852	0.000	0.000
Aproach Slab	1.35	25	6.689	225.737	0.040	9.029	0.000	0.000
				728.173		**-218.134**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	25.189	850.136	-1.700	-1445.230	0.000	0.000
Abutment Shaft	1.35	25	53.278	1798.123	0.570	1025.287	0.000	0.000
Back filling over heel slab	1.35	20	314.317	8486.571	-1.731	-14687.220	0.000	0.000
Front Filling over toe slab	1.35	20	74.848	2020.883	2.188	4421.576	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	29.812	1006.142	-1.377	-1385.379	0.000	0.000
Toe slab	1.35	25	20.703	698.709	2.026	1415.334	0.000	0.000
portion between heel & toe	1.35	25	11.466	386.978	0.550	212.838	0.000	0.000
Vertical Components of active earth pressure	1.5			1136.344	-3.500	-3977.203	0.000	0.000
				16604.023		**-14387.424**		**0.000**
Total				**19368.321**		**-13159.909**		**0.000**



Summary of Forces About C.G. OF Footing

P	19368.321	KN
ML	3771.896	kNm
MT	0.000	kNm

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	Forces along Long. Axis ML Cosθ	Forces along Trans. Axis FL Sinθ	Forces along Trans. Axis ML Sin θ
due to Superstructure		1018.063	1756.944	6904.500	1018.06	6904.50	0.00	0.00
due to Earth pressure	1.5	3122.079		10027.305	3122.08	10027.30	0.00	0.00
					4140.141	**16931.805**	**0.000**	**0.000**

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		19368.321		-13159.909		0.000
CWLL-Max. Reaction case	1.5	1398.490	0.710	992.928	2.411	3372.330
Vertical Components of LL Surcharge	1.2	285.312	-3.500	-998.592	0.000	0.000
Total		**21052.123**		**-13165.573**		**3372.330**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	Forces along Long. Axis ML Cosθ	Forces along Trans. Axis FL Sinθ	Forces along Trans. Axis ML Sin θ
due to Superstructure		1018.063	1756.944	6904.500	1018.06	6904.50	0.00	0.00
due to Earth pressure	1.5	3122.079		10027.305	3122.08	10027.30		
due to Live load surcharge	1.2	783.888		2997.195	783.89	2997.20		
		4924.029		**19929.000**	**4924.029**	**19929.000**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	21052.123	KN
ML	6763.427	kNm
MT	3372.330	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				**2036.125**		**1445.649**		**0.000**
Substructure & Foundation -Portion 1				**728.173**		**-218.134**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.35	25	25.189	850.136	-1.700	-1445.230	0.000	0.000
Shaft above HFL	1.35	25	29.203	985.605	0.615	606.153	0.000	0.000
Shaft below HFL	1.35	15	24.075	487.511	0.570	277.978	0.000	0.000
Back filling above HFL over heel slab	1.35	20	190.198	5135.346	-1.700	-8730.089	0.000	0.000
Back filling below HFL over heel slab	1.35	10	130.712	1764.617	-1.774	-3129.874	0.000	0.000
Front Filling over toe slab	1.35	10	74.848	1010.441	2.188	2210.788	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	29.812	603.685	-1.377	-831.228	0.000	0.000
Toe slab	1.35	15	20.703	419.226	2.026	849.201	0.000	0.000
Portion between Heel & Toe	1.35	15	11.466	232.187	0.550	127.703	0.000	0.000
Vertical Components of active earth pressure	1.5			1017.320	-3.500	-3560.620	0.000	0.000
				12685.967		**-13596.551**		**0.000**
Total				**15450.265**		**-12369.036**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)	Forces along Long. Axis FL Cosθ	Forces along Long. Axis ML Cosθ	Forces along Trans. Axis FL Sinθ	Forces along Trans. Axis ML Sin θ
due to Superstructure		1018.063	1756.944	6904.500	1018.06	6904.50	0.00	0.00
due to Earth pressure	1.5	2795.064		7906.662	2795.06	7906.66		
					3813.126	**14811.162**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	15450.265	KN
ML	2442.126	kNm
MT	0.000	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		15450.265		-12369.036		0.000
CWLL-Min. Reaction case	1.5	699.410	0.710	496.581	3.109	2174.391
Vertical Components of LL Surcharge	1.2	285.312	-3.500	-998.592	0.000	0.000
Total		**16434.987**		**-12871.047**		**2174.391**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1018.063	1756.944	6904.500
due to Earth pressure	1.5	2795.064		7906.662
due to Live load surcharge	1.2	783.888		2997.195

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1018.06	6904.50	0.00	0.00
2795.06	7906.66		
783.89	2997.20		
4597.014	**17808.358**	**0.000**	**0.000**

Summary of Forces About C.G. OF Footing

P	16434.987	KN
ML	4937.311	kNm
MT	2174.391	kNm

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

Seismic Effect Factor = 1.50

αh= 0.000 In Longitudinal direction
αh= 0.000 In Transverse direction
αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 514.16 KN
Weight of back fill below Ground level = 1834.56 KN

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 = 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			1687.500		0.000	0.000	0.710	1198.125	0.000	1757.477		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.000		0.000	0.000	0.710	76.680	0.000	1758.258		0.000	0.000	0.000
Wearing Course	1.75			240.625		0.000	0.000	0.710	170.844	0.000	1757.809		0.000	0.000	0.000
				2036.125		**0.000**	**0.000**		**1445.649**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.293	77.396	0.000	0.000	0.000	0.340	26.314	0.000	1757.509	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.467	15.769	0.000	0.000	0.000	0.340	5.362	0.000	1757.148	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.147	38.698				0.040	1.548						
Bracket - Tapered portion	1.35	25	0.573	19.349				0.090	1.741						
Cap - (uniform portion)	1.35	25	2.752	92.875	0.000	0.000	0.000	0.550	51.081	0.000	1756.793	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.550	0.000	0.000	1756.643	0.000	0.000	0.000	0.000
Cantilever Return Wall-Rectangle po	1.35	25	2.400	81.010	0.000	0.000	0.000	-1.900	-153.939	0.000	1757.509	0.000	0.000	0.000	0.000
Cantilever Return Wall-Traingle port	1.35	25	4.135	139.540	0.000	0.000	0.000	-1.234	-172.123	0.000	1756.520	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35			37.800				0.340	12.852				0.000	0.000	
Aproach Slab	1.35	25	6.689	225.737				0.040	9.029				0.000	0.000	
				728.173	**0.000**	**0.000**	**0.000**		**-218.134**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	25.189	850.136	0.000	0.000	0.000	-1.700	-1445.230	0.000	1754.382	0.000	0.000	0.000	0.000
Abutment Shaft	1.35	25	53.278	1798.123	0.000	0.000	0.000	0.570	1025.287	0.000	1754.903	0.000	0.000	0.000	0.000
Back filling over heel slab	1.35	20	314.317	8486.571	0.000	0.000	0.000	-1.731	-14687.220	0.000	1754.382	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	20	74.848	2020.883				2.188	4421.576				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	29.812	1006.142				-1.377	-1385.379				0.000	0.000	
Toe slab	1.35	25	20.703	698.709				2.026	1415.334				0.000	0.000	
portion between heel & toe	1.35	25	11.466	386.978				0.550	212.838				0.000	0.000	
Vertical component of active earth pressure	1.00			757.562				-3.500	-2651.469						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				16225.242	**0.000**	**0.000**	**0.000**		**-13061.689**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**18989.540**	**0.000**	**0.000**	**0.000**		**-11834.175**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	2081.386			6684.870	
due to dynamic increment of EP	1.50	0.000			0.000	

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
1018.06	6904.50	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	0.00
2081.39	6684.87						
0.00	0.00						
3099.45	**13589.37**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	18989.540	18989.540	KN
ML	1755.195	1755.195	kNm
MT	0.000	0.000	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstucture				2036.125		0.000	0.000		1445.649	0.000				0.000	0.000
Forces from Substructure				16953.415	0.000	0.000	0.000		-13279.823	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.75			699.25		0.000	0.000	0.710	496.464	0.000	1759.009		2.411	1686.165	0.000
Vertical component of LL Surcharge	0.20			47.552				-3.500	-166.432						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				**19736.337**	**0.000**	**0.000**	**0.000**		**-11504.143**	**0.000**		**0.000**		**1686.165**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1	2081.386			6684.870	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	130.648			499.533	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
1018.06	6904.50	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	0.00
2081.39	6684.87						
0.00	0.00						
130.65	499.53						
0.00	0.00						
3230.10	**14088.90**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	19736.337	19736.337	KN
ML	2584.760	2584.760	kNm
MT	1686.165	1686.165	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**2036.125**		**0.000**	**0.000**		**1445.649**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**728.173**	**0.000**	**0.000**	**0.000**		**-218.134**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2															
Solid Return wall	1.35	25	25.189	850.136	0.000	0.000	0.000	-1.700	-1445.230	0.000	1754.382	0.000	0.000	0.000	0.000
Shaft above HFL	1.35	25	29.203	985.605	0.000	0.000	0.000	0.615	606.153	0.000	1755.153	0.000	0.000	0.000	0.000
Shaft below HFL	1.35	15	24.0746253	487.511	0.000	0.000	0.000	0.570	277.978	0.000	1753.412	0.000	0.000	0.000	0.000
Back filling above HFL over heel slab	1.35	20	190.198008	5135.346	0.000	0.000	0.000	-1.700	-8730.089	0.000	1755.736	0.000	0.000	0.000	0.000
Back filling below HFL over heel slab	1.35	10	130.7124	1764.617	0.000	0.000	0.000	-1.774	-3129.874	0.000	1752.162	0.000	0.000	0.000	0.000
Front Filling over Pile Cap	1.35	10	74.848	1010.441				2.188	2210.788				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	29.812	603.685				-1.377	-831.228				0.000	0.000	
Toe slab	1.35	15	20.703	419.226				2.026	849.201				0.000	0.000	
portion between heel & toe	1.35	15	11.466	232.187				0.550	127.703				0.000	0.000	
Vertical component of active earth pressure	1.00			678.213				-3.500	-2373.747						
Vertical component of dynamic increment of earth pressure	1.50			0.000				-3.500	0.000						
				12346.861	**0.000**	**0.000**	**0.000**		**-12409.678**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**15111.159**	**0.000**	**0.000**	**0.000**		**-11182.163**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1863.376			5271.108	
due to dynamic increment of EP	1.50	0.000			0.000	

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
1018.06	6904.50	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	0.00
1863.38	5271.11						
0.00	0.00						
2881.44	**12175.61**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	15111.159	15111.159	KN
ML	993.445	993.445	kNm
MT	0.000	0.000	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 = 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				2036.125		0.000	0.000		1445.649	0.000				0.000	0.000
Forces from Substructure				13075.034	0.000	0.000	0.000		-12627.812	0.000		0.000		0.000	0.000
CWLL-Min. Reaction case	0.75			349.70		0.000	0.000	0.710	248.291	0.000	1759.009		3.109	1087.196	0.000
Vertical component of LL Surcharge	0.20			47.552				-3.500	-166.432						
Vertical component of dynamic increment LL Surcharge	1.50			0.000				-3.500	0.000						
Total =				**15508.416**	**0.000**	**0.000**	**0.000**		**-11100.305**	**0.000**		**0.000**		**1087.196**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1	1863.376			5271.108	
due to dynamic increment of EP	1.50	0.000			0.000	
due to Live load surcharge	0.20	130.648			499.533	
due to dynamic increment of Surcharge	1.50	0.000			0.000	

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
1018.06	6904.50	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	0.00
1863.38	5271.11						
0.00	0.00						
130.65	499.53						
0.00	0.00						
3012.09	**12675.14**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	15508.416	15508.416	KN
ML	1574.836	1574.836	kNm
MT	1087.196	1087.196	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 αh x P (kN)	FT = αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				**2036.125**		**0.000**	**0.000**		**1445.649**	**0.000**				**0.000**	**0.000**
Substructure & Foundation -Portion 1				**728.173**	**0.000**	**0.000**	**0.000**		**-218.134**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure & Foundation -Portion 2				**16225.242**	**0.000**	**0.000**	**0.000**		**-13061.689**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**18989.540**	**0.000**	**0.000**	**0.000**		**-11834.175**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0.000
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure		2081.386			6684.870	
due to dynamic increment of EP		0.000			0.000	

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
1018.06	6904.50	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	0.00
2081.39	6684.87						
0.00	0.00						
3099.45	**13589.37**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	18989.540	18989.540	KN
ML	1755.195	1755.195	kNm
MT	0.000	0.000	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**19736.337**	**0.000**	**-11504.143**	**0.000**	**1686.165**
					0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0
due to Substructure		0.000	0.000		0.000	0
due to Earth pressure		2081.386			6684.870	
due to dynamic increment of EP		0.000			0.000	
due to Live load surcharge		130.648			499.533	
due to dynamic increment of Surcharge		0.000			0.000	

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
1018.06	6904.50	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	0.00
2081.39	6684.87						
0.00	0.00						
130.65	499.53						
0.00	0.00						
3230.10	**14088.90**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	19736.337	19736.337	KN
ML	2584.760	2584.760	kNm
MT	1686.165	1686.165	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT

Superstructure				**2036.125**	**0.000**	**1445.649**	**0.000**	**0.000**
Substructure & Foundation -Portion 1				**728.173**	**0.000**	**-218.134**	**0.000**	**0.000**
Substructure & Foundation -Portion 2				**12346.861**	**0.000**	**-12409.678**	**0.000**	**0.000**
Total =				**15111.159**	**0.000**	**-11182.163**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0
due to Substructure		0.000	0.000		0.000	0.000
due to Active Earth pressure	1.00	1863.376			5271.108	
due to dynamic increment of EP	1.50	0.000			0.000	

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
1018.06	6904.50	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	0.00
1863.38	5271.11						
0.00	0.00						
2881.44	**12175.61**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Downward	Upward	
P	15111.159	15111.159	KN
ML	993.445	993.445	kNm
MT	0.000	0.000	kNm

Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case

Forces due to Vertical Load

Loads	Vertical Load(P) kN.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =	**15508.416**	**0.000**	**-11100.305**	**0.000**	**1087.196**
		0.000		0.000	

Forces due to Horizontal Load

		FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)
due to Superstructure		1018.063	0.000	1756.944	6904.500	0
due to Substructure		0.000	0.000		0.000	0
due to Earth pressure		1863.376			5271.108	
due to dynamic increment of EP		0.000			0.000	
due to Live load surcharge		130.648			499.533	
due to dynamic increment of Surcharge		0.000			0.000	

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
1018.06	6904.50	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	0.00
1863.38	5271.11						
0.00	0.00						
130.65	499.53						
0.00	0.00						
3012.09	**12675.14**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces About C.G. OF Footing

	Seismic Downward	Seismic Upward	
P	15508.416	15508.416	KN
ML	1574.836	1574.836	kNm
MT	1087.196	1087.196	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 (1759.009 - 1750.162) = 0.000 kNm

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.) = 0.75 x 0.00 = 0.000 KN

Transverse Moment due to C.F. = 0.000 x (1759.009 - 1750.162) = 0.000 kNm

	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

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

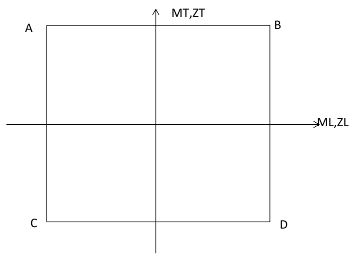
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

Base pressure on corner C = σ_C = P/A - ML/ZL - MT/ZT

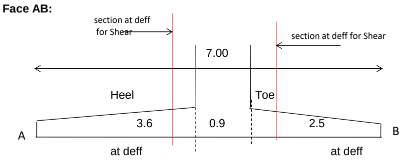
Base pressure on corner D = σ_D = P/A + ML/ZL - MT/ZT

Summary of Design Base Pressure

LOAD CASES	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	19368.321	3771.896	0.000	180.929	253.435	180.929	253.435
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	21052.123	6763.427	3372.330	188.867	318.878	153.248	283.260
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	15450.265	2442.126	0.000	149.776	196.720	149.776	196.720
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	16434.987	4937.311	2174.391	148.319	243.227	125.353	220.262
Longitudinal Seismic Dry Case							
Case 5 : DL+SIDL-Long. Seismic Dry Case	18989.540	1755.195	0.000	196.065	229.805	196.065	229.805
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	19736.337	2584.760	1686.165	205.370	255.057	187.561	237.248
Longitudinal Seismic HFL Case							
Case 7 : DL+SIDL-Long. Seismic HFL Case	15111.159	993.445	0.000	159.897	178.994	159.897	178.994
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	15508.416	1574.836	1087.196	164.505	194.778	153.022	183.295
Transverse Seismic Dry Case							
Case 9 : DL+SIDL-Trans. Seismic Dry Case	18989.540	1755.195	0.000	196.065	229.805	196.065	229.805
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	19736.337	2584.760	1686.165	205.370	255.057	187.561	237.248
Transverse Seismic HFL Case							
Case 11 : DL+SIDL-Trans. Seismic HFL Case	15111.159	993.445	0.000	159.897	178.994	159.897	178.994
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	15508.416	1574.836	1087.196	164.505	194.778	153.022	183.295



Pressure calculation along Face AB:



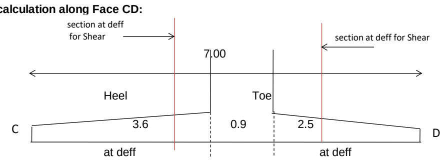
		at deff			at deff	
Case : 1	180.929	208.720	218.22	227.540	237.039	253.435
Case : 2	188.867	238.698	255.73	272.446	289.477	318.878
Case : 3	149.776	167.769	173.92	179.954	186.104	196.720
Case : 4	148.319	184.696	197.13	209.331	221.764	243.227
Case : 5	196.065	208.997	213.42	217.755	222.175	229.805
Case : 6	205.370	224.415	230.92	237.312	243.821	255.057
Case : 7	159.897	167.217	169.72	172.174	174.675	178.994
Case : 8	164.505	176.108	180.07	183.966	187.932	194.778
Case : 9	196.065	208.997	213.42	217.755	222.175	229.805
Case : 10	205.370	224.415	230.92	237.312	243.821	255.057
Case : 11	159.897	167.217	169.72	172.174	174.675	178.994
Case : 12	164.505	176.108	180.07	183.966	187.932	194.778

Average MAX Base Pressure for Design of Heel Slab-along Face AB = 222.298 kN/m^2

Average MIN Base Pressure for Design of Heel Slab-along Face AB = 161.847 kN/m^2

Average MAX Base Pressure for Design of Toe Slab-along Face AB = 295.662 kN/m^2

Pressure calculation along Face CD:



		at deff			at deff	
Case : 1	180.929	208.720	218.22	227.540	237.039	253.435
Case : 2	153.248	203.080	220.11	236.827	253.859	283.260
Case : 3	149.776	167.769	173.92	179.954	186.104	196.720
Case : 4	125.353	161.730	174.16	186.366	198.799	220.262
Case : 5	196.065	208.997	213.42	217.755	222.175	229.805
Case : 6	187.561	206.605	213.11	219.502	226.011	237.248
Case : 7	159.897	167.217	169.72	172.174	174.675	178.994
Case : 8	153.022	164.625	168.59	172.483	176.449	183.295
Case : 9	196.065	208.997	213.42	217.755	222.175	229.805
Case : 10	187.561	206.605	213.11	219.502	226.011	237.248
Case : 11	159.897	167.217	169.72	172.174	174.675	178.994
Case : 12	153.022	164.625	168.59	172.483	176.449	183.295

Average MAX Base Pressure for Design of Heel Slab-along Face CD = 204.741 kN/m^2

Average MIN Base Pressure for Design of Heel Slab-along Face CD = 149.758 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 260.044 kN/m^2

Max. Base Pressure at deff for Design of Toe Slab-along Face AB	=	289.477 kN/m²	Max. Base Pressure at deff for Design of Toe Slab-along Face CD	=	253.859 kN/m²
Max. Base Pressure at deff for Design of Heel Slab-along Face AB	=	238.698 kN/m²	Max. Base Pressure at deff for Design of Heel Slab-along Face CD	=	208.997 kN/m²

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	=	222.298 kN/m²								
Upward moment due to Base pressure	=	1440.492 kNm/m								
		factor								
Downward moment due to backfill	=	1.35 x	314.317	/	12.740	x	20	x	1.800	
	=	1199.045 kNm/m								
Downward moment due to self weight of Heel slab	=	1.35 x	29.812	/	12.740	x	25	x	1.477	
	=	116.640 kNm/m								
Net Moment at face of shaft	=	1440.492	-1199.045	-116.640 =	124.808 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	=	149.758 kN/m²							
Upward moment due to Base pressure	=	970.431 kNm/m							
		factor							
Downward moment due to backfill	=	1.35 x	314.317	/	12.740	x	10	x	1.800
	=	599.522 kNm/m							
Downward moment due to self weight of Heel slab	=	1.35 x	29.812	/	12.740	x	15	x	1.477
	=	69.984 kNm/m							
Net Moment at face of shaft	=	970.431	-599.522	-69.984 =	300.925 kNm/m		Tension at Bottom of Heel Slab		

Heel Slab - Shear Calculation at deff from face of Wall

Depth of slab at critical section	=	0.822 m					
effective depth at critical section	=	0.739 m					
Base pressure at deff from face of wall	=	238.698 kN/m²					
Shear Force due to upward pressure at deff from face of wall	=	222.034 x	2.683 x	12.740	=	7589.450	KN
		factor					
Downward Force due to backfill	=	1.35 x	314.317 x	20	=	8486.571	KN
Downward Force due to self weight of Heel slab	=	1.35 x	29.812 x	25	=	1006.142	KN
Net Shear Force	=	7589.450	-8486.571	-1006.142	=	-1903.263	KN
Net Shear Force / unit meter	=	-1903.263 /	12.740		=	149.393	KN/m

Toe Slab - Moment Calculation

Maximum Average Base Pressure for Design of Toe Slab	=	295.662 kN/m²							
Upward moment due to Base pressure	=	923.944 kNm/m							
Downward moment due to self weight of Toe slab	=	1.35 x	20.703	/	12.740	x	25	x	1.026
	=	56.2500 kNm/m							
Net Moment at face of shaft	=	923.944	-56.250 =		867.694 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	=	0.658 m							
Base pressure at deff from face of wall	=	289.477 kN/m²							
upward shear force due to base pressure	=	304.178 x	1.583 x		12.740		=	6134.480	KN
C.g. Of base pressure	=	0.679 m							
moment due to upward pressure at critical section	=	4167.799 kNm							
tanβ	=	0.280							
reduction in shear force (V_{ccd})	=	$\frac{M \tan\beta}{d}$ =	1570.131	KN					
Downward force due to self weight of toe slab	=	1.35 x	0.522	x	1.583	x	12.740	x 25	
	=	355.041 KN							
Net Shear Force at deff	=	6134.480 -	355.041	-	1570.131		=	4209.309	KN
Net Shear Force / unit meter	=	4209.309 /	12.740				=	330.401	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/mm²	
fctm	=	2.77 N/mm²	
Grade of Reinforcement Steel	=	Fe 500D	(HYSD Bars)
fy or fyk	=	500.00 N/mm²	
fyd	=	434.78 N/mm²	(fy/1.15)
Es	=	200000.00 N/mm²	

Flexural Reinforcement Calculation:

		Along Traffic Direction	
		Heel Slab	Toe Slab
Ultimate bending moment,Mu (kNm/m)	=	300.92	867.69
Effective depth required (dreq) (mm)	=	226.90	385.29
Effective depth provided (dpro) (mm)	=	917.00	917.00
Check for provided depth	=	SAFE	SAFE
R = Mu/(b d2)	=	0.36	1.03
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	108.41
Check for Neutral axis depth	=	OK	OK
Lever arm (z) , mm	=	888.14	871.90
Moment of Resistance w.r.to steel	=	776.39	1190.94
Check for Moment Capacity	=	SAFE	SAFE
Ast reqd (mm2 / m)	=	832.64	2458.64
cl. 16.6.1 (2) of IRC :112-2011			
$A_{S,min}$ = 0.26 f_{ctm} b_t d / f_{yk} >= 0.0013 b_t d	=	1321.45	1321.45
Governing Ast (mm2 / m)	=	1321.45	2458.64
Tension Reinforcement			
Dia (mm)	=	16.00	20.00
Spacing (mm)	=	200.00	200.00
+ Dia (mm)	=	16.00	20.00
Spacing (mm)	=	200.00	200.00
Ast provided (mm2 / m)	=	2010.62	3141.59
Check for Ast provided	=	OK	OK
As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least			
Secondary Reinforcement (mm2/m)	=	402.12	628.32
Dia (mm)	=	12.00	12.00
Spacing (mm)	=	200.00	150.00
Ast provided (mm2 /m)	=	565.49	753.98
Check for Ast provided	=	OK	OK

Shear Reinforcement Calculation:

		Along Traffic Direction		
		Heel Slab	Toe Slab	
Ultimate Shear Force (V_{Ed})	=	149.393	330.401	kN/m
Ast provided	=	2010.619	3141.59	mm2/m
Depth of slab at critical section	=	821.694	743.240	mm
Effective depth at critical section	=	738.694	658.240	mm
Percentage of steel provided (ρ1)	=	0.0024	0.0042	
cl. 10.3.1 of IRC :112-2011				
ρ1 = $A_{st}/(b_w d)$ <= 0.02	=	OK	OK	
Actual shear stress=v_{ED} = (V_{Ed}/b*0.9d)	=	0.225	0.558	N/mm²
Max shear capacity, 0.135 fck(1-fck/310)	=	4.192	4.192	N/mm²
Depth Check for Shear Resistance	=	SAFE	SAFE	
cl. 10.3.2(2) Eq. 10.2 of IRC :112-2010				

K = 1+Sqrt(200/d) <= 2.0	=	1.520	1.551	
cl. 10.3.2(2) Eq. 10.3 of IRC :112-2010				
v_{min} = 0.031 $K^{3/2}$ $fck^{1/2}$	=	0.344	0.354	N/mm^2
0.12 K (80 ρ1 f_{ck})$^{0.33}$	=	0.344	0.421	N/mm^2
σ_{cp} = N_{Ed} / A_c <= 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(80ρ1 f_{ck})$^{0.33}$ + 0.15σ_{cp}]b_wd subjected to minimum (v_{min} + 0.15 σ_{cp}) b_w d	=	315.716	385.809	kN
Check for Shear Reinforcement		**OK, No shear reinf. Req.**	**OK, No shear reinf. Req.**	
Balance Shear Force= $V_{Rd,s}$ = V_{Ed} - $V_{Rd,c}$	=	0.000	0.000	KN/m
b	=	12.740	12.740	m
Total Shear Force	=	0.000	0.000	kN
θ = 0.5 x sin^-1 [vEd / (0.18 fck (1- fck/250))]		1.189	2.954	
cot θ = (< 1 cotθ <2.5)	=	2.500	2.500	
fywd = 0.8 x fy/1.15	=	347.826	347.826	N/mm^2
Provide Shear Reinforcement				
Legged	=	**0**	**0**	
Dia	=	**0**	**0**	mm
Area of Shear Reinf, Asw	=	0.000	0.000	mm^2
z = 0.9 * d	=	664.825	592.416	mm
Spacing of shear Reinforcement required				
S = Asw*z* fywd* cotθ / V_{Rd}	=	0.000	0.000	mm
As per Clause 10.3.3.5 of IRC:112-2011				
Asw/ (b S) = $\rho_{w,min}$ = (0.072 $f_{ck}^{0.5}$) / f_{yk}	=	0.001	0.001	
Spacing of shear Reinforcement required	=	0.000	0.000	mm
As per Clause 16.5.2 , eq. 16.6 of IRC:112-2011				
S_{max} = 0.75 d	=	554.021	493.680	mm
Governing Spacing of Shear Reinf.	=	**0.000**	**0.000**	mm
Provided Spacing of Shear Reinf.	=	**200**	**150**	mm

SLS CHECK OF FOUNDATION

Foundation Lvl = 1750.162 m

Properties of Footing Base:

A	=	89.180	m^2
ZL	=	104.043	m^3
ZT	=	189.359	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			1250.000	0.710	887.500	0.000	0.000
SIDL except Wearing Course	1.00			80.000	0.710	56.800	0.000	0.000
Wearing Course	1.20			165.000	0.710	117.150	0.000	0.000
				1495.000		**1061.450**		**0.000**
Substructure & Foundation -Portion 1								
Dirt Wall-Uniform portion	1.00	25	2.293	57.330	0.340	19.492	0.000	0.000
Dirt Wall-Tapered portion	1.00	25	0.467	11.681	0.340	3.972	0.000	0.000
Bracket - Uniform portion	1.00	25	1.147	28.665	0.040	1.147	0.000	0.000
Bracket - Tapered portion	1.00	25	0.573	14.333	0.090	1.290	0.000	0.000
Cap - (uniform portion)	1.00	25	2.752	68.796	0.550	37.838	0.000	0.000
Cap - (corbel portion)	1.00	25	0.000	0.000	0.550	0.000	0.000	0.000
Cantilever Return Wall-Rectangle portion	1.00	25	2.400	60.008	-1.900	-114.029	0.000	0.000
Cantilever Return Wall-Traingle portion	1.00	25	4.135	103.363	-1.234	-127.498	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1.00	25		28.000	0.340	9.520	0.000	0.000
Aproach Slab	1.00	25	6.689	167.213	0.040	6.688	0.000	0.000
				539.387		**-161.581**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	25.189	629.730	-1.700	-1070.541	0.000	0.000
Abutment Shaft	1.00	25	53.278	1331.943	0.570	759.472	0.000	0.000
Back filling over heel slab	1.00	20	314.317	6286.349	-1.731	-10879.422	0.000	0.000
Front Filling over toe slab	1.00	20	74.848	1496.950	2.188	3275.242	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	29.812	745.290	-1.377	-1026.207	0.000	0.000
Toe slab	1.00	25	20.703	517.563	2.026	1048.396	0.000	0.000
portion between heel & toe	1.00	25	11.466	286.650	0.550	157.658	0.000	0.000
Vertical Components of active earth pressure	1.00			757.562	-3.500	-2651.469	0.000	0.000
				12215.103		**-10362.743**		**0.000**
Total				**14249.490**		**-9462.874**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		747.500	1756.944	5069.545
due to Earth pressure	1.00	2081.386		6684.870

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
747.50	5069.54	0.00	0.00
2081.39	6684.87		
2828.886	**11754.415**	**0.000**	**0.000**

Summary of Forces

P	14249.490	KN
ML	2291.541	kNm
MT	0.000	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		14249.490		-9462.874		0.000
CWLL-Max. Reaction case	1.00	932.327	0.710	661.952	2.411	2248.220
Vertical Components of LL Surcharge	0.80	190.208	-3.500	-665.728	0.000	0.000
Total		**15372.025**		**-9466.650**		**2248.220**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		747.500	1756.944	5069.545
due to Earth pressure	1.00	2081.386		6684.870
due to Live load surcharge	0.80	522.592		1998.130

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
747.50	5069.54	0.00	0.00
2081.39	6684.87		
522.59	1998.13		
3351.478	**13752.545**	**0.000**	**0.000**

Summary of Forces

P	15372.025	KN
ML	4285.895	kNm
MT	2248.220	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				**1495.000**		**1061.450**		**0.000**

Substructure & Foundation -Portion 1				**539.387**		**-161.581**		**0.000**
Substructure & Foundation -Portion 2								
Solid Return wall	1.00	25	25.189	629.730	-1.700	-1070.541	0.000	0.000
Shaft above HFL	1.00	25	29.203	730.078	0.615	449.002	0.000	0.000
Shaft below HFL	1.00	15	24.075	361.119	0.570	205.910	0.000	0.000
Back filling above HFL over heel slab	1.00	20	190.198	3803.960	-1.700	-6466.732	0.000	0.000
Back filling below HFL over heel slab	1.00	10	130.712	1307.124	-1.774	-2318.425	0.000	0.000
Front Filling over toe slab	1.00	10	74.848	748.475	2.188	1637.621	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	29.812	447.174	-1.377	-615.724	0.000	0.000
Toe slab	1.00	15	20.703	310.538	2.026	629.038	0.000	0.000
Portion between Heel & Toe	1.00	15	11.466	171.990	0.550	94.595	0.000	0.000
Vertical Components of active earth pressure	1.00			678.213	-3.500	-2373.747	0.000	0.000
				9321.656		**-9807.770**		**0.000**
Total				**11356.043**		**-8907.901**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		747.500	1756.944	5069.545
due to Earth pressure	1.00	1863.376		5271.108

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
747.50	5069.54	0.00	0.00
1863.38	5271.11		
2610.876	**10340.653**	**0.000**	**0.000**

Summary of Forces

P	11356.043	KN
ML	1432.753	kNm
MT	0.000	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		11356.043		-8907.901		0.000
CWLL-Min. Reaction case	1.00	466.273	0.710	331.054	3.109	1449.594
Vertical Components of LL Surcharge	0.80	190.208	-3.500	-665.728	0.000	0.000
Total		**12012.525**		**-9242.574**		**1449.594**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ FND. (kNm)
due to Superstructure		747.500	1756.944	5069.545
due to Earth pressure	1.00	1863.376		5271.108
due to Live load surcharge	0.80	522.592		1998.130

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
747.50	5069.54	0.00	0.00
1863.38	5271.11		
522.59	1998.13		
3133.468	**12338.784**	**0.000**	**0.000**

Summary of Forces

P	12012.525	KN
ML	3096.209	kNm
MT	1449.594	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 (1759.009 - 1750.162)

= 0.000 kNm

Base pressure on corner A = σ_A = P/A - ML/ZL + MT/ZT

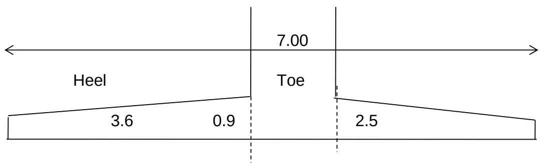
Base pressure on corner B = σ_B = P/A + ML/ZL + MT/ZT

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

LOAD CASES	P	ML	MT	Design Base Pressure 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	14249.490	2291.541	0.000	137.759	181.808	137.759	181.808
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	15372.025	4285.895	2248.220	143.050	225.437	119.305	201.691
Normal HFLCase							
Case 3 : DL+SIDL-Normal HFL Case	11356.043	1432.753	0.000	113.568	141.109	113.568	141.109
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	12012.525	3096.209	1449.594	112.596	172.114	97.286	156.803

Pressure calculation along Face AB:



Case 1 :	137.759	160.41	166.076	181.808
Case 2:	143.050	185.42	196.013	225.437
Case 4:	113.568	127.73	131.273	141.109
Case 5:	112.596	143.21	150.858	172.114

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 164.235 kN/m^2

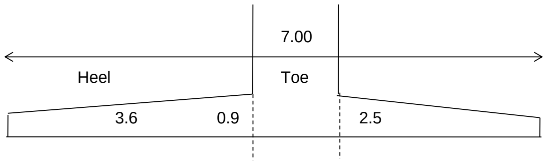
Average Base Pressure for Design of Toe Slab-along Face AB = 210.725 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face AB = 149.086 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face AB = 173.942 kN/m^2

Pressure calculation along Face CD:



Case 1 :	137.759	160.41	166.076	181.808
Case 2:	119.305	161.67	172.267	201.691
Case 4:	113.568	127.73	131.273	141.109
Case 5:	97.286	127.89	135.547	156.803

For Rare Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 149.086 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 186.979 kN/m^2

For Quasi Permenant Combination

Average Base Pressure for Design of Heel Slab-along Face CD = 149.086 kN/m^2

Average Base Pressure for Design of Toe Slab-along Face CD = 173.942 kN/m^2

Moment Calculation

	Rare Combination		Quasi-Parmenant		
	Heel Slab	Toe Slab	Heel Slab	Toe Slab	
Max Average Base Pressure	164.24	210.73	149.09	173.94	kN/m^2
Upward moment due to Base pressure	1064.25	658.52	966.08	543.57	kNm/m
Downward moment due to backfill	888.18	0.00	888.18	0.00	kNm/m
Downward moment due to self weight of slab	86.40	41.67	86.40	41.67	kNm/m
Net Moment	**89.66**	**616.85**	**-8.51**	**501.90**	kNm/m
	Tension at Bottom of Heel Slab	**Tension at Bottom of Toe Slab**	**Tension at Top of Slab**	**Tension at Bottom of Toe Slab**	

Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff = 1.2

Ecm = 32308.25 N/mm^2

Es = 200000.00 N/mm^2

Eceff = $\frac{Ecm}{(1 + \phi)}$ = 1.47E+04

Modular Ratio (m) = Es/ Eceff = 13.62

		Rare Combination		Quasi Permanent Comb.		
		Heel Slab	**Toe Slab**	**Heel Slab**	**Toe Slab**	
Working bending moment, M	**=**	**89.66**	**616.85**	**-8.51**	**501.90**	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)	=	0.538	3.701			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	20.000			mm
Effective Depth deff (dy)	=	917.000	917.000			mm
Ast provided	=	2010.619	3141.593			mm^2/m
Percentage of steel , pt	=	0.0022	0.0034			
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m	=	0.216	0.262			
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	198.381	240.584			mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	718.619	676.416			mm
Depth of neutral axis from c.g. Of tesnion steel (ys)	=	635.619	591.416			mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2				
Icr	=	1.674E+10	2.422E+10			mm^4
Maximum compressive stress in concrete	=	**1.062**	**6.128**	**-0.101**	**4.986**	**< 16.8, SAFE**
Maximum Tensile stress in steel	=	**46.358**	**205.156**	**-4.398**	**166.926**	**< 300, SAFE**

Check For Crack Width in Quasi-Permanent Load Combination

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)	=	415.000	425.000	mm
Provided Spacing	=	100.000	100.000	mm
Check for Applicability of Formula	=	OK	OK	
Maximum crack spacing , $S_{r\,max}$	=	3.4 c +	$\frac{0.425\ k1\ k2\ \varphi}{\rho_{\rho\,eff}}$	
K1	=	0.800	0.800	for deformed bars
K2	=	0.500	0.500	for bending
depth of neutral axis , yc	=	198.381	240.584	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.		
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	207.500	207.500	mm
Ac, eff = Dx * hc,eff	=	207500.000	207500.000	mm
$\rho_{\rho\,eff}$ = As/Ac eff	=	0.010	0.015	
Maximum crack spacing , $S_{r\,max}$	=	535.710	479.568	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct\,eff} (1 + \alpha_e \rho_{\rho\,eff})}{\rho_{\rho\,eff}}$	/ Es	
tensile stress in steel, ,σsc	=	-4.398	166.926	N/mm^2
Kt	=	0.500	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	2.771	N/mm^2
αe = Es/Ecm	=	6.190	6.190	
(εsm - εcm)	=	-0.00001	0.0005	
Crack width , Wk=Sr max (εsm - εcm)	**=**	**0.000**	**0.240**	mm
Check	**=**	**SAFE**	**SAFE**	

CALCULATION OF ULS FORCES FOR DESIGN OF ABUTMENT SHAFT

Abutment shaft bottom M = 1751.162 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			1687.500	0.160	270.000	0.000	0.000
SIDL except Wearing Course	1.35			108.000	0.160	17.280	0.000	0.000
Wearing Course	1.75			240.625	0.160	38.500	0.000	0.000
				2036.125		**325.780**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1.35	25	2.293	77.396	-0.210	-16.253	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.467	15.769	-0.210	-3.312	0.000	0.000
Bracket - Uniform portion	1.35	25	1.147	38.698	-0.510	-19.736	0.000	0.000
Bracket - Tapered portion	1.35	25	0.573	19.349	-0.460	-8.900	0.000	0.000
Cap - (uniform portion)	1.35	25	2.752	92.875	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800	-0.210	-7.938	0.000	0.000
Aproach Slab	1.35	25	6.689	225.737	-0.510	-115.126	0.000	0.000
				507.623		**-171.265**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1.35	25	53.278	1798.123	0.065	116.889	0.000	0.000
Total				**4341.871**		**271.404**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1018.06	1756.944	5886.44
due to Earth pressure	1.5	2358.92		6585.49

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1018.06	5886.44	0.00	0.00
2358.92	6585.49	0.00	0.00
3376.98	**12471.93**	**0.000**	**0.000**

Summary of Forces

P	4341.87	KN
ML	12743.33	kNm
MT	0.00	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		4341.871		271.404		0.000
CWLL-Max. Reaction case	1.5	1398.490	0.160	223.758	2.411	3372.330
Total		**5740.361**		**495.163**		**3372.330**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1018.063	1756.944	5886.437
due to Earth pressure	1.5	2358.919		6585.488
due to Live load surcharge	1.2	681.379		2264.562

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1018.06	5886.44	0.00	0.00
2358.92	6585.49		
681.38	2264.56		
4058.360	**14736.488**	**0.000**	**0.000**

Summary of Forces

P	5740.361	KN
ML	15231.651	kNm
MT	3372.330	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				2036.125		325.780		0.000
Substructure-Portion 1				507.623		-171.265		0.000
Substructure-Portion 2								
Shaft above HFL	1.35	25.000	29.203	985.605	0.065	86.495	0.000	0.000
Shaft below HFL	1.35	23.500	24.075	763.767	0.020	20.826	0.000	0.000
Total				**4293.120**		**261.837**		**0.000**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1018.063	1756.944	5886.437
due to Earth pressure	1.5	2192.074		5417.542

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1018.06	5886.44	0.00	0.00
2192.07	5417.54	0.00	0.00
3210.137	**11303.980**	**0.000**	**0.000**

Summary of Forces

P	4293.120	KN
ML	11565.816	kNm
MT	0.000	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		4293.120		261.837		0.000
CWLL-Max. Reaction case	1.5	699.410	0.160	111.906	3.109	2174.391
Total		**4992.530**		**373.742**		**2174.391**

Forces due to Horizontal Load

	load factor	FL (KN)	R.L. of Force (m)	ML (kNm)
due to Superstructure		1018.063	1756.944	5886.437
due to Earth pressure	1.5	2192.074		5417.542
due to Live load surcharge	1.2	681.379		2264.562

Forces along Long. Axis		Forces along Trans. Axis	
FL Cosθ	ML Cosθ	FL Sinθ	ML Sin θ
1018.06	5886.44	0.00	0.00
2192.07	5417.54		
681.38	2264.56		
3891.516	**13568.542**	**0.000**	**0.000**

Summary of Forces

P	4992.530	KN
ML	13942.284	kNm
MT	2174.391	kNm

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

Seismic Effect Factor = 1.50 αh= 0.000 In Longitudinal direction
αh= 0.000 In Transverse direction
αv= 0.000 In Vertical direction

Weight of shaft below Ground level = 514.2 KN

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 = 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			1687.500		0.000	0.000	0.160	270.000	0.000	1757.477		0.000	0.000	0.000
SIDL except Wearing Course	1.35			108.000		0.000	0.000	0.160	17.280	0.000	1758.258		0.000	0.000	0.000
Wearing Course	1.75			240.625		0.000	0.000	0.160	38.500	0.000	1757.809		0.000	0.000	0.000

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 = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
				2036.125		**0.000**	**0.000**		**325.780**	**0.000**				**0.000**	**0.000**
Substructure-Portion 1															
Dirt Wall-Uniform portion	1.35	25	2.293	77.396	0.000	0.000	0.000	-0.210	-16.253	0.000	1757.509	0.000	0.000	0.000	0.000
Dirt Wall-Tapered portion	1.35	25	0.467	15.769	0.000	0.000	0.000	-0.210	-3.312	0.000	1757.148	0.000	0.000	0.000	0.000
Bracket - Uniform portion	1.35	25	1.147	38.698				-0.510	-19.736						
Bracket - Tapered portion	1.35	25	0.573	19.349				-0.460	-8.900						
Cap - (uniform portion)	1.35	25	2.752	92.875	0.000	0.000	0.000	0.000	0.000	0.000	1756.793	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1.35	25	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1756.643	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier	1.35	25		37.800				-0.210	-7.938				0.000		
Aproach Slab	1.35	25	6.689	225.737				-0.510	-115.126				0.000		
				507.623	**0.000**	**0.000**	**0.000**		**-171.265**	**0.000**		**0.000**		**0.000**	**0.000**
Substructure-Portion 2															
Abutment Shaft	1.35	25	53.278	**1798.123**	**0.000**	**0.000**	**0.000**	**0.065**	**116.889**	**0.000**	**1754.903**	**0.000**	**0.000**	**0.000**	**0.000**
Total =				**4341.871**	**0.000**	**0.000**	**0.000**		**271.404**	**0.000**		**0.000**		**0.000**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (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
due to Superstructure		1018.063	0.000	1756.944	5886.437	0.000	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1572.613			4390.326		1572.61	4390.33						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
							2590.68	**10276.76**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4341.871	4341.871	KN
ML	10548.167	10548.167	kNm
MT	0.000	0.000	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Forces from Superstructure				2036.125		0.000	0.000		325.780	0.000				0.000	0.000
Forces from Substructure				2305.746	0.000	0.000	0.000		-54.376	0.000		0.000		0.000	0.000
CWLL-Max. Reaction case	0.20			186.47		0.000	0.000	0.160	29.834	0.000	1759.009		2.411	449.644	0.000
Total =				**4528.337**	**0.000**	**0.000**	**0.000**		**301.239**	**0.000**		**0.000**		**449.644**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (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
due to Superstructure		1018.063	0.000	1756.944	5886.437	0.000	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1572.613			4390.326		1572.61	4390.33						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
due to Live load surcharge	0.20	113.563			377.427		113.56	377.43						
due to dynamic increment of Surcharge	1.50	0.000			0.000		0.00	0.00						
							2704.24	**10654.19**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4528.337	4528.337	KN
ML	10955.429	10955.429	kNm
MT	449.644	449.644	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 = αh x P (kN)	FT = 0.3 x αh x P (kN)	Fv = 0.3 x αv x P (kN)	Long. Ecc. (eL1) (m)	ML = PxeL1	MLs due to Fv	C.g. of Force (m)	MLs due to FL	Transv. Ecc. (eT) (m)	MT = PxeT	MTs due to FT
Superstructure				2036.125		0.000	0.000		325.780	0.000				0.000	0.000
Substructure-Portion 1				507.623	0.000	0.000	0.000		-171.265	0.000		0.000		0.000	0.000
Substructure-Portion 2															
Shaft above HFL	1.350	25.000	29.203	985.605	0.000	0.000	0.000	0.065	64.070	0.000	1755.153	0.000	0.000	0.000	0.000
Shaft below HFL	1.350	23.500	24.075	763.767	0.000	0.000	0.000	0.020	15.427	0.000	1753.412	0.000	0.000	0.000	0.000
				1749.372	**0.000**	**0.000**	**0.000**		**79.497**	**0.000**		**0.000**		**0.000**	**0.000**
Total =				**4293.120**			**0.000**		**234.013**	**0.000**				**0.000**	
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (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
due to Superstructure		1018.063	0.000	1756.944	5886.437	0.000	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1461.383			3611.695		1461.38	3611.69						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
							2479.45	**9498.13**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Downward	Upward	
P	4293.120	4293.12	KN
ML	9732.145	9732.14	kNm
MT	0.000	0.00	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 = 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				2036.125		0.000	0.000		325.780	0.000				0.000	0.000
Forces from Substructure				2256.995	0.000	0.000	0.000		-91.767	0.000		0.000		0.000	0.000
CWLL-Min. Reaction case	0.20			93.25		0.000	0.000	0.160	14.921	0.000	1759.009		3.109	289.919	0.000
Total =				**4386.375**	**0.000**	**0.000**	**0.000**		**248.933**	**0.000**		**0.000**		**289.919**	**0.000**
							0.000			0.000					

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (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
due to Superstructure		1018.063	0.000	1756.944	5886.44	0.000	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0.000		0.00	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1461.383			3611.69		1461.38	3611.69						
due to dynamic increment of EP	1.50	0.000			0.00		0.00	0.00						
due to Live load surcharge	0.20	113.563			377.43		113.56	377.43						

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
due to dynamic increment of Surcharge	1.50	0.000			0.00		0.00	0.00						
							2593.01	**9875.56**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4386.375	4386.375	KN
ML	10124.493	10124.493	kNm
MT	289.919	289.919	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4341.871**	**0.000**	**271.404**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		1018.063	0	1756.944	5886.437	0	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0		0.000	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1572.613			4390.326		1572.61	4390.33						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
							2590.68	**10276.76**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4341.871	4341.871	KN
ML	10548.167	10548.167	kNm
MT	0.000	0.000	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4528.337**	**0.000**	**301.239**	**0.000**	**449.644**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		1018.063	0	1756.944	5886.437	0	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure	1.00	1572.613			4390.326		1572.61	4390.33						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
due to Live load surcharge	0.20	113.563			377.427		113.56	377.43						
Surcharge	1.50	0.000			0.000		0.00	0.00						
							2704.24	**10654.19**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4528.337	4528.337	KN
ML	10955.429	10955.429	kNm
MT	449.644	449.644	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 αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4293.120**	**0.000**	**234.013**	**0.000**	**0.000**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		1018.063	0	1756.944	5886.437	0.000	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Active Earth pressure	1.00	1461.383			3611.695		1461.38	3611.69						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
							2479.45	**9498.13**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4293.120	4293.120	KN
ML	9732.145	9732.145	kNm
MT	0.000	0.000	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.	Fv = 0.3 x αv x P (kN)	ML = PxeL1	MLs due to Fv	MT = PxeT
Total =				**4386.375**	**0.000**	**248.933**	**0.000**	**289.919**
					0.000		0.000	

Forces due to Horizontal Load

	load factor	FL (KN)	FT (KN)	R.L. of Force (m)	ML (kNm)	MT (kNm)	FL Cosθ	ML Cosθ	FT Sinθ	MT Sinθ	FL Sinθ	ML Sin θ	FT Cosθ	MT Cosθ
							Forces along Long. Axis				Forces along Trans. Axis			
due to Superstructure		1018.063	0	1756.944	5886.437	0	1018.06	5886.44	0.00	0.00	0.00	0.00	0.00	0.00
due to Substructure		0.000	0		0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
due to Earth pressure	1.00	1461.383			3611.695		1461.38	3611.69						
due to dynamic increment of EP	1.50	0.000			0.000		0.00	0.00						
due to Live load surcharge	0.20	113.563			377.427		113.56	377.43						
Surcharge	1.50	0.000			0.000		0.00	0.00						
							2593.01	**9875.56**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

Summary of Forces

	Seismic Downward	Seismic Upward	
P	4386.37	4386.37	KN
ML	10124.49	10124.49	kNm
MT	289.92	289.92	kNm

Centrifugal Force : Normal Case

Centrifugal Force (C.F.)	=	1.50	x	0.00		=	0.000 KN	Normal
Transverse Moment due to C.F.	=	0.000	x (	1759.009 -	1751.162)	=	0.000 kNm	

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

Centrifugal Force : Seismic Case

Centrifugal Force (C.F.)	=	0.20	x	0.00		=	0.000 KN	
Transverse Moment due to C.F.	=	0.000	x (	1759.009 -	1751.162)	=	0.000 kNm	

Seismic	0.00	0.00	0.00	0.00

Summary of ULS Forces for Design of Abutment Shaft

LOAD CASES		Total forces at bottom of abutment shaft P	ML	MT
Normal Dry Case		kN	kNm	kNm
Case 1 : DL+SIDL-Normal Dry Case		4341.871	12743.330	0.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case		5740.361	15231.651	3372.330
Normal HFLCase				
Case 3 : DL+SIDL-Normal HFL Case		4293.120	11565.816	0.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case		4992.530	13942.284	2174.391
Longitudinal Seismic Dry Case				
Case 5 : DL+SIDL-Long. Seismic Dry Case	DN	4341.871	10548.167	0.000
	UP	4341.871	10548.167	0.000
Case 6 : DL+SIDL+LL-(Maximum Reaction Case)-Long. Seismic Dry Case	DN	4528.337	10955.429	449.644
	UP	4528.337	10955.429	449.644
Longitudinal Seismic HFL Case				
Case 7 : DL+SIDL-Long. Seismic HFL Case	DN	4293.120	9732.145	0.000
	UP	4293.120	9732.145	0.000
Case 8 : DL+SIDL+LL- (Minimum Reaction Case)-Long. Seismic HFL Case	DN	4386.375	10124.493	289.919
	UP	4386.375	10124.493	289.919
Transverse Seismic Dry Case				
Case 9 : DL+SIDL-Trans. Seismic Dry Case	DN	4341.871	10548.167	0.000
	UP	4341.871	10548.167	0.000
Case 10 : DL+SIDL+LL-(Maximum Reaction Case)-Trans. Seismic Dry Case	DN	4528.337	10955.429	449.644
	UP	4528.337	10955.429	449.644
Transverse Seismic HFL Case				
Case 11 : DL+SIDL-Trans. Seismic HFL Case	DN	4293.120	9732.145	0.000
	UP	4293.120	9732.145	0.000
Case 12 : DL+SIDL+LL- (Minimum Reaction Case)-Trans. Seismic HFL Case	DN	4386.375	10124.493	289.919
	UP	4386.375	10124.493	289.919
	MAX =	**5740.36**	**15231.65**	**3372.33**

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)	=	0.900 m
Depth of Wall (D)	=	12.740 m
Area of Concrete (Ac)	=	11.466 m^2
Clear Cover to earth faces	=	75 mm
Clear Cover to non earth faces	=	50 mm
Maximum Dia of Vertical Reinf.	=	25 mm
Dia of Horizontal Reinf.	=	12 mm
Effective cover	=	137 mm

As per Clause 7.6.4.1 of IRC:112-2011

Ultimate axial force (Pu) = **5740.36** kN

0.1 fcd Ac = 0.1 x 15.63 x 11466000
= 17925180 N
= **17925.18** 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)	=	15231.65 kNm	=	**1195.577 kNm/m**

Check For Depth of Wall :

Mult = 0.165 x fck x b x d^2
= 1195.58 kNm/m

b = 1000.00 mm

Effective Depth Required (dreq) = $SQRT\left(\frac{1195.58 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

(dreq) = 455.001 mm

Total Depth Required (Dreq) = 554.50 mm
Total Depth Provided (Dprov) = 900.00 mm **OK**
Effective depth provided(deff) = 763.00 mm

R= Mu/(b d^2) = **2.05**

Minimum Longitudinal Reinforcement in wall on each face
= 0.0012 x b x D
Ast min = **1080.00** mm^2/m

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\,D} = \frac{fck \{ 1 - \sqrt{1- 4.598 R/fck} \}}{2\,fy}$

= 0.0051

Ast_{req} = 4582.558 mm^2/m

Ast required = max(Astmin, Astreq) = **4582.56** mm^2/m
Total are of steel required in full length = **58381.79** mm^2

Provide	25	mm dia	@	100.00	mm c/c	=	4908.74	mm^2/m	OK
Provide	0	mm dia	@	90.00	mm c/c	=			

Effective length of shaft = 12508 mm

Calculation of reinforcement in numbers

Provide	25	mm dia	-	125.00	nos	=	61359.23	mm^2	OK
Provide	0	mm dia	-	139.00	nos	=			

Percentage of steel = **0.535** %

Check for Moment of Resistance of Section due to Steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

= $\frac{0.0035 \times 763.00}{0.0035 + 0.0022}$

= 470.66 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 4908.74}{0.36 \times 35.00 \times 1000.00}$

= 169.38 mm **OK**

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

z = d - 0.416 . X
= 763.00 - 70.46
= 692.54 mm

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

MR = fyd . Ast . z

= 434.78 x 4908.74 x 692.54

= 1.48E+09 Nmm /m

= 1478.03 kNm/m > 1195.58 kNm/m

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

= 0.0012 x b x D
Ast min = **1080.00** mm^2/m
= **13759.20** mm^2

Provide	12	mm dia	@	100.00	mm c/c	=	1130.97 mm^2/m	OK

Calculation of reinforcement in numbers

Provide	12	mm dia	@	125.00	nos	=	14137.17 mm^2	OK

PART 3 : HORIZONTAL REINFORCEMENT CALCULATION

Horizontal Reinforcement for wall

maximum of following = 0.2500 x 6039.71 = 1509.928 As per IRC: 112-2011 , Clause
= 0.001 x 9.00E+05 = 900.000 16.3.2

Maximum Horizontal Reinf. **1510** mm^2 per meter

Min dia of bar = 0.250 x 25 = 6.25 mm
or 8 mm

Maximum Spacing between <= 300 mm c/c

2 Legged	12 dia	@	140 c/c	=	1615.676 mm^2	OK

Confinement Reinforcement

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

Distance between links or ties (ST) = 1/3 x 751 = 250.333
or 200.00 mm

Governing Spacing = 200.00 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)

SL	=		5	x	25 =	125 mm
	or	1/5		x	751 =	150.2 mm
Min	=		100 mm			

2 Legged	12 dia	@	100 c/c	=	2261.947 mm^2	OK
24 Legged	10 dia	@	100 c/c	=	18849.556 mm^2	
40 links	10 dia	@	100 c/c	=	31415.927 mm^2	
					52527.429 mm^2	

Minimum Confinement Reinforrcement:

nk = $\frac{NED}{Ak\ fck}$ = $\frac{5740361.3}{401310000}$ = 0.0143

AC = 11.466 mm^2

ACC = 0.775 x 12.640 = 9.796 mm^2

ρ L = 0.00536 per meter

ρ L = 0.06834

fyd = 434.783

fcd = 15.633

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

$\omega_{wd.req}$ = 0.2171

ω_{wd} = max ($\omega_{wd,req}$, 0.12) = 0.2171

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

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

Volumetric ratio,

Asw	=	52527.429 mm^2
SL	=	100.000 mm
b	=	751.000 mm
ρw	=	0.699
$\omega_{wd.c}$	=	19.452

$\omega_{wd,c}$ ≥ ω_{wd} as per equation 17.7 of IRC:112-2011

$\omega_{wd,c}$ = 19.45 OK

Length of Potential Plastic Hinges Refer clause 17.2.1.4 of IRC:112-2011

nk = $\frac{NED}{Ak\ fck}$ = 0.0143 < 0.30

CALCULATION OF SLS FORCES FOR DESIGN ABUTMENT SHAFT

Abutment shaft bottom lvl = 1751.162 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			1250.000	0.160	200.000	0.000	0.000
SIDL except Wearing Course	1			80.000	0.160	12.800	0.000	0.000
Wearing Course	1			137.500	0.160	22.000	0.000	0.000
				1467.500		**234.800**		**0.000**
Substructure-Portion 1								
Dirt Wall-Uniform portion	1	25	2.293	57.330	-0.210	-12.039	0.000	0.000
Dirt Wall-Tapered portion	1	25	0.467	11.681	-0.210	-2.453	0.000	0.000
Bracket - Uniform portion	1	25	1.147	28.665	-0.510	-14.619	0.000	0.000
Bracket - Tapered portion	1	25	0.573	14.333	-0.460	-6.593	0.000	0.000
Cap - (uniform portion)	1	25	2.752	68.796	0.000	0.000	0.000	0.000
Cap - (corbel portion)	1	25	0.000	0.000	0.000	0.000	0.000	0.000
RCC Railing or Crash Barrier or Crash Barrier	1	25		28.000	-0.210	-5.880	0.000	0.000
Aproach Slab	1	25	6.689	167.213	-0.510	-85.278	0.000	0.000
				376.017		**-126.863**		**0.000**
Substructure-Portion 2								
Abutment Shaft	1	25	53.278	1331.943	0.065	86.584	0.000	0.000
Total				**3175.460**		**194.522**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		747.500	1756.944	4322.045
due to Earth pressure	1	1572.613		4390.326
				8712.371

Summary of Forces at Bottom of abutment shaft

P	3175.460	KN
ML	8906.892	kNm
MT	0.000	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		3175.460		194.522		0.000
CWLL-Max. Reaction case	1	932.327	0.160	149.172	2.411	2248.220
Total		**4107.787**		**343.694**		**2248.220**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		747.500	1756.944	4322.045
due to Earth pressure	1	1572.613		4390.326
due to Live load surcharge	0.8	454.252		1509.708
				10222.079

Summary of Forces at Bottom of abutment shaft

P	4107.787	KN
ML	10565.773	kNm
MT	2248.220	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				1467.500		234.800		0.000
Substructure-Portion 1				376.017		-126.863		0.000
Substructure-Portion 2								
Shaft above HFL	1.000	25.000	29.203	730.078	0.07	47.46	0.00	0.00
Shaft below HFL	1.000	23.500	24.075	565.754	0.02	11.43	0.00	0.00
				3139.348		**166.824**		**0.000**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	Moment @ Shaft (kNm)
due to Superstructure		747.500	1756.944	4322.045
due to Earth pressure	1	1461.383		3611.695
				7933.740

Summary of Forces at Bottom of abutment shaft

P	3139.348	KN
ML	8100.564	kNm
MT	0.000	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		3139.348		166.824		0.000
CWLL-Max. Reaction case	1	466.273	0.160	74.604	3.109	1449.594
Total		**3605.622**		**241.428**		**1449.594**

Forces due to Horizontal Load

	load factor	Horizontal Force (KN)	R.L. of Force (m)	ML @ Shaft (kNm)
due to Superstructure		747.500	1756.944	4322.045
due to Earth pressure	1	1461.383		3611.695
due to Live load surcharge	0.8	454.252		1509.708
				9443.448

Summary of Forces at Bottom of abutment shaft

P	3605.622	KN
ML	9684.876	kNm
MT	1449.594	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 (1759.009 - 1751.162) = 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	3175.460	8906.892	0.000
Case 2 : DL+SIDL+LL-(Maximum Reaction Case)-Normal Dry Case	4107.787	10565.773	2248.220
Normal HFLCase			
Case 3 : DL+SIDL-Normal HFL Case	3139.348	8100.564	0.000
Case 4 : DL+SIDL+LL- (Minimum Reaction Case)-Normal HFL Case	3605.622	9684.876	1449.594

IN RARE COMBINATION

Max SLS Moment = 10565.773 kNm

Max Moment per meter = 829.339 kNm/m

IN QUASI-PERMANENT

Max SLS Moment = 8906.892 kNm

Max Moment per meter = 699.128 kNm/m

Check For Stresses in Rare and Quasi-Permanent Load Combination

Creep Coeff = 1.2

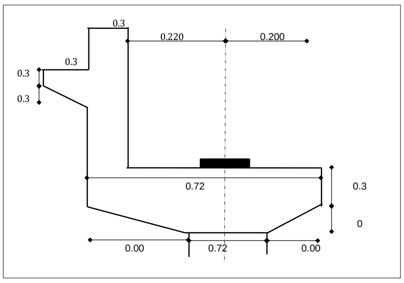
		Rare Combination		Quasi permanent	
		Short term	**Long Term**		
Working bending moment, M	=	829.34	829.34	699.13	kNm/m
Dx (unit width of shaft)	=	1.00	1.00	1.00	m
Dy (Thickness of shaft)	=	0.90	0.90	0.90	m
Section Modulus (ZL) of uncracke	=	0.14	0.14	0.14	m^3

Bending Stress (M/ZL)	=	6.143	6.143	5.179	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	14685.57	14685.57	N/mm^2
Modular Ratio (m)	=	6.19	13.62	13.62	
Clear Cover, c	=	75.000	75.000	75.00	mm
Maximum dia used, ϕ	=	25.000	25.000	25.00	mm
Effective Depth deff (dy)	=	763.000	763.000	763.00	mm
Ast provided	=	4908.739	4908.739	4908.74	mm^2/m
Percentage of steel , pt	=	0.0054	0.0054	0.0054	
k=sqrt{ 2 pt *m + (pt*m)^2 } - pt* m		0.226	0.316	0.316	
Depth of neutral axis from extreme Compression face (yc = k * dy)	=	172.739	240.955	240.955	mm
Depth of neutral axis from extreme tension face (yt = dy-yc)	=	590.261	522.045	522.045	mm
Depth of neutral axis from c.g. Of tension steel (ys)	=	502.761	434.545	434.545	mm
Cracked moment of Inertia (Icr)	=	Dx *(k * dy)^3/3 + m Ast * (dy - k * dy)^2			
Icr	=	1.231E+10	2.288E+10	2.288E+10	mm^4
Maximum compressive stress in concrete	=	**11.642**	**8.733**	**7.362**	**< 16.8, SAFE**
Maximum tensile stress in concrete	=	39.782	18.921	15.950	
Maximum Tensile stress in steel	=	**209.760**	**214.490**	**180.814**	**< 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)	=	437.500	mm
Provided Spacing	=	100.000	mm
Check for Applicability of Formula	=	OK	
Maximum crack spacing , $S_{r\ max}$	=	3.4 c +	0.425 k1 k2 φ / $\rho_{\rho\ eff}$
K1	=	0.800	for deformed bars
K2	=	0.500	for bending
depth of neutral axis , yc	=	240.955	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	, where Ac,eff =effective area of concrete in tension surrounding the reinf.	
hc eff = Min of 2.5 (Dy - dy) ,Dy - yc/3 , Dy/2	=	342.500	mm
Ac, eff = Dx * hc,eff	=	342500.000	mm
$\rho_{\rho\ eff}$ = As/Ac eff	=	0.014	
Maximum crack spacing , $S_{r\ max}$	=	551.537	mm
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - k_t f_{ct\ eff} (1 + \alpha_e \rho_{\rho\ eff})}{\rho_{\rho\ eff}}$	/ Es
tensile stress in steel, ,σsc	=	180.814	N/mm^2
Kt	=	0.500	
Tensile strength of concrete = fct eff = fctm	=	2.771	N/mm^2
αe = Es/Ecm	=	13.619	
(εsm - εcm)	=	0.00054	
Crack width , Wk=Sr max (εsm - ε	=	**0.299**	**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.

Assuming a cap thickness of = 225 mm
Volume of abutment cap = 225 x 720 x 12740
= 2.06E+09 mm^3

as per cl. 710.8.7 of IRC : 78- 2014
Quantity of steel = 1 % of volume

= 1/100 x 2.06E+09 = 2.06E+07 mm^3

(a) Longitudinal steel

Quantity of steel to be provided in longitudinal direction
= 1.03E+07 mm^3

Clear cover = 50 mm

Length of bar = 12740 -100 = 12640 mm

Area of steel required in longitudinal direct = 1.03E+07 / 12640 = 816.4082 mm^2 (top +Bottom)

Provide 7 Nos. of 12 mm dia bar as longitudinal steel on top & Bottom face of abutment cap.

Provided steel = 792 mm^2

(b) Transverse steel

Volume of steel to be provided in transverse direction = 1.03E+07 mm^3

Volume of steel required per meter = 1.03E+07 / 12.74 = 8.10E+05 mm^3/m

Provide 2 L 12 mm dia bar @ 150 mm c/c stirrups

Length of each stirrups = 720 -100 -12 = 608 mm

Volume of steel provided per meter = 9.17E+05 mm^3/m **OK**

Thickness of Abutment Cap (uniform) s = 0.3 m
Thickness of Abutment Cap (tapered) = 0 m

C.G. of dirt wall from face of abutment shaft (a) = -0.150 m
Overall depth of Abutment cap at face of abutment shaft = 0.300 m
Clear cover = 50 mm
Diameter of the main bar = 12 mm
Effective cover d' = 56 mm
Effective depth of cap d = 0.244 m
For the section to be designed as corbel " a / d " shall be less than 1.

Hence a / d = -0.15 / 0.24
= -0.615 **< 1.0**

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

Hence s / d = 0.3 / 0.24
= 1.23 **> 0.5** **Proceed with the design**

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 = 5.41688 kN
Self Weight of Bracket = 0.135 m^3/m x 25
= 3.375 kN
Total Dead Load = 8.79188 kN
Load Factor = 1.35
Ultimate Dead Load = **11.869** 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 "	**311.87**	kN

Horizontal Load (H_u)

H_u = 1.7 x actual horizontal force in working load condition
but not less than
= 0.2 * V_u

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 < = 0.15 f_c'**

where; V_u = 311.87 kN
= 311869.03 N

bd = 1000 x 244 sqmm
= 244000.00 mm^2

V_u / b d = 311869.04 / 244000

V_u / b d = **1.28** N / mm^2

f_c' = 28 - day standard cylinder strength of concrete used.
= 0.80 times the standard cube strength
Grade of Concrete M 35
0.15 f_c' = 0.15 x 0.8 x 35
= **4.20** N / mm^2 **Ensured**

Hence **V_u / b d < = 0.15 f_c'** is **Ensured**

STEP II

Calculation of Shear Friction Reinforcement " A_{vf} "

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

where; f_{sy} = yeild stress value of the reinforcement used.
= 500.00 N/ mm^2

	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

Type of Surface (1 / 2 / 3 / 4) ? = **1.00**

m = 1.40

(Note: Only monolithic construction is recommended)

$A_{vf} = \frac{311.87 \times 1000}{0.87 \times 500 \times 1.4}$ mm^2

A_{vf} = **524.15** mm^2

STEP III

Calculation for Direct Tension Reinforcement " A_t ".

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

H_u = 68.00 kn

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

= **156.32** mm^2

STEP IV

Calculation for Flexural Tension Reinforcement " A_f "

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

$= \frac{311.87 \times 1000 \times -150 + 68 \times 1000 (300 - 56)}{0.87 \times 500 \times 244}$

= **-284.42** mm^2

STEP V

Total Primary Tensile Reinforcement " A_s "

A_s >= ($A_f + A_t$)
>= (2 / 3 $A_{vf} + A_t$)
>= (0.04 f_c' / f_{sy}) b d

Provide the largest of these three magnitudes as A_s.

($A_f + A_t$) = -284.43 + 156.33 mm^2
= -128.10 mm^2

(2 / 3 $A_{vf} + A_t$) = 2 / 3 x 524.15 + 156.3: mm^2
= 505.75 mm^2

(0.04 f_c' / f_{sy}) b d = 0.04 x 0.8 x 35 / 500 x 1000 x 244
= 546.56 mm^2

Hence A_s = **546.56** 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 >= 0.25 * A s
>= 0.333 * A_{vf}

Provide the largest of these two magnitudes as Ah.

0.25 * A s = 0.25 x 546.56
= -142.21 mm^2

0.333*A_{vf} = 0.333 x 524.15
= 174.54 mm^2

Hence A_h = **174.54** mm^2

STEP VII

The total steel in vertical stirrups is " A_v "

V c = 10 bd in kgs ***where b & d are in cms***
= 10 x 100 x 24.4
= 24400.00 kg
= 244.00 kN

Pitch = 200 mm

$A_v = \frac{0.50 * (V_u - V_c) * p}{f_{sy} d}$

$= \frac{0.5 \times (311.87 - 244) \times 200 \times 1000}{500 \times 244}$

A_v = **55.63** mm^2

Reinforcement Details

Total Primary Tensile Reinforcement " A_s "

A_s = **546.56** mm^2

Diameter of Primary Steel = 12 mm

Area of one bar = 113.10 mm^2

Spacing of Bar = **150** mm c/c

$A_{s\ provided}$ =
= 753.982 mm^2 > A_s R/F is adequate

provide	150 mm c/c 12 mm diameter bars as main reinforcement

Horizontal Steel (Closed Stirrups) " A_h "

A_h = **174.54** mm^2

A_h**/m** = **174.54** mm^2 /m

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

2 / 3 d = (2 / 3) x 244
= 163.00 mm

Diameter of Stirrup Bar = 12.00 mm

Area of one bar = 113.10 mm^2

No. of layers of Stirrups = 2.00 nos.

Spacing of Stirrups = 81.500 mm

No. of legs of stirrups = 2.00

$A_{h\ provided}$ = 113.1 x 2 x 2
= 452.389 mm^2 > A_h R/F is adequate

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

Vertical Steel (Closed Stirrups) " A_v "

A_v = **55.63** mm^2

A_v = 0.25 x As = 136.64 mm^2

Av max = 136.64

A_v**/m** = **136.64** mm^2 /m

Diameter of Stirrup Bar = 10.00 mm

Area of one bar = 78.54 mm^2

Pitch = 200 mm

No. of legs of stirrups = 2.00

$A_{h\ provided}$ =
= 785 mm^2 > A_v R/F is adequate

provide	2 legged. 10 mm diameter bars as vertical stirrups at 200 mm spacing per meter width

DESIGN OF DIRT WALL

Dirt wall will be designed as a vertical cantilever.

1.) NORMAL CASE

1a. Dead Load

Self Weight of Dirt Wall	=	2.760	m^3 x	25.00	=	69.011 kN
Self Weight of Dirt Wall/ m	=	69.011 /		12.74	=	5.417 kN

1b. Live Load

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

Vertical Load on Dirt Wall = 200 kN

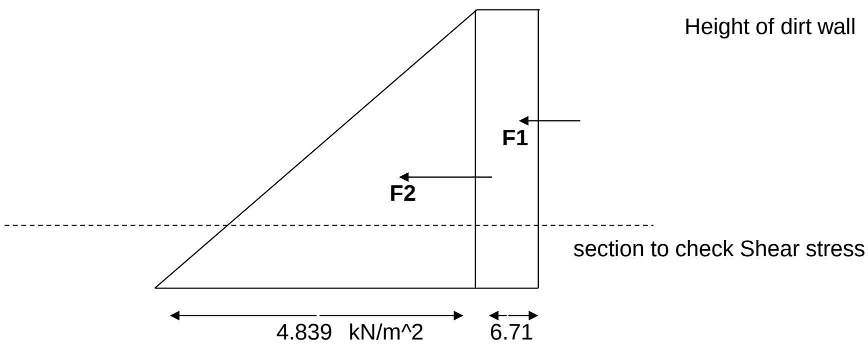
<u>Braking Load</u>

Assuming 20% braking Force i.e. 0.2 *200 = 40.000 kN acting at 1.2 m above deck

Effective Width = 2.79 m

Moment (due to Breaking) = $\dfrac{40.000 \times 2.066}{2.79}$ = 29.620 kNm/m

<u>1c. EARTH PRESSURE</u>



Height of dirt wall = 0.866 m

Normal Earth Pressure

Earth Pressure Diagram

Intensity for rectangular portion	=	0.279	x 20	x 1.2	=	6.705	kN/m^2
F1	=	6.705	x 0.87	x 1.00	=	5.807	kN/m
Intensity for triangular portion	=	0.2794	x 20	x 0.866	=	4.839	kN/m^2
F2	=	0.50	x 4.84	x 0.866	x 1.00 =	2.095	kN/m

Moment @ RL **1756.94** m (at dirt wall base)

M1 = 5.807 x 0.433 = 2.514 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 = 2.095 x 0.364 = 0.762 kN.m/m

<u>Design Horizontal Forces (Normal Case):</u>

Load Factor For Live Load Surcharge = 1.2
Ultimate Moment due to Live Load Surcharge = 3.017 kN.m/m

Load Factor For Earth Pressure = 1.5
Ultimate Moment due to Earth Pressure = 1.143 kN.m/m

Load Factor For Braking Force = 1.5
Ultimate Moment due to Braking Force = 44.430 kN.m/m

Total Ultimate Moment	**=**	**48.590 kN.m/m**

Material Property:

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 N/mm^2
Design Yield Strength of Reinforcement, fyd	=	434.783 N/mm^2
Modulus of Elasticity of Steel (Es)	=	200000 N/mm^2

(a) Vertical steel on earth face As per Clause 16.3.1 of IRC:112-2011

Adorting clear cover on either face	=	50 mm
Minimum Dia of Reinforcement	=	12 mm
Maximum Spacing of Steel	=	200 mm
Thickness of dirtwall	=	0.300 m
Available effective depth	=	300 -50 -6 = 244 mm

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 48.59 kNm/m

Effective Depth of Cap Required (dreq) = $SQRT\left(\frac{48.59 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$ = 91.727 mm

Total Depth Required (Dreq) = 147.73 mm
Total Depth Provided (Dprov) = 300.00 mm **OK**

R= Mu/(b d^2) = 0.816

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d}$ = $\frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2\ fy}$ = 0.002

Ast_{req} = 470.803 mm^2/m

As per Clause 16.3.1 of IRC:112-2011

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

Maximum (Ast_{req}, Ast_{min}) = 470.803 mm^2/m

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

Provide Ast = 565 mm^2/m) OK

Percentage of Steel Provided = 0.232 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035\ .\ d}{(0.0035 + fyd/ Es)}$ = $\frac{0.0035 \times 244}{0.0035 + 0.00217}$

= 150.5134 mm

Depth of Neutral Axis , = $\frac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b}$

= $\frac{435 \times 565}{0.36 \times 35.00 \times 1000}$ = 19.523 mm **OK**

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

z = d - 0.416 . X = 244 - 8.121
= 235.879 mm

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

MR = fyd . Ast . z = 434.78 x 565 x 235.88

= 5.8E+07 Nmm = 57.994 kNm/m > 48.59 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	117.701 mm^2/m
Governing Ast	=	150.000 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

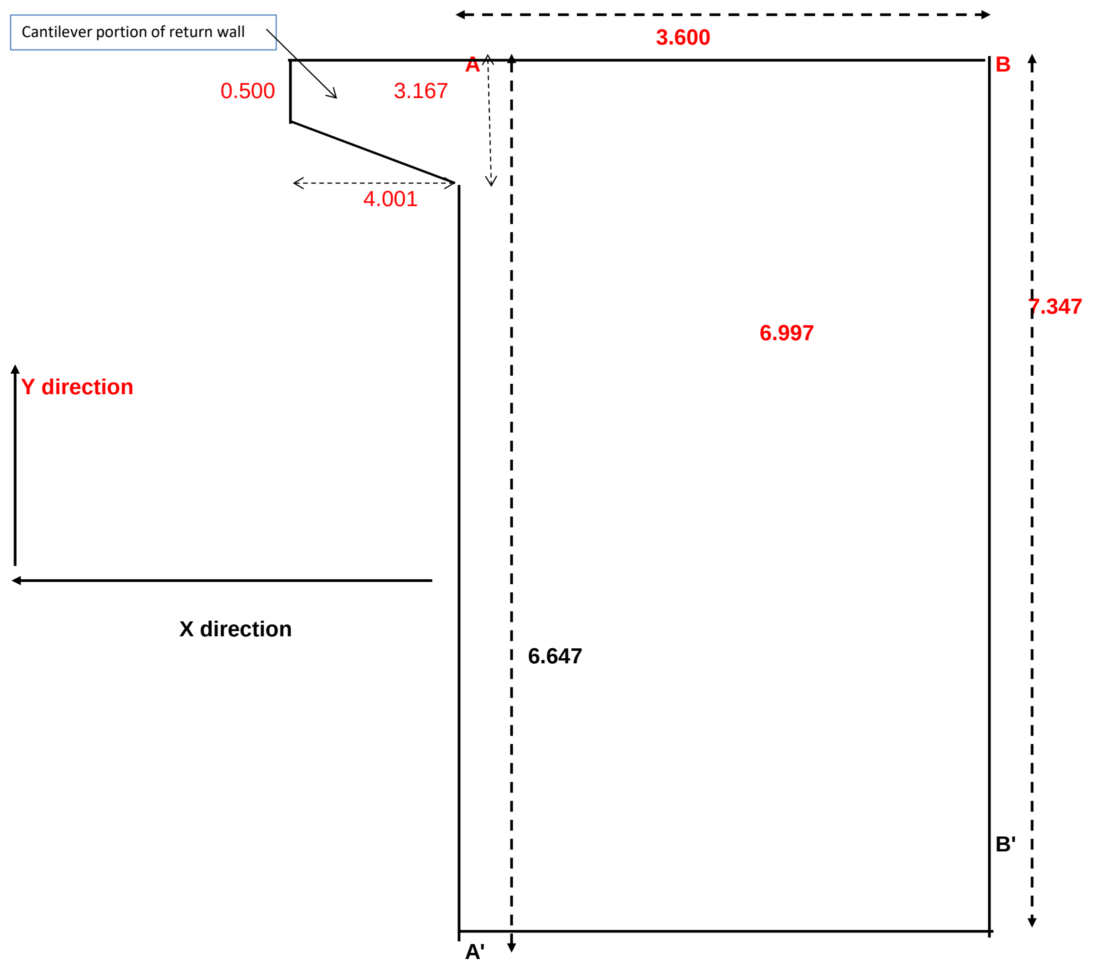
Provide 10 mm dia bar @ 200 mm c/c as vertical steel at earth face.

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	=	3.60	m
Avg. Heigth of Solid Return	b	=	6.997	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 = 0.515

a/b =	0.5	β1 =	0.631	β2 =	0.632
a/b =	0.75	β1 =	1.246	β2 =	1.186

For uniformly distributed load over entire plate

For, a/b = 0.515 β1 = 0.667

β2 = 0.664

Live Load Surcharge Intensity:

q = 0.2794 x 20.00 x 1.200 = 6.705 kN/m^2

Max. $.\sigma b = \dfrac{\beta 1 \times q \times b^2}{(t1)^2}$

$.\sigma a = \dfrac{\beta 2 \times q \times b^2}{(t2)^2}$

$.\sigma b = \dfrac{0.667 \times 6.705 \times 48.958}{0.250}$

At bottom edge = 875.421 kN/m2 = **0.875 MPa**

For 1000 mm of width , Z = $\dfrac{1000 \times 250000}{}$

6
= 4.17E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 0.875 x 4.167E+07

= 36475864 Nmm/m = **36.476 kN.m/m**

σ a = (0.664 x 6.705 x 48.958) / 0.250

= 872 = **0.8721 MPa**

For 1000 mm of height , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence, Moment /m height along X direction -

Mx /m height = 0.8721 x 4.167E+07 = 3.634E+07 Nmm/m

= **36.337 kN.m/m**

For triangular loading due to Earth Pressure

Refer Load case No. 11 d

a/b =	0.500	β1 =	0.328	β2 =	0.200
a/b =	0.75	β1 =	0.537	β2 =	0.276

For, a/b = 0.515 β1 = **0.340**

β2 = **0.204**

q = 0.279 x 20.00 x 7.00

= 39.097 kN/m^2

Max. σ b = (β1 x q x b^2) / (t1)^2

σ a = (β2 x q x b^2) / (t2)^2

σ b = (0.340 x 39.097 x 48.958) / 0.25

= 2604.16 kN/m^2

= **2.604 MPa**

For 1000 mm of width , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence Moment /m width along Y direction -

My /m width = 2.604 x 4.167E+07

= 108506735 Nmm/m = **108.507 kN.m/m**

σ a = (0.204 x 39.097 x 48.958) / 0.25

= 1565.1 kN/m^2 = 1.565 MPa

For 1000 mm of height , Z = (1000 x 250000) / 6

= 4.167E+07 mm^3

Hence Moment /m height along X direction -

Mx /m height = 1.565 x 4.167E+07 = 6.521E+07 Nmm/m

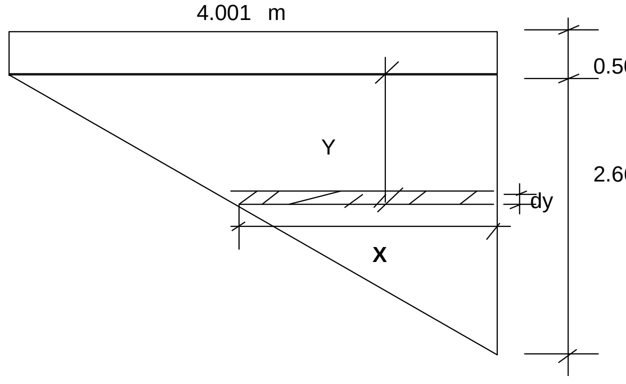
= **65.210 kN.m/m**

Total Moment in Solid Return Wall / m height = 101.547 kN.m/m

Total Moment in Solid Return Wall / m width = 144.983 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 =		141	kN.m/m
Total Moment(My) in Solid Return Wall / m width =		207	kN.m/m



X = 4.001 + -1.500 y

Earth pressure due to LL surchage = 0.279 x 20.000 x 1.200
= 1.20 x 6.705 = **8.046** kN / m^2
load factor

Earth pressure at a depth of 0.500 m from top
= 0.279 x 20 x 0.5
= 1.50 x 2.794 = **4.191** kN / m^2
load factor

Earth pressure at a depth y from top = 0.279 x 20 x y
= 1.50 x 5.588 = **8.382 y**
load factor

A *LL surchage on top* *0.5 m*

Force = 8.046 x 4.00 x 0.5 = 16.095 kN

Moment at face BB' = 16.095 x (4.001 / 2 + 3.600)
(Mx) 1
= **90.133** kN.m

B *LL surchage on triangular portion*

Force for unit strip = 8.046 x **X** x dy = 8.046 X dy

(dMx) 2 = 8.046 **X** dy x { **X** / 2 + 3.60 }

= 8.046 x { **X** ^2 / 2 + 3.6 **X** } dy

= 8.046 x { (4.001 + -1.500 y) ^2 / 2 +
3.60 x (4.001 + -1.500 y) } dy

= 8.046 x { (8.002 + 1.125 y^2 +
-6.001 y) + 14.402 + -5.400 y } dy

= 8.046 x (22.404 + 1.125 y^2 +
-11.401 y) dy

After integrating between limits 0 and 2.667 m

(Mx) 2 = 8.046 x (22.404 x 2.667 + 0.375
x 2.667 ^3 + -5.700 x 2.67 ^2)

= 8.046 x (59.751 + 7.114 + -40.546)

= **211.766** kN.m

C *Earth press. on top* *0.5 m*

Force = 0.5 x 4.191 x 4.001 x 0.5
= 4.191 kN

Moment at face BB' = 4.191 x (4.001 / 2 + 3.600)
(Mx) 3
= **23.472 kN.m**

D *Earth pressure on triangular portion*

Force for unit strip = (4.191 + 8.382 y) **X** x dy

(dMx) 4 = (4.191 + 8.382 y) X x { **X** / 2 + 3.6 }

= (4.191 **X** ^2 + 15.087 **X** + 4.1908 **X** ^2 x y +
(Part 1) *(Part 2)* *(Part 3)*

30.173 **X** x y) dy
(Part 4)

Part 1 :
(dMx) 4.1 = 4.191 **X** ^2 dy

= 4.191 x (4.001 + -1.500 y) ^2 dy

= 4.191 x (16.0040003 + 2.2500 y^2 + -12.002 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.1 = 4.191 x (16.0040003 x 2.667 + 0.7500 x 2.67 ^3 + -6.001 x 2.667 ^2)

= 4.191 x (16.004 x 2.6670 + 0.750 x 18.97 + -6.001 x 7.113)

= 4.191 x (42.683 + 14.228 + -42.6827)

= 4.191 x (14.228)

= **59.624 kN.m**

Part 2 :
(dMx) 4.2 = 15.087 **X** dy

= 15.087 x (4.001 + -1.500 y) dy
= (60.354 + -22.630 y) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.2 = 60.354 x 2.6670 + -11.315 x 2.6670 ^2

= 160.965 + -80.483 **= 80.483 kN.m**

Part 3 :

(dMx) 4.3 = 4.191 **X** ^2 x y dy

= 4.191 x (4.001 + -1.500 y) ^2 x y x dy

= 4.191 x (16.0040003 y + 2.250 y ^3 + -12.0015 y^2) x dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.3 = 4.191 x (8.00200013 x 2.6670 ^2 + 0.5625 x 2.66700 ^4 + -4.00050 x 2.66700 ^3)

= 4.191 x (56.917 + 28.459 + -75.890)

= 4.191 x (9.486) = **39.754 kN.m**

Part 4 :

(dMx) 4.4 = 30.173 **X** x y dy

= 30.173 x (4.001 + -1.500 y) x y x dy
= (120.709 y + -45.260 y^2) dy

After integrating between limits 0.000 and 2.667 m

(Mx) 4.4 = 60.354 x 2.667 ^2 + -15.087 x 2.667 ^3

= 429.294 + -286.196 **= 143.098 kN.m**

(Mx) 4 = 59.624 + 80.483 + 39.754 + 143.098
= 322.959 kN.m

Total Moment at face BB' = (Mx) 1 + (Mx) 2 + (Mx) 3 + (Mx) 4

90.133 + 211.766 + 23.472 + 322.959

= **648.331 kN.m**

Horizontal moment per meter = 648.331 / 6.997 = **92.658 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) =
= 141.420 + 92.66
= 234.08 kN.m / m

Adorting clear cover on either face	=	75 mm		
Minimum Dia of Reinforcement	=	16 mm		
Maximum Spacing of Steel	=	125 mm		
Thickness of wall	=	0.500 m		
Available effective depth	=	500	-75	-8
	=	417 mm		

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 234.08 kNm/m

Effective Depth of Cap Required (dreq) = SQRT((234.08 x 1000000) / (0.165 x 35.00 x 1000))

Effective Depth of Cap Required (dreq) = 201.328 mm

Total Depth Required (Dreq) = 284.33 mm
Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 1.35

Area of Steel Required:

pt / 100 = Ast_{req} / b d = fck { 1 - sqrt(1- 4.598 R/fck) } / 2 fy
= 0.003

Ast_{req} = 1353.256 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1353.256 mm^2/m

Provide 16 mm dia bar @ 125 mm c/c as Horizontal steel at earth face.

Provide Ast= 1608 mm^2/m) OK

Percentage of Steel Provided = 0.386 %

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

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

= 257.230 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 1608}{0.36 \times 35.00 \times 1000}$

= 55.504 mm OK

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

z = d - 0.416 . X
= 417 - 23.090
= 393.910 mm

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

MR = fyd . Ast . z

= 434.78 x 1608 x 393.910

= 2.75E+08 Nmm

= 275.480 kNm/m > 234.08 kNm/m

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

Provide 12 mm dia bar @ 125 mm c/c as Horizontal steel at non earth face.

Provided Ast = **905 mm^2/m)**

2. Design for Face A'B'

Moment in Solid Return /m width = 206.53 kN.m / m

Adorting clear cover on either face	=	75 mm			
Minimum Dia of Reinforcement	=	16 mm			
Maximum Spacing of Steel	=	150 mm			
Thickness of wall	=	0.500 m			
Available effective depth	=	500	-75	-16	-8
	=	401 mm			

Check for Depth:

Mult = 0.165 x fck x b x d^2 = 206.53 kNm/m

Effective Depth of Cap Required (dreq) = $\mathrm{SQRT}\left(\frac{206.53 \times 1000000}{0.165 \times 35.00 \times 1000}\right)$

Effective Depth of Cap Required (dreq) = 189.111 mm

Total Depth Required (Dreq) = 272.11 mm
Total Depth Provided (Dprov) = 500.00 mm **OK**

R= Mu/(b d^2) = 1.28

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\,d} = \frac{fck \{ 1 - \mathrm{sqrt}(1 - 4.598\, R/fck) \}}{2\, fy}$

= 0.003

Ast_{req} = 1238.744 mm^2/m

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 600 mm^2/m

Maximum (Ast_{req}, Ast_{min}) = 1238.744 mm^2/m

Provide 16 mm dia bar @ 150 mm c/c as vertical steel at earth face.

Provide Ast= 1340 mm^2/m) OK

Percentage of Steel Provided = 0.3343 %

Provide 12 mm dia bar @ 150 mm c/c as Vertical steel at non earth face.

Check for Moment of Resistance of section due to steel

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035 \cdot d}{(0.0035 + fyd/Es)}$

= $\frac{0.0035 \times 401}{0.0035 + 0.00217}$

= 247.36 mm

Depth of Neutral Axis , X = $\frac{fyd \cdot Ast}{0.36 \cdot fck \cdot b}$

= $\frac{434.78 \times 1340}{0.36 \times 35.00 \times 1000}$

= 46.253 mm OK

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

z = d - 0.416 . X
= 401 - 19.241
= 381.76 mm

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

MR = fyd . Ast . z

= 434.78 x 1340 x 381.759

= 2.22E+08 Nmm

= 222.484 kNm/m > 206.53 kNm/m

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

b) Cantilever Portion of Return Wall

Self-weight of cantilever portion of return wall	=	23	kN/m
Crash Barrier weight	=	10.0	kN/m
Total Load	=	33	kN/m
Moment at Cantilever Face	=	263	kNm
Load Factor	=	1.35	
Design Moment	=	356	kNm
Effective Depth	=	3104.500	mm

R= Mu/(b d^2) = 0.07

Area of Steel Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\,d} = \frac{fck \{ 1 - sqrt(1- 4.598 R/fck) \}}{2 fy}$

= 0.000

Ast_{req} = 263.985 mm^2

Minimum Reinforcement = 0.12/100 b x D As per Clause 16.3.1 of IRC:112-2011
= 1862.7 mm^2

Maximum (Ast_{req}, Ast_{min}) = 1862.7000 mm^2

Provide 25 4 = 1963 mm^2

DESIGN OF RCC SOLID SLAB

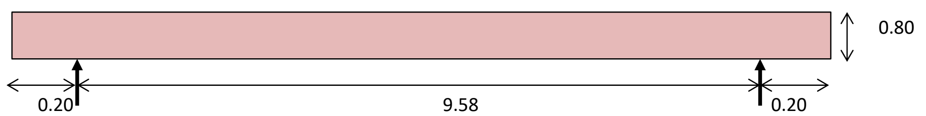
SPAN - 10.00m (SKEW 0^o)

OVERALL DECK WIDTH –12.5 m

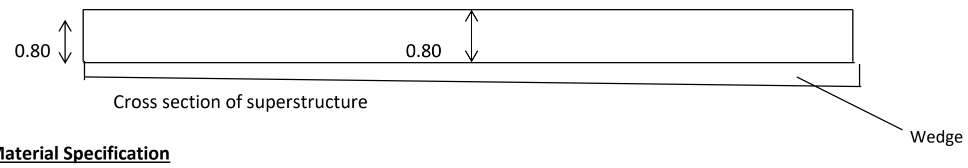
INPUTS FOR DESIGN OF RCC SOLID SLAB

1.0 Input Data :

Clear span of the slab	=	9.18	m
Bearing over support	=	0.40	m
Span between Centre to centre of bearing	=	9.58	m
Thickness of filler type exp. Gap	=	0.02	m
Overall span of slab	=	10.00	m
Overall width of slab	=	12.50	m
Carriageway width	=	7.50	m
Width of footpath	=	1.50	m
Camber	=	2.50	%(uni directional)
Depth of Slab at Carriageway edge	=	0.800	m
Depth of Slab at Carriageway centre	=	0.800	m
Wearing Coat thickness	=	0.065	m
Clear cover to steel	=	0.050	m
Diameter of main reinforcement bar	=	0.020	m
Avg. Depth of Solid Slab	=	0.80	m
Effective Depth of Slab	=	0.80 - 0.05 - 0.010	
	=	0.740	m
Effective Span Min(l+ws , l+d) L	=	9.58	m



Span Arrangment of superstructure



Cross section of superstructure

2.0 Material Specification

Concrete Grade	=	M 30		
Characteristic Compressive Strength of Concrete,fck	=	30.00	Mpa at 28 days	
Design Compressive strength of Concrete,fcd	=	13.40	Mpa at 28 days	(0.67/1.5 * fck)
Tensile strength of concrete , fctm	=	2.50	MPa	
Strain at reaching Characteistic Strength, ec2	=	0.02		
Ultimate Strain, ecu2	=	0.035		
Modulus of Elasticity of Concrete (Ec)	=	2.74E+04	N/mm^2	(5000 x sqrt (fck)
Ecm	=	3.12E+04	N/mm^2	
Steel Grade		Fe 500		(HYSD Steel)
Yield Strength of Reinforcement, fy or fyk	=	500	Mpa	
Design Yield Strength of Reinforcement, fyd	=	434.78	Mpa	(1/1.15 * fy)
Modulus of Elasticity of Steel (Es)	=	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	= 14.4	N/mm^2
Maximum tensile stress in steel under rare combination	=	300	N/mm^2	

3.0 Calculation of Load ,Bending Moment & Shear Force

3.1 Self-weight of Deck Slab

Average thickness of deck slab	=	0.80	m
U.D.L. Due to deck slab weight	=	20.00	kN/m^2
Bending Moment Deck Slab weight	=	229.44	kNm/m
Reaction	=	100.00	kN/m
Shear Force at deff section	=	81.20	kN/m
Shear Force at mid-span	=	0.00	

INPUTS FOR DESIGN OF RCC SOLID SLAB

3.1 SIDL1 Due to Crash Barrier & Railing

Due to crash barrier	=	2.00	x 8.00 =		16.00 kN/m
U.D.L due to crash barrier	=	16.00	/ 12.50 =		1.28 kN/m^2
Due to RCC Railing	=	2.00	x 0.00 =		0.00 kN/m
	=	0.00	/ 12.50 =		0.00 kN/m^2
Total SIDL1 due to crash barrier & railing	=	1.28 kN/m^2			
Bending Moment due to SIDL1	=	14.68 kNm/m			
Reaction	=	6.40 kN/m			
Shear Force at deff section	=	5.20 kN/m			
Shear Force at mid-span	=	0.00			

3.2 SIDL2 Due to Wearing Course

due to wearing course and additional overlay	=	2.00 kN/m^2
Bending Moment due to SIDL2	=	22.94 kNm/m
Reaction	=	10.00 kN/m
Shear Force at deff section	=	8.12 kN/m
Shear Force at mid-span	=	0.00

3.3 Footpath Load

Refer clause 206.3 of IRC:6-2014

P1	=	0 kg/m^2			
Effective span L	=	9.58 m			
Intensity of FPLL , P	=	P1	-	(40x L - 300)/9	
	=	0	-	9.24 =	-9.24 kg/m^2
P due to both side footpath	=	2	x	-9.24 =	-18.49 kg/m^2
Intensity of FPLL per running meter	=	-0.18	x	1.50 =	-0.277333333 kN/m
U.D.L. Due to FPLL	=	-0.28	/	12.50 =	-0.02 kN/m^2
Bending Moment due to FPLL	=	-0.25 kNm/m			
Reaction	=	-1.39 kN/m			
Shear Force at deff section	=	-1.13 kN/m			
Shear Force at mid-span	=	0.00			

3.4 CWLL Bending Moment

			interpolation	
b/L	=	1.30	1.30	2.24
α	=	2.25 For simply supported slab	1.40	2.36

beff = $\alpha * a*(1 - a / L) + b_1$

A = $\alpha * a* (1 - a / L)$

b_1 = Breadth of the concentration area of the load

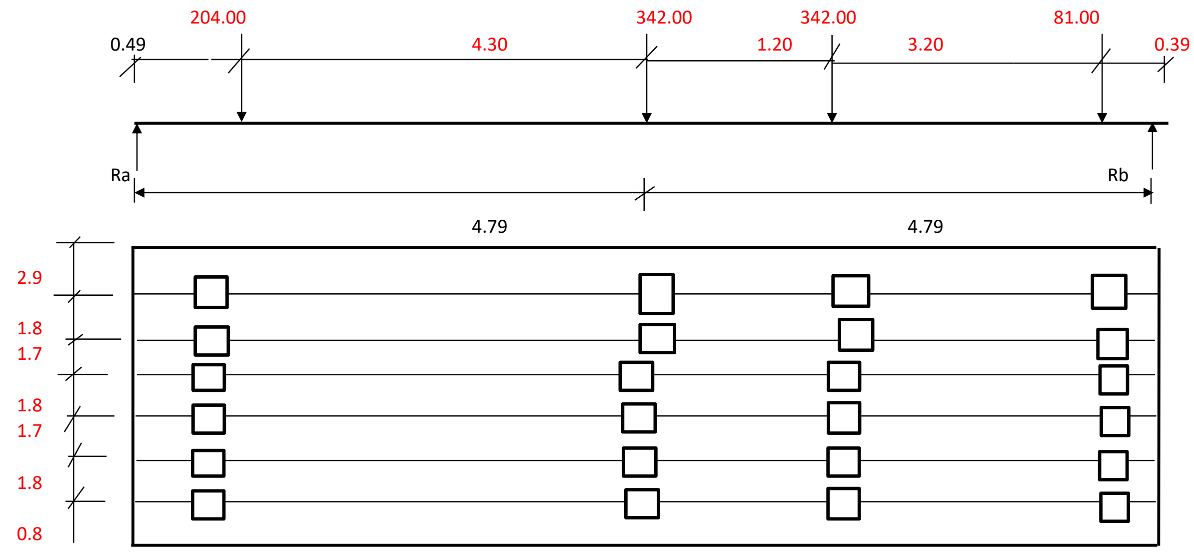
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
114.00	0.50	0.25
68.00	0.38	0.20
27.00	0.20	0.15

b_1	=	0.63	114 kN
b_1	=	0.51	68 kN
b_1	=	0.33	27 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

For Class A - 3 Lanes

I.F = 1.289

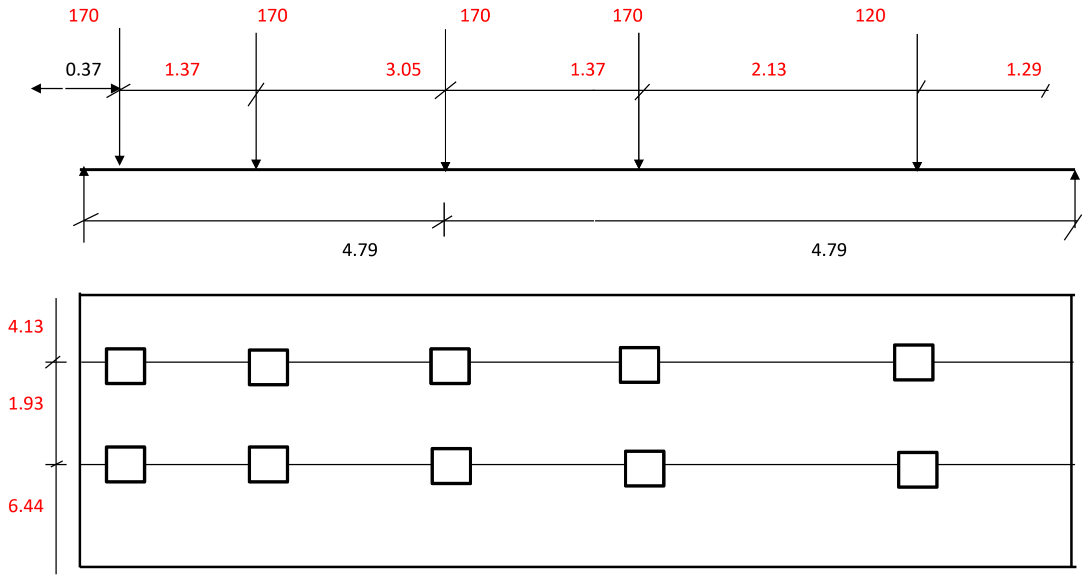


Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Bending Moment (kNm/m)
81.00	0.39	0.84	0.33	1.17	7.02	2.90
342.00	3.59	5.04	0.63	5.67	12.44	63.62
342.00	4.79	5.38	0.63	6.01	12.50	84.45
204.00	0.49	1.04	0.51	1.55	9.32	6.91
						157.89

For Class A - 3 Lanes	B . M =	**157.89** kN-m/m

CLASS 70 RW Loading

Impact Factor = 1.25



INPUTS FOR DESIGN OF RCC SOLID SLAB

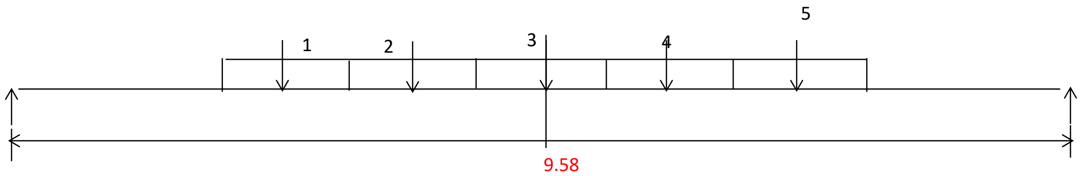
Dimensions of 70RWheel Tyre presssure = 5.273 kg/cm^2 527.30 kN/m^2

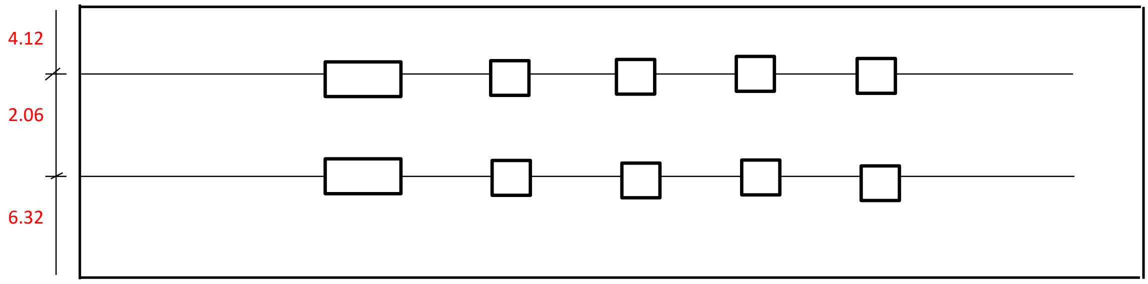
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
170.00	0.86	0.37
120.00	0.86	0.26

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
120	1.29	2.51	0.99	3.50	5.43	17.83
170	3.42	4.94	0.99	5.93	7.86	46.24
170	4.79	5.38	0.99	6.37	8.30	61.33
170	1.74	3.20	0.99	4.19	6.12	30.22
170	0.37	0.80	0.99	1.79	3.58	10.99
					B . M =	**166.60**

CLASS 70 R(T) Loading

Impact Factor = 1.10





Class 70R (T) = 700.00 KN
Length of track load = 4.57 m

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

Divide the track load into 5 equal wheel load

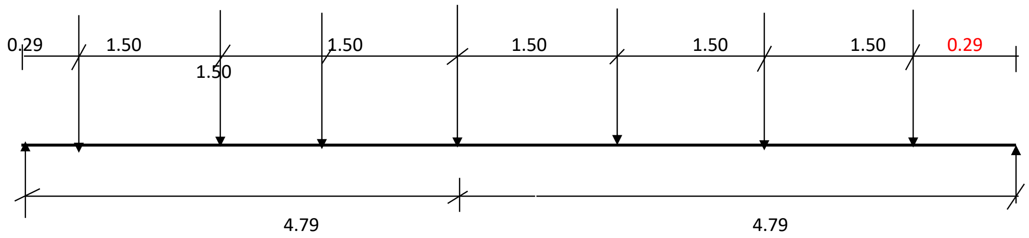
Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	2.96	4.60	0.97	5.57	7.63	29.91
140	3.88	5.18	0.97	6.15	8.21	36.34
140	4.79	5.38	0.97	6.35	8.41	43.86
140	3.88	5.18	0.97	6.15	8.21	36.34
140	2.96	4.60	0.97	5.57	7.63	29.91
					B . M =	**176.36**

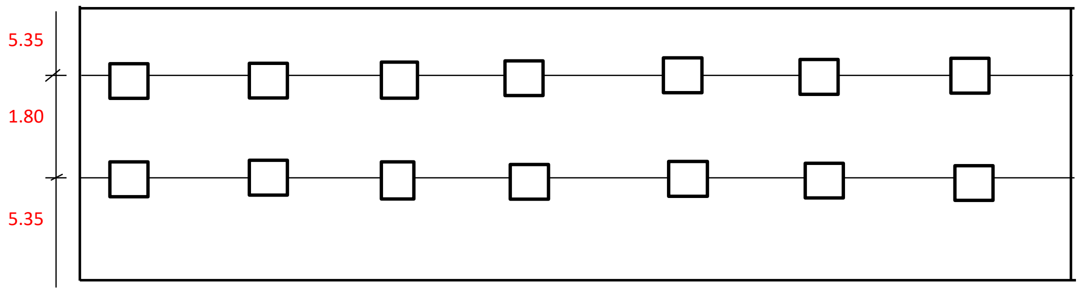
CLASS SV Loading

Impact Factor = 1.00

180 180 180 180 180 180 180

INPUTS FOR DESIGN OF RCC SOLID SLAB





Dimensions of 70RWheel Tyre presssure = 2.250 kg/cm^2 225.00 kN/m^2

Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	0.290	0.63	0.58	1.21	2.42	10.77
180	1.79	3.27	0.58	3.85	5.65	28.52
180	3.29	4.85	0.58	5.43	7.23	40.95
180	4.79	5.38	0.58	5.96	5.96	72.35
180	3.29	4.85	0.58	5.43	5.43	54.52
180	1.79	3.27	0.58	3.85	5.65	28.52
180	0.29	0.63	0.58	1.21	2.42	10.77
					B . M =	**246.40**

CWLL Bending Moment For

1.) CLASS A - 3 Lane	=	157.89	kN-m/m
2.) CLASS 70 R Wheel	=	166.60	kN-m/m
3.) CLASS 70 R Track	=	176.36	kN-m/m
4.) SV Loading	=	246.40	kN-m/m

INPUTS FOR DESIGN OF RCC SOLID SLAB

Total Bending Moment at Mid Span

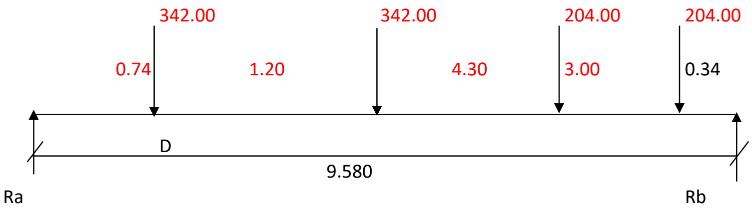
Loads	Unfactored B.M. (kNm/m)	ULS Factor	ULS Moment (kNm/m)	Rare Combination factor	Rare Combination Moment (kNm/m)	Quasi permanent combination Factor	Quasi permanent combination Moment (kNm/m)
DL	229.44	1.35	309.75	1.00	229.44	1.00	229.44
SIDL1	14.68	1.35	19.82	1.00	14.68	1.00	14.68
SIDL2	22.94	1.75	40.15	1.00	22.94	1.00	22.94
FPLL	-0.25	1.50	-0.38	1.00	-0.25	0.00	0.00
CWLL - (Class A /70R)	176.36	1.50	264.55	1.00	176.36	0.00	0.00
CWLL - SV Load	246.40	1.00	246.40	1.00	246.40	0.00	0.00
Total Moment		=	**633.89**		**513.21**		**267.07**

3.5 CWLL Shear Force at a distance of Effective Depth from the centre of support

Thus Shear force at Point D = 68.21 kN (Including Impact Factor)

3- Lane Class A Load

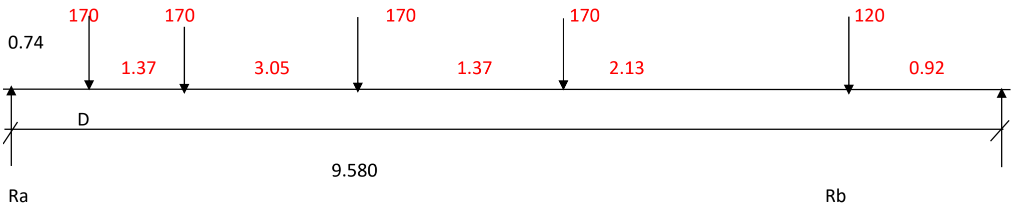
Effective Depth = 0.74 m
Distance from Support = 0.74 m



Axle Load	a (m)	α*a (1 - a / L)	b1 (m)	beff	beff for all axles	Shear Force (kN/m)
204.00	0.34	0.74	0.51	1.25	7.48	1.25
204.00	3.34	4.89	0.51	5.40	12.30	7.45
342.00	1.94	3.47	0.63	4.10	11.65	30.17
342.00	0.74	1.53	0.63	2.16	10.68	38.08
						76.95

Thus Shear force at Point D = 76.95 kN (Including Impact Factor)

1- 70 R Wheeled Load



Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Shear Force (kN/m)
120	0.92	1.87	0.99	2.86	4.79	3.01
170	3.05	4.67	0.99	5.66	7.59	8.91
170	4.42	5.35	0.99	6.34	8.27	11.86
170	2.11	3.69	0.99	4.68	6.61	25.05
170	0.74	1.53	0.99	2.52	4.45	44.03
						92.86

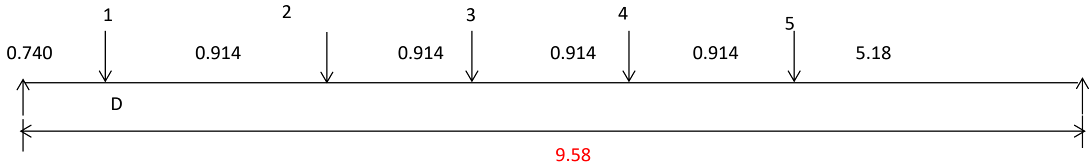
Thus Shear force at Point D = 92.86 kN (70 R Wheeled Load)

Governing Shear Force at Deff = 92.86 kN

INPUTS FOR DESIGN OF RCC SOLID SLAB

CLASS 70 R(T) Loading

Impact Factor = 1.10



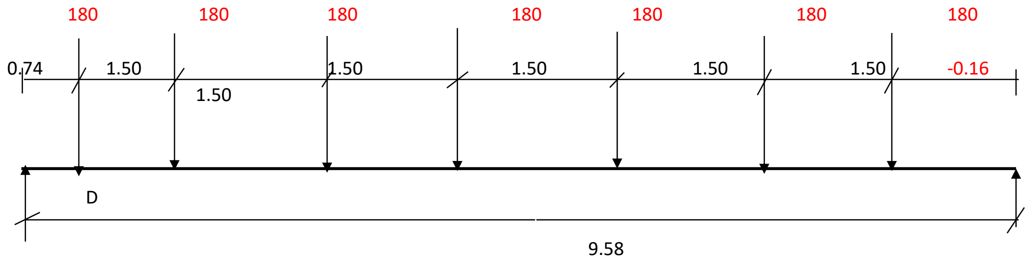
Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
140.00	0.840	0.914

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
140	0.74	1.53	0.97	2.50	4.56	18.26
140	1.65	3.07	0.97	4.04	6.10	16.06
140	2.57	4.22	0.97	5.19	7.25	15.54
140	3.48	4.98	0.97	5.95	8.01	15.91
140	4.40	5.34	0.97	6.31	8.37	16.97
					S.F =	**82.75**

Thus Shear force at Point D = 82.75 kN (70 R Track Load)

CLASS SV Loading

Effective Depth = 0.74 m
Distance from Support = 0.74 m



Axle Load Class A	Breadth of tyre across span W	Breadth of tyre along span B
180.00	0.450	0.274

Axle Load	a	α*a (1 - a / L)	b1	beff	beff for all axles	Bending Moment (kNm/m)
180	-0.160	-0.37	0.58	0.21	0.43	-7.00
180	1.34	2.59	0.58	3.17	4.97	5.07
180	2.84	4.49	0.58	5.07	6.87	7.77
180	4.34	5.33	0.58	5.91	7.71	10.57
180	3.74	5.12	0.58	5.70	7.50	14.63
180	2.24	3.85	0.58	4.43	6.23	22.12
180	0.74	1.53	0.58	2.11	3.91	42.44
					S.F =	**95.60**

Thus Shear force at Point D = 95.60 kN (SV Load)

INPUTS FOR DESIGN OF RCC SOLID SLAB

CWLL Shear Force for

1.) CLASS A - 3 Lane	=	76.95	kN/m
2.) CLASS 70 R Wheel	=	92.86	kN/m
3.) CLASS 70 R Track	=	82.75	kN/m
4.) SV Loading	=	95.60	kN/m

Summary of Shear Force

Loads	S.F. At deff (kN/m)	ULS Factor	ULS-S.F. (kN/m)
DL	81.20	1.35	109.62
SIDL1	5.20	1.35	7.02
SIDL2	8.12	1.75	14.21
FPLL	-1.13	1.50	-1.69
CWLL - (Class A	92.86	1.50	139.29
CWLL - SV Load	95.60	1.00	95.60
Total			**268.45**

ULS DESIGN OF SOLID SLAB

DECK SLAB FOR ULS FLEXURAL MOMENT

Min. Thickness of slab	=	**800**	mm
Clear Cover to outer steeel	=	**50**	mm
Maximum Diameter of Reinformcement	=	**20**	mm
Effective Depth Provided (deff)	=	740	mm
Ultimate Design bending moment	=	**633.89**	kNm/m

Mulim = 0.167 x fck x b x d^2 = 633.89 kNm/m (Equation derived based on IRC:112-2011)

Effective Depth of Cap Required (dreq) = SQRT($\frac{633.89 \times 1000000}{0.167 \times 30.00 \times 1000}$)

Effective Depth of Cap Required (dreq)	=	355.702	mm	
Total Depth Required (Dreq)	=	415.70	mm	
Total Depth Provided (Dprov)	=	800.00	mm	**OK**

$R = M_{RD}/(b\ d^2)$ = 1.16

Ast Required:

$\frac{pt}{100} = \frac{Ast_{req}}{b\ d} = \frac{fck \{ 1 - \mathrm{sqrt}(1- 4.598\ R/fck) \}}{2\ fy}$

= 0.003

Ast_{req} = 2065.407 mm^2/m

Minimum Longitudinal Reinforcement :

As. Min = 0.26 x $\frac{fctm}{fyk}$ x b . d - Refer Eq. 16.5.1.1 & 16.6.1.1 of IRC: 112-2011

Whichever is higher OR = 0.0015 x b . d -Refer Clause 16.9 of IRC:112-2011'

	b	=	1000.00	mm
	d	=	740.00	mm
	Ast min	=	**1110.00**	mm^2/m
Governing Reinf. Ast		=	**2065.41**	mm^2/m

Provide **20** mm dia @ **200** mm c/c + **20** mm dia @ **190** mm c/c

Area provided= 3224.27 mm^2/m > 2065.41 mm^2/m OK

Percentage of Steel (pt%) = 0.44 %

Maximum Spacing of Bars : as per Clause 16.6.1.1 of IRC:112-2011

Smax = 2 h = 1480.00

OR = 250.00 mm whichever is max

Provided Spacing is less than Smax, Hence OK

Limiting Depth of Neutral Axis , Xm = $\frac{0.0035\ .\ d}{(0.0035 + fyd/ Es)} = \frac{0.0035 \times 740.00}{0.0035 + 0.0022}$

= 456.48 mm

Depth of Neutral Axis , X = $\frac{fyd\ .\ Ast}{0.36\ .\ fck\ .\ b} = \frac{434.78 \times 3224.27}{0.36 \times 30.00 \times 1000}$

= 129.80 mm **UNDER REINFORCED DESIGN, OK**

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

z = d - 0.416 . X = 740.00 - 54.00

= 686.00 mm

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

MR = fyd . Ast . z

= 434.78 x 3224.27 x 686.00

= 9.62E+08 Nmm / m

= 961.68 kNm / m > 633.89 kNm / m SAFE

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

ULS DESIGN OF SOLID SLAB

Distribution reinforcement:

As per Clause 16.6.1.1. of IRC:112-2011 , Secondary Reinforcement shall be at least 20 % of the main reinforcement

$\frac{20.00}{100.00}$ x 3224.27 = 644.853 mm^2/m

Provide	12 dia bar	150 direction in top face. (Providing =	753.98 mm^2)	OK

DESIGN OF SLAB FOR ULS SHEAR

Ultimate Design Shear Force = **268.45 kN/m**

Design Shear Strength of Concrete,(t_c) without Shear Reinforcement:

As per Clause 10.3.2 of IRC:112-2011,

Design shear resistance of the member without shear reinforcement is given by:

$$V_{Rd,c} = \left[0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}\right]b_w d \quad \textbf{eq.1}$$

Subjected to minimum of $$V_{Rd,c} = (V_{min} + 0.15\sigma_{cp})b_w d \quad \textbf{eq.2}$$

where,

K = 1 + SQRT(200/d) ≤ 2.0 , where d is depth in mm

K = 1.52

vmin = 0.031 $K^{3/2}$ $fck^{1/2}$, fck = 30.00 N/mm^2

Hence **vmin= 0.318 N/mm^2**

σ_{cp} = Concrete compressive stress in concrete at centroidal axis in the direction of axial load or prestressing

σ_{cp} = N_{Ed}/A_c < 0.2 f_{cd} where , f_{cd} = 0.67 fck/1.5

σ_{cp} = 0.00 N/mm^2

Hence,

τ_c = $V_{Rd,c}/(b_w.d)$ = V_{min} + 0.15 σ_{cp} = 0.3182 N/mm^2 **From eq.2**

ρ1 = Steel Ratio = Asl/(bw . d) ≤ 0.02

Hence ρ1 = 0.0044

τ_c = $V_{Rd,c}/(b_w.d)$ = 0.399 N/mm^2 **From eq.1**

Max of eq.1 & eq.2

τ_c = $V_{Rd,c}/(b_w.d)$ = 0.399 N/mm^2 Corresponds to steel ratio = 0.436% & M30 Grade of Concrete

Shear stress(v_{Ed}) = V_{Ed}/(bw*d)

v_{Ed} = $\frac{268450.58}{1000.00 \times 740.00}$ = 0.363 N/mm^2 < 0.399 MPa

AsTv is lesser than Tc Hence No Shear Reinforcement is need to be provided.

CHECK FOR PUNCHING AROUND VEHICLE LOAD (as per IRC:112-2011, Clause 10.4.3):

Maximum load on each wheel = 100 KN (70 RW Boggie load)

Maximum tyre pressure = 5.273 kg/cm^3

Contact width perpendicular to span, L = 0.86 m

Contact width parallel to span,B = 0.190 m

Basic Equation for Punching shear stress(v_{Ed}) = $\frac{\beta V_{Ed,req}}{ui * d}$

Depth of Slab , d = 740.00 mm

Length of perimeter, ui = pi() * 4 d + L * 2 + B * 2 for Rectangular section = 11398.40 mm

Loaded area under perimeter = pi() * (4 d)^2/4 + L * 2d*2 + B * 2d*2 + L * B = 1.02E+07 mm2

ΔVEd = 0.00 for deck slab

$V_{Ed,req}$ = V_{Ed} - ΔV_{Ed} = 100.00 kN.

β = 1.00 for axial load without bending

v_{Ed} = 0.01 N/mm^2

ULS DESIGN OF SOLID SLAB

Governing Punching Shear Resistance of Concrete VRd,c = As per IRC:112-2011, Clause 10.4.4

$$v_{Rd.c} = \frac{0.18}{\gamma_c} K\left(80\rho_1 f_{ck}\right)^{1/3} + \left(0.1\sigma_{cp}\right) \geq v_{\min} + 0.1\sigma_{cp}$$

where,						
K	=	1.52				
ρ1	=	0.0044				
fck	=	30.00	N/mm^2			
σ_{cp}	=	0.00	N/mm^2			
γc	=	1.50				
Vmin	=	$0.031*k^{3/2}*fck^{1/2}$		=	0.318	N/mm^2
Vmin + 0.1 σcp	=	0.318				
$v_{Rd,c}$	**=**	**0.399**	**N/mm^2**	**OK**		

SINCE	**v_{Ed}**	<	**$v_{Rd,c}$**	**HENCE SAFE IN PUNCHING**

SLS DESIGN OF DECK SLAB

1. Early Age Check (before creep has taken place)
2.Long Term Check (after creep has taken place)

Maximum compressive stress in concrete under rare combination	0.48 . fck	=	14.4	N/mm^2
Maximum tensile stress in steel under rare combination		=	300	N/mm^2
Maximum Tensile stress in concrete	fctm	=	2.50	N/mm^2
Permissible crack width		=	0.3	mm

Section Properties of Uncracked Section:

Width of Section , b	=	1000.00	mm
Depth of Section , D	=	800.00	mm
Gross Cross sectional area , Ag	=	800000	mm^2
Gross Moment of Inertia Igxx	=	4.27E+10	mm^4
Gross Moment of Inertia Igyy	=	6.67E+10	mm^4

Modular Ratio : for Early Age Check

Modulus of elasticity of concrete, Eceff = Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2
Modular Ratio (m) = Es/ Eceff	=	6.41	

Modular Ratio: for Long Term Check

Perimeter of section (u)	=	3600	mm
2 Ac/u	=	444.444	mm
Modulus of elasticity of concrete, Ecm	=	31187	N/mm^2
Modulus of elasticity of steel, Es	=	200000	N/mm^2

For Moist atmosperic condition

Creep coeff. ø = 1.30

$$Eceff = \frac{Ecm}{(1 + ø)} = 1.35E+04$$

Modular Ratio (m) = Es/ Eceff = **14.77**

STRESS CHECK IN CRACKED SECTION

Depth of neutral axis =

$$dc = \frac{-A_s E_s + \sqrt{(A_s E_s)^2 + 2bA_s E_s E_{c,eff} d}}{bE_{c,eff}}$$

Cracked Second Moment of Area I_{NA} (in steel units) $= A_s(d-d_c)^2 + \frac{1}{3}\frac{E_{c,eff}}{E_s} bd_c^{\,3}$

		Rare Case		Quasi-Permanent Case	
		For Early Age	For Long Term Check		
	unit	Sagging Moment	Sagging Moment	Sagging Moment	Remarks
SLS Moment	**kNm/m**	**513.212**	**513.212**	**267.069**	
Width of Section, b	mm	1000.00	1000.00	1000.00	
Depth of section , D	mm	800.00	800.00	800.00	
Effective cover , C_{eff}	mm	60.00	60.00	60.00	
Effective depth , d	mm	740.00	740.00	740.00	
Eceff	N/mm^2	31186.57	13537.58	31186.57	
Es	N/mm^2	200000.00	200000.00	200000.00	
Flexural Ast Provided , As	mm^2/m	3224.27	3224.27	3224.27	
dc		155.48	222.12	155.48	
Cracked Second Moment of area , I_{NA}	mm^4	1296976443.04	1112006587.06	1296976443.04	
section modulus, Zt = I_{NA} / dc	mm^3	8341981.95	5006315.33	8341981.95	
section modulus, Zb = I_{NA} / (d-dc)	mm^3	2218858.48	2147231.46	2218858.48	
Maximum compressive stress in concrete= M/Zt x	N/mm2	9.59	6.94	4.99	**< 14.4 SAFE**
Maximum Tensile stress in steel = M/Zb	N/mm2	231.30	239.01	120.36	**< 300 SAFE**

SLS DESIGN OF DECK SLAB

CRACK WIDTH CHECK Refer Clause 12.3.4 of IRC:112-2011

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

Where, Sr max = maximum crack spacing
εsm = mean strain in the reinforcement
εcm = mean strain in concrete between cracks

Spacing between reinf. = 5*(c+ϕ/2)	mm	300	
Spacing provided	mm	100	
Check for spacing criteria		OK	
$S_{r\,max} =$	3.4 c	+	$\frac{0.425\ k1\ k2\ \phi}{\rho_{\rho\,eff}}$
Clear Cover , c	mm	50	
Diameter of Main Bar , ϕ	mm	20	
Coefficient , k_1		0.8	
Coefficient , k_2		0.5	
Width of section , b	mm	1000	
Depth of section , D	mm	800	
Effective Depth of Section , d	mm	740	
Depth of Neutral axis , yt=dc	mm	155.476	
hc eff = Min of 2.5 (D - d) ,D - dc/3 , D/2	mm	150.00	
Ac, eff = b * hc,eff	mm^2	150000	
$\rho_{\rho\,eff} = As/A_{c\,eff}$		0.021	
Sr max	mm	328.176	
$(\varepsilon_{sm} - \varepsilon_{cm})$	=	$\frac{\sigma_{sc} - \frac{k_t f_{ct\,eff}(1 + \alpha_e \rho_{\rho\,eff})}{\rho_{\rho\,eff}}}{E_s}$	
Stress in tension steel , σsc (in Quasi-Permanent Case)	**N/mm2**	**120.36**	
Kt		0.500	
Tensile strength of concrete = fct eff = fctm	N/mm2	2.501	
a_e = Es/ Ecm		6.413	
$(\varepsilon_{sm} - \varepsilon_{cm})$		0.000	
Crack Width , Wk	**mm**	**0.09**	
Limited Carck width	**mm**	**0.30**	
Check for Crack width		**< 0.3 mm SAFE**	