DEPARTMENT OF ROADS

Ministry of Public Works and Transport, LAOS

DESIGN OF PIER SUBSTRUCTURE AND FOUNDATION FOR NAM THONE BRIDGE AT 239+092

(PKG-3)

DOCUMENT NO:-80073/LASA/NTN/DN/SUB-03

National Road 13 South: Implementation Support and Work Supervision (ISWS) Consultancy Services - Detailed Design for Upgrading of 14 Bridges (Additional Task Under Amendment 2 to the Consultant Contract) LAO PDR

NOVEMBER 2023

Revision-R0

LEA ASSOCIATES SOUTH ASIA PVT. LTD., *India*
In Joint Venture
PYUNGHWA ENGINEERING CONSULTANTS, *Korea*
in association (as sub-consultant) with
LAO TRANSPORT ENGINEERING CONSULTANT, *Lao PDR*

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	LEA Associates South Asia Pvt. Ltd.	Date:	08-11-2023	Note No.
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DESIGN OF PIER SUBSTRUCTURE & PILE FOUNDATION

This design note pertains to the structural design of Pile, Pile Cap & Pier of Pier P1 & P2 for Nam Thone Bridge at Ch. 239+092

•The length of pile and capacity of pile has been considered as per the Geotechnical Report. Soil properties has been considered as per the Geotech Report.

●For Calculating the Bending Moment in a pile corresponding to unit lateral force, a single pile is idealized in STAAD. The pile is restrained by spring support along the length of pile representing soil stiffness with appropriate value of soil subgrade reaction g_h depending on the SPT Value.The average SPT values are taken from geotechnical report. Also the top of pile cap is considered as fixed against rotation.

●Pile Cap has been designed as a Bending Member. Pile Cap has been checked for oneway shear and punching shear also.

●Pier has been designed for combined axial and biaxial bendings as per codal provision of AASHTO LRFD.

●Pier Cap has been designed as bending member and checked for maximum bending moment, torsional moment and shear.Pier cap has been checked either as a flexural member or as a bracket, depending upon the span / depth ratio. In case it is a flexural member, the bending moments are checked at the face of pier support. Shear force has been checked at a distance deff away from the face of support.

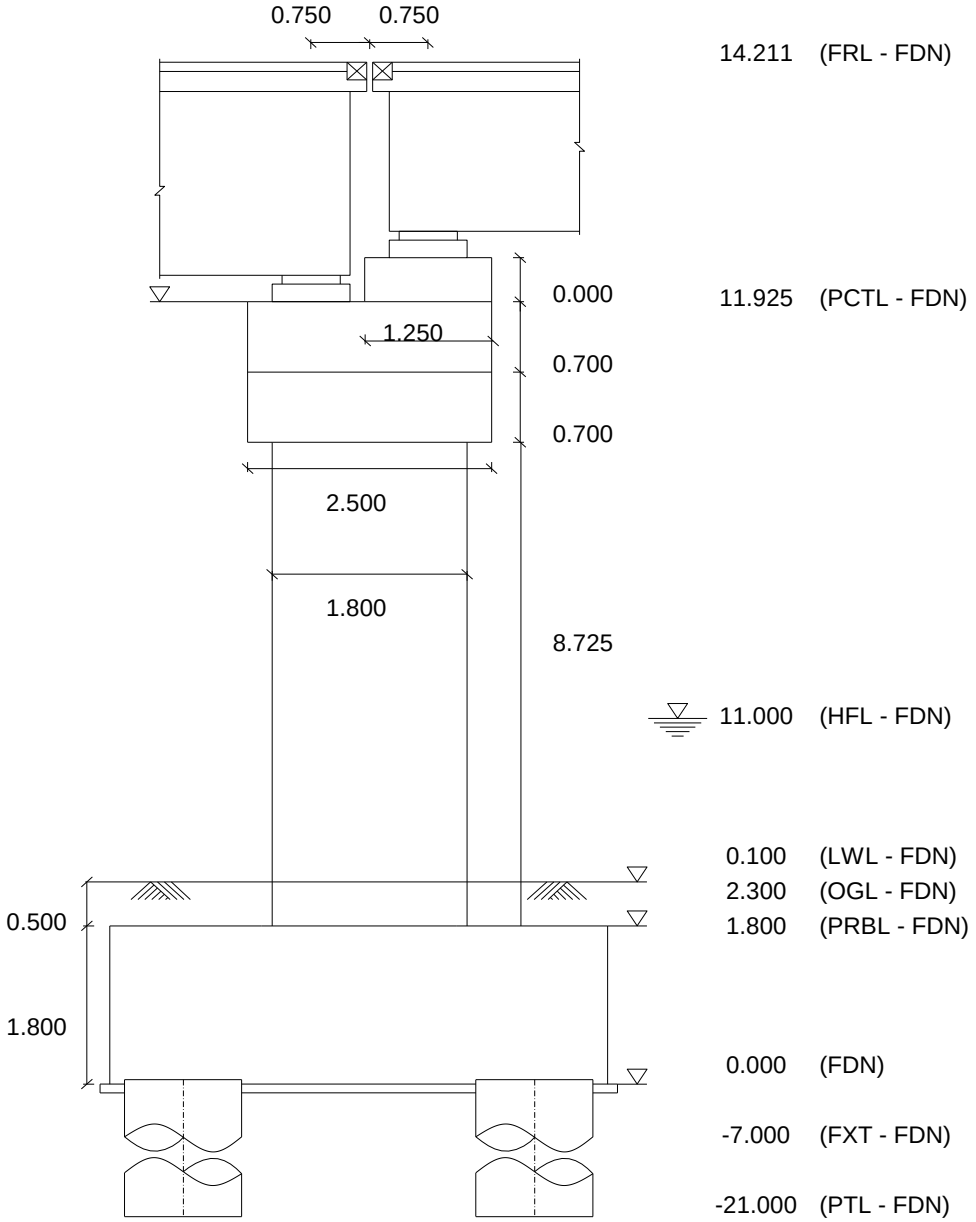
Pile data considered for design of pile foundation are as follows.

Length of Pile	=	**25m**
Vertical Pile Capacity	=	**3070 KN**
Horizontal Pile Capacity	=	**120 KN**

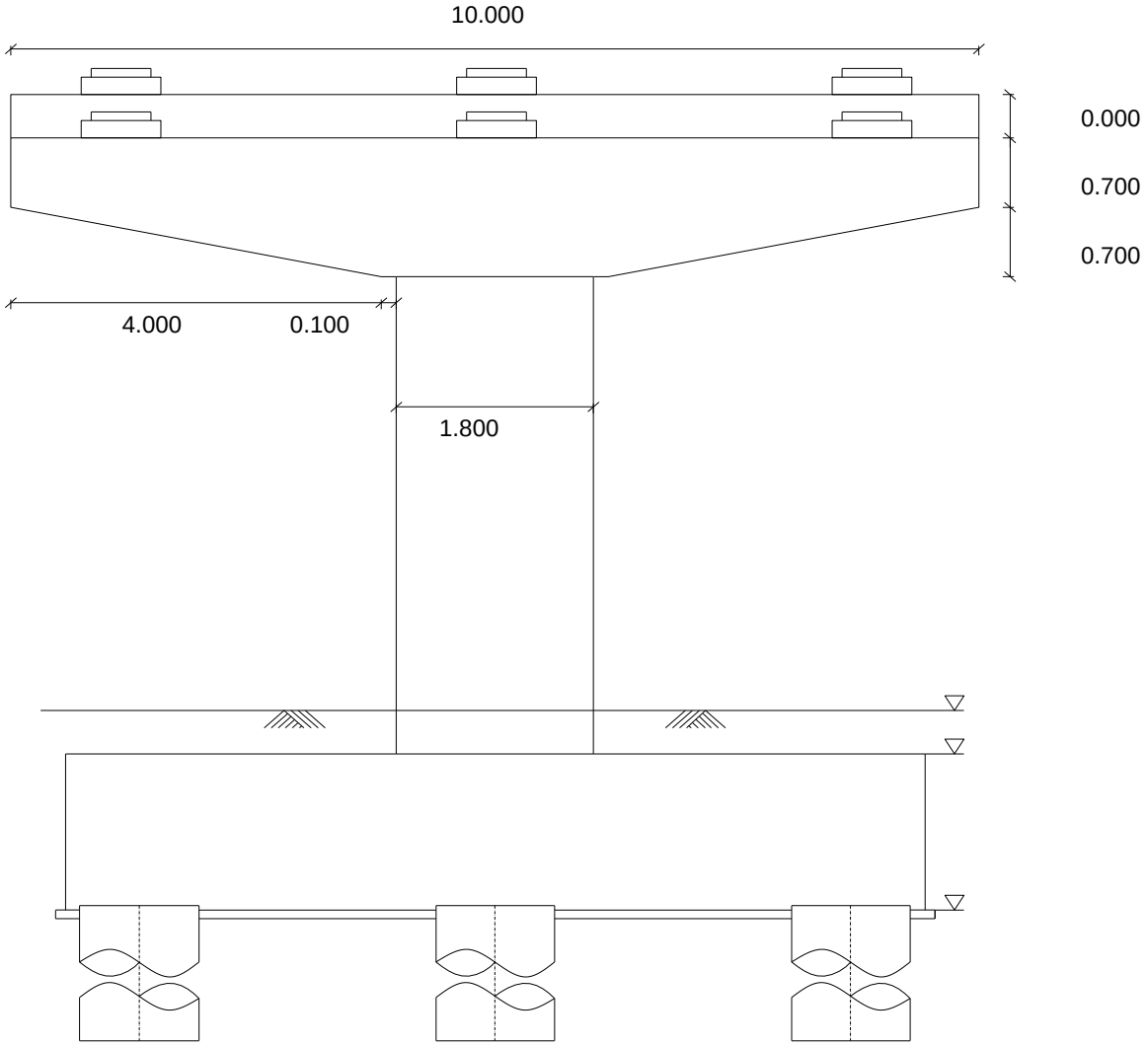
BASIC DESIGN INPUT

		Left Span	Right Span	
C/C to distance between Exp. Joint	=	26	26.4	m
Dist. between C/L's of Brg. & Exp. Jt.	=	0.75	0.75	m
C/C distance between Bearing	=	24.5	24.9	m
Deck Width	=	10.8	10.8	m
Type Of superstructure	=	PSC I	PSC I	
No. of Girder	=	4	4	Nos.
C/C distanace of bearing in transverse direction	=	2.75	2.75	m
Cant. Projection from C/L of bearing in transverse direction	=	1.275	1.275	m
Depth of Superstructure including Deck slab	=	1.77	1.77	m
Thk. of Wearing Coat	=	0.065	0.065	m
Cross Slope	=	3.0%	3.0%	
Bearing type	=	Elastomer	Elastomer	
Unit weight of concrete	=	25	KN/m^3	
Unit weight of concrete	=	20	KN/m^3	
Unit Weight Of Water	=	10	KN/m^3	

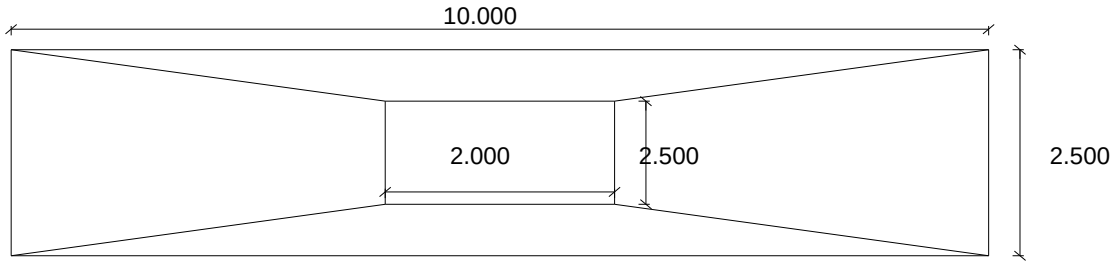
Formation Level, FRL	=	14.211	m
Brearing Top Level	=		
For Span 1	=	12.225	m
For Span 2	=	12.225	m
Minimum Height of Pedestal + Bearing	=		
For Span 1	=	0.300	m
For Span 2	=	0.300	m
Pier Cap Top Level	=	11.925	m
Highest Flood Level, HFL	=	11.000	m
Low Water Level, LWL	=	0.100	m
Max. Scour Level, MSL	=	-4.000	m
Original Ground Level, OGL	=	2.300	m
Foundation Level, FDN	=	0.000	m
Fixity Level, FXT	=	-7.000	m
Pile Termination Level, PTL	=	-21.000	m



LONGITUDINAL ELEVATION (Pile Foundation as shown is indicative only)



TRANSVERSE ELEVATION (No of Pedestal shown is indicative only)



PLAN OF PIER CAP

LOAD CALCULATION FOR SUBSTRUCTURE & FOUNDATION

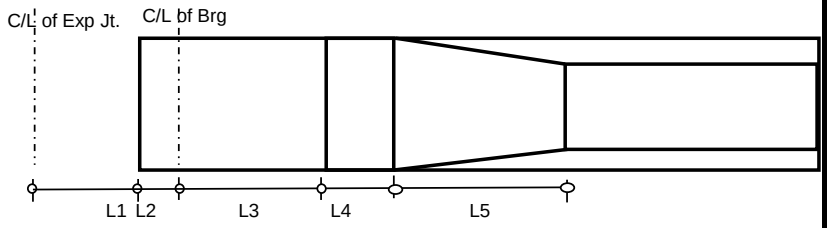
Pedestal			
Number of Pedestal (Both Span)	=	10	
Average Height of Pedestal	=	0.250	m
Size of Pedestal	=		
Length	=	0.750	m
Width	=	0.750	m
Volume of Pedestal	=	0.141	m^3
Weight of Pedestal	=	35.156	KN
Seismic Arrester	=		
Number of Seismic Arrester (Both Span)	=	4	
Average Height of Seismic Arrester	=	0.500	m
Size of Seismic Arrester	=		
Length	=	0.400	m
Width	=	0.750	m
Volume of Seismic Arrester	=	0.150	m^3
Weight of Seismic Arrester	=	15.000	KN
Pier Cap Beam	=		
Volume of Pier Cap Beam	=	0.000	m^3
Weight of Pier Cap Beam	=	0.000	KN
Pier Cap	=		
Volume of Pier Cap (Constant Part)	=	17.500	m^3
Volume of Pier Cap (Varying Part)	=	10.500	m^3
Weight of Pier Cap (Constant + Varying)	=	700.000	KN
Pier			
Number of Pier	=	1	
Cross Section of Pier Shaft	=	Circular	
Cross Sectional Area of Pier	=	2.545	m^2
Height of Pier Shaft	=	8.725	m
Volume of Pier Shaft	=	22.203	m^3
Weight of Pier	=	555.076	KN
Pile Cap	=		
Length of Pile Cap	=	5.300	m
Width of Pile Cap	=	5.300	m
Depth of Pilecap	=	1.800	m
Weight of Pile Cap	=	1264.1	KN
Soil above Foundation			
Depth of Soil above Pilecap	=	0.500	m
Weight of Soil above Pilecap	=	255.453	KN
Buoyancy			
Up to Pier level			
Volume of water displaced	=	23.41	m^3
Weight to be deducted	=	234.1	KN
Up to Pile cap Bottom/ Fdn level			
Volume of water displaced	=	50.6	m^3
Weight to be deducted	=	505.6	KN

Substructure Components	Weight (kN)	CG w.r.t FDN (m)	Long. Ecc. w.r.t FDN (m)	Tran. Ecc. w.r.t FDN (m)
Pedestal	35.156		0.000	0.000
Seismic Arrester	15.000		0.000	0.000
Pier Cap Beam	0.000	11.925	0.750	0.000
Pier Cap (Constant Part)	437.500	11.575	0.000	0.000
Pier Cap (Varying Part)	262.500	10.953	0.000	0.000
Pier	555.076	6.163	0.000	0.000
Substructure	**1305.233**	**8.703**	**0.000**	**0.000**

Substructure Components	Weight (kN)	CG w.r.t Pier Base (m)	Long. Ecc. w.r.t Pier Base (m)	Tran. Ecc. w.r.t Pier Base (m)
Pedestal	35.156	-1.800	0.000	0.000
Seismic Arrester	15.000	-1.800	0.000	0.000
Pier Cap Beam	0.000	10.125	0.000	0.000
Pier Cap (Constant Part)	437.500	9.775	0.000	0.000
Pier Cap (Varying Part)	262.500	9.153	0.000	0.000
Pier	555.076	4.363	0.000	0.000
Substructure	**1305.233**	**6.903**	**0.000**	**0.000**

LOAD DUE TO SUPERSTRUCTURE

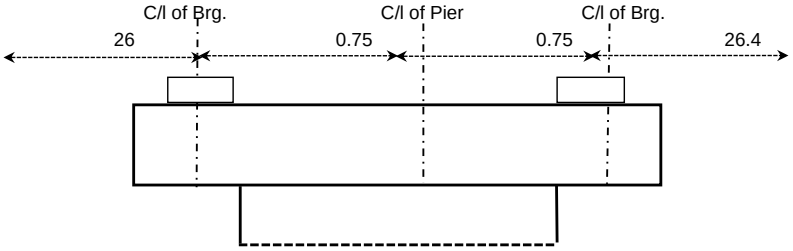
		Span1			Span2	
Total Width		= 10.800	m	= 10.800	m	
C/C spacing of Expansion joint		= 26.000	m	= 26.400	m	
Skew angle		= 0.000	Degree	0.000	Degree	
Depth of Superstructure(Excluding Deck Slab)		= 1.550	m	= 1.550	m	
Distance between C/L of Brg. and C/L of Exp. Joint	(Opp. End)	= 0.750	m	= 0.750	m	
Distance between C/L of Brg. and C/L of Exp. Joint	(Pier End)	= 0.750	m	= 0.750	m	
C/C spacing of Bearing		= 24.500	m	= 24.900	m	
Thickness of Deck Slab		= 0.220	m	= 0.220	m	
Thickness of Deck Slab At Support		= 0.350	m	= 0.350	m	
No. of Longitudinal Girders		= 4	no.	= 4	no.	
No. of Int Cross Girders		= 1	no.	= 1	no.	
C/C spacing of Longitudinal Girder		= 2.750	m	= 2.750	m	
Average Cantilever length		= 1.275	m	= 1.275	m	
Top Flange Width of each girder		= 1.000	m	= 1.000	m	
Length of Girder beyond Support		= 0.500	m	= 0.500	m	
Thickened Portion of Girder beyond Support		= 1.500	m	= 1.500	m	
Web Varying Thickness Portion of Girder		= 1.500	m	= 1.500	m	
Bulb Varying Thickness Portion of Girder		= 0.000	m	= 0.000		
Web thickness of Intermediate Cross Girder		= 0.300	m	= 0.300	m	
Web thickness of End Cross Girder		= 0.400	m	= 0.400	m	
Cross section area of girder at Span		= 0.726	m^2	= 0.726	m^2	
Cross section area of girder at Bulb Varying		= 1.204	m^2	= 1.204	m^2	
Cross section area of girder at Support		= 1.204	m^2	= 1.204	m^2	
Length of Int. Cross girder		= 9.00	m	= 9.00	m	
Length of End. Cross girder		9.00	m	= 9.00	m	
Density of Girder Concrete		= 25	kN/m^2	= 25	kN/m^2	
Density of Deck Slab Concrete		= 25	kN/m^2	= 25	kN/m^2	
Superimposed Dead Load (W/C)		= 20.90	kN/m	= 20.90	kN/m	
Superimposed Dead Load (C/B)		= 25.00	kN/m	= 25.00	kN/m	



	Span1		Span2	
L1 =	0.250		L1 =	0.250
L2 =	0.500		L2 =	0.500
L3 =	1.500		L3 =	1.500
L4 =	0.000		L4 =	0.000
L5 =	1.500		L5 =	1.500

Load of Individual Girder						
Span Portion	=	336	kN	=	343	kN
Web Varying Thickeness Portion of Girder	=	72	kN	=	72	kN
Bulb Varying Thickness Portion of Girder	=	0	kN	=	0	kN
Support Portion of Girder	=	120	kN	=	120	kN
Intermediate Cross Girder	=	72	kN	=	72	kN
End Cross Girder	=	153	kN	=	153	kN
Deck Slab Between Brg. To Brg.	=	1455	kN	=	1479	kN
Deck Slab Cantilever At Abutment Side	=	71	kN	=	71	kN
Deck Slab Cantilever At Pier Side	=	71	kN	=	71	kN
Total Load on Pier due to DL from one side span(2% extra)	=	**2007**	kN	=	**2034**	kN
Total Load on Pier due to W/C from one side span	=	**272**	kN	=	**276**	kN
Total Load on Pier due to C/B from one side span	=	**325**	kN	=	**330**	kN

LIVE LOAD CALCULATION



Depth of SuperStructuer Span 1 Side	=	1.770	m
Depth of SuperStructuer Span 2 Side	=	1.770	m
Thickness Of Wearing coat	=	0.065	m
Radius of Curvature	=	1000000	m
Speed of Vehicle (kmph)	=	100	Km/h
Coefficient of Friction for Bearing, μ	=	0.05	
Skew angle	=	0.000	Degree
Application of Braking/CF Force	=	1.8	m
Dynamic Load allowance Factor For Fdn.	=	1.33	
Dynamic Load allowance Factor For Sub.	=	1.33	
Multiple Presence Factor For			
1 Lane	=	1.2	
2 Lane	=	1	
3 Lane	=	0.85	
>3 Lane	=	0.65	

Maximum Reactuion at Support 'A' due to DL+SIDL	=	2604	KN
Maximum Reactuion at Support 'B' due to DL+SIDL	=	2604	KN
Maximum Reactuion at Support 'C' due to DL+SIDL	=	2640	KN
Maximum Reactuion at Support 'D' due to DL+SIDL	=	2640	KN
Overall width of Structure	=	10.8	m
Width of clear carraigeway	=	8.0	m
No.of design lane		2.00	Nos.

Maximum Longitudinal moment case

(The Vehicular loads are placed in such a way that CG of the load system is at the center line of pier to get the max reaction)

Description		Type of Live Load								
		1L Design Truck	1L Design Tandem	1L Lane Load	2L Design Truck	2L Design Tandem	2L Lane Load	3L Design Truck	3L Design Tandem	3L Lane Load
Total Live Load on Span (without DA)	KN	325	220	9.3xL	650	440	2x9.3xL	975	660	3x9.3xL
Live Load Reaction at End 'A'	KN	0.0	0	0	0	0	0	0	0	0
Live Load Reaction at End 'B'	KN	0.0	0	0	0	0	0	0	0	0
Live Load Reaction at End 'C'	KN	295.0	220.0	123.2	590	440	246.45	885	660	123.2
Live Load Reaction at End 'D'	KN	29.8	0.0	123.2	59.6	0	246.45	89.4	0	369.675
Total Reaction On Pier (without DA)	KN	295.0	220.0	123.2	590.0	440.0	246.5	885.0	660.0	123.2
Multiple presence Factor		1.20	1.20	1.20	1.00	1.00	1.00	0.85	0.85	0.85
Dynamic load Allowance		1.3	1.3	1.0	1.33	1.3	1.0	1.3	1.3	1.0
Design Support Reacton	KN	**471**	**351**	**148**	**785**	**585**	**246**	**1000**	**746**	**105**
Maximum Transverse Eccentricity	m	3.400	3.400	3.400	1.600	1.600	1.600	-0.200	-0.200	-0.200
Maximum Transverse Moment(M_T)	KN-m	**1601**	**1194**	**503**	**1256**	**936**	**394**	**-200**	**-149**	**-21**
Maximum Long. Eccentricity	m	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Maximum Long. Moment(M_L)	KN-m	**353**	**263**	**111**	**589**	**439**	**185**	**750**	**560**	**79**
Centrifugal force	KN	0.0	0.0	0.0	0.1	0.0	0.0	0.1	0.1	0.0

Summary of loads for Maximum Longitudinal moment case

SL.No.	Cases	P(KN)	M_L (KN-M)	M_T (KN-M)
1	Truck load+Lane Load (1 Lane)	618.7	464	2104
2	Tandem load+Lane Load (1 Lane)	499.0	374	1697
3	Truck load+Lane Load (2 Lane)	1031.2	773.4	1650
4	Tandem load+Lane Load (2 Lane)	831.7	623.7	1331
5	Truck load+Lane Load (3 Lane)	0.0	0.0	0
6	Tandem load+Lane Load (3 Lane)	0.0	0.0	0

Maximum Reaction case

(The Vehicular loads are placed in such a way that CG of the load system is at the center line of pier to get the max reaction)

Description		Type of Live Load								
		1L Design Truck	1L Design Tandem	1L Lane Load	2L Design Truck	Design Tandem	2L Lane Load	3L Design Truck	3L Design Tandem	3L Lane Load
Total Live Load on Span (without DA)	KN	325	220	9.3xL	650	440	2x9.3xL	975	660	3x9.3xL
Live Load Reaction at End 'A'	KN	1.8	-3.2	123.2	3.6	-6.4	246.4	5.4	-9.6	369.6
Live Load Reaction at End 'B'	KN	179.0	113.2	123.2	358	226.4	246.4	537	339.6	369.6
Live Load Reaction at End 'C'	KN	125.0	108.9	123.2	250	217.8	246.4	375	326.7	369.6
Live Load Reaction at End 'D'	KN	20.0	1.1	123.2	40	2.2	246.4	60	3.3	369.6
Total Reaction On Pier (without DA)	KN	304.0	222.1	246.4	608.0	444.2	492.8	912.0	666.3	739.2
Multiple presence Factor		1.20	1.20	1.20	1.00	1.00	1.00	0.85	0.85	0.85
Dynamic load Allowance		1.33	1.33	1.00	1.33	1.33	1.00	1.33	1.33	1.00
Design Support Reacton	KN	**485**	**354**	**296**	**809**	**591**	**493**	**1031**	**753**	**628**
Maximum Transverse Eccentricity	m	3.400	3.400	3.400	1.600	1.600	1.600	-0.200	-0.200	-0.200
Maximum Transverse Moment(M_T)	KN-m	**1650**	**1205**	**1005**	**1294**	**945**	**788**	**-206**	**-151**	**-126**
Maximum Long. Eccentricity	m	-0.13	-0.01	0.00	-0.13	-0.01	0.00	-0.13	-0.01	0.00
Maximum Long. Moment(M_L)	KN-m	**-65**	**-5**	**0**	**-108**	**-9**	**0**	**-137**	**-11**	**0**
Centrifugal force	KN	0.0	0.0	0.0	0.1	0.0	0.0	0.1	0.1	0.0

Summary of loads for Maximum Reaction case

SL.No.	Cases	P(KN)	M_L (KN-M)	M_T (KN-M)
1	Truck load+Lane Load (1 Lane)	780.9	-65	2655
2	Tandem load+Lane Load (1 Lane)	650.2	-5	2211
3	Truck load+Lane Load (2 Lane)	1301.4	-107.7	2082
4	Tandem load+Lane Load (2 Lane)	1083.6	-8.6	1734
5	Truck load+Lane Load (3 Lane)	0.0	0.0	0
6	Tandem load+Lane Load (3 Lane)	0.0	0.0	0

Summary of Live Load for Different case

SL.No.	Cases	LL CASE	P(KN)	M_L (KN-M)	M_T (KN-M)
1	Maximum Recation case	Truck load+Lane Load (2 Lane)	1301.4	-108	2082
2	Maximum Long. Moment case	Truck load+Lane Load (2 Lane)	1031.2	773	1650
3	Maximum Trans. Moment case	Truck load+Lane Load (1 Lane)	780.9	-64.6	2655

Braking force

(As Per Cl. 3.6.4 Of AASHTO LRDF)

Cases	1 Deisgn Lane	2 Deisgn Lane	3 Deisgn Lane	
25% Of design Truck Load	81	163	244	KN
25% Of design Tandem Load	55	110	165	KN
5% of design Truck+Lane Load	28	57	85	KN
5% of design Tandem+Lane Load	23	46	69	KN
Governing Load (with MF)	**98**	**163**	**0**	KN
No of Bearing	8	8	8	Nos.
Force per Bearing	12	20	0	KN

Loads due to Temperature force

Thermal coefficient ε = 0.0000108 mm/mm/°C

Girder setting Temperature t_s = 20 °C

Climate Condition = Moderate

Girder type = Concrete

L_{span} = 26000 mm

Expansion Calculation

$$\Delta_{exp}=\varepsilon \Delta t\ L_{span}$$

Δt Rise = 40 - ts

= 20 °C

Δ_{exp} = 5.62 mm

Contraction Calculation

Δt Fall = ts"- 0

= 20 °C

Δ_{exp} = 5.62 mm

Longitudinal Contraction due to shinkage = 0 mm

Longitudinal Contraction due to Creep = 0.0 mm

Design Translaton Δ_{exp} = 5.62 mm

Size of the one Elastomeric Bearing

L	B	H_e
350.00	400.00	75.00

Shear Modulus of Bearing (G) = 0.9 Mpa

Longitudinal Force per Bearing H =GA Δ_{exp}/H_e = 9.435 KN

Total Force on Pier = **37.7** KN

WIND LOAD CALCULATION

				With LL	With Out LL	
Basic Wind Speed V_B				160	160	Km/hr
Basic Wind Speed at the Location				90	120	Km/hr
Factor				0.32	0.56	
Height of bed level To Fdn				2.3	2.3	m
Wind load on Superstructure Span 1						
Length of Span			=	26.00	26.00	m
Width of Span			=	10.8	10.80	m
Depth of Superstructure			=	1.77	1.77	m
Height of Railing/CB			=	1.1	1.1	m
Span/Width Ratio			=	2.4	2.4	
Span/Depth Ratio			=	14.7	14.7	
Aeroelastic Instability Investigation			=	**NO**	**NO**	
Tributary length for transverse direction			=	13	13	m
Tributary length for Longitudinal direction			=	13	13	m
Height of superstructure top from Bed level/HFL		H1	=	11.911	11.911	m
Height of superstructure top from Bed level/HFL		H2	=	10.14	10.141	m
Height of CB/Railing top from Bed level/HFL		H3	=	13.011	13.011	m
Terrain Clasification			=	Open Country	Open Country	
Friction Velocity V_0			=	13.2	13.2	Km/hr
Friction Length Zo			=	70	70	mm
Design Wind Velocity at design elevation	$V_{DZ}=2.5\ V_0(V_{10}/V_b)\ln(Z/Z_0)$	H1	=	169.51	169.5	Km/hr
		H2	=	164.2	164.2	Km/hr
		H3	=	172.43	172.4	Km/hr
Base Wind pressure Correspondiing to $V_B(P_B)$			=	0.0024	0.0024	Mpa
Design Wind pressure p_D	$P_D=P_B(V_{DZ}/V_B)^2$	H1	=	0.0027	0.0027	Mpa
		H2	=	0.0025	0.0025	Mpa
		H3	=	0.0028	0.0028	Mpa
Transverse wind Loading in superstructure				4.62	4.62	KN/m
Transverse wind Loading in CB/railing				3.01	3.01	KN/m
Total Transverse wind				**7.64**	**7.64**	KN/m
				OK	**OK**	

Summary of Wind Load On Superstructure Span1

	WITH LL					WITH OUT LL					
Wind attack angle	**0**	**15**	**30**	**45**	**60**	**0**	**15**	**30**	**45**	**60**	
P_B Trans	0.0024	0.0021	0.0020	0.0016	0.0008	0.0024	0.0021	0.0020	0.0016	0.0008	MPa
P_B Long	0.0000	0.0003	0.0006	0.0008	0.0009	0.0000	0.0003	0.0006	0.0008	0.0009	MPa
P_D at H1 Trans	0.0027	0.0024	0.0022	0.0018	0.0009	0.0027	0.0024	0.0022	0.0018	0.0009	MPa
P_D at H2 Trans	0.0025	0.0022	0.0021	0.0017	0.0008	0.0025	0.0022	0.0021	0.0017	0.0008	MPa
P_D at H3 Trans	0.0028	0.0024	0.0023	0.0019	0.0009	0.0028	0.0024	0.0023	0.0019	0.0009	MPa
P_D at H1 Long	0.000	0.0003	0.0007	0.0009	0.0010	0.000	0.0003	0.0007	0.0009	0.0010	MPa
P_D at H2 Long	0.000	0.0003	0.0006	0.0008	0.0009	0.000	0.0003	0.0006	0.0008	0.0009	MPa
P_D at H3 Long	0.000	0.0003	0.0007	0.0009	0.0010	0.000	0.0003	0.0007	0.0009	0.0010	MPa
Long. wind Load at Sup.	0.00	2.38	4.75	6.34	7.13	0.00	4.22	8.45	11.26	12.67	KN
Long wind Load at CB/Railing	0.00	1.55	3.10	4.13	4.65	0.00	2.76	5.51	7.35	8.27	KN
Trans wind Load at Sup.	19.01	16.63	15.84	12.67	6.34	33.79	29.57	28.16	22.53	11.26	KN
Trans wind Load at CB/Railing	12.40	10.85	10.33	8.27	4.13	22.04	19.29	18.37	14.70	7.35	KN
Total wind force In Trans direc.	**31.41**	**27.48**	**26.17**	**20.94**	**10.47**	**55.84**	**48.86**	**46.53**	**37.22**	**18.61**	KN
Total wind force In long direc.	**0.00**	**3.93**	**7.85**	**10.47**	**11.78**	**0.00**	**6.98**	**13.96**	**18.61**	**20.94**	KN
CG of Load acting from Fdn. Bot.(Trans	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	m
CG of Load acting from Fdn. Bot.(Long.)		13.89	13.89	13.89	13.89		13.89	13.89	13.89	13.89	m

Upward/ Downward Vertical wind Load = **135** KN

Wind load on Superstructure Span 2						
Length of Span			=	26.40	26.40	m
Width of Span			=	10.8	10.80	m
Depth of Superstructure			=	1.77	1.77	m
Height of Railing/CB			=	1.1	1.1	m
Span/Width Ratio			=	2.4	2.4	
Span/Depth Ratio			=	14.9	14.9	
Aeroelastic Instability Investigation			=	**NO**	**NO**	
Tributary length for transverse direction			=	13.2	13.2	m
Tributary length for Longitudinal direction			=	13.2	13.2	m
Height of superstructure top from Bed level/HFL		H1	=	11.911	11.911	m
Height of superstructure top from Bed level/HFL		H2	=	10.14	10.141	m
Height of CB/Railing top from Bed level/HFL		H3	=	13.011	13.011	m
Terrain Clasification			=	Open Country	Open Country	
Friction Velocity V_0			=	13.2	13.2	Km/hr
Friction Length Zo			=	70	70	mm

Design Wind Velocity at design elevation	$V_{DZ}=2.5\ V_0(V_{10}/V_b)\ln(Z/Z_0)$	H1	=	169.51	169.5	Km/hr
		H2	=	164.2	164.2	Km/hr
		H3	=	172.43	172.4	Km/hr
Base Wind pressure Correspondiing to $V_B(P_B)$			=	0.0024	0.0024	Mpa
Design Wind pressure p_D	$P_D=P_B(V_{DZ}/V_B)^2$	H1	=	0.0027	0.0027	Mpa
		H2	=	0.0025	0.0025	Mpa
		H3	=	0.0028	0.0028	Mpa
Transverse wind Loading in superstructure				4.62	4.62	KN/m
Transverse wind Loading in CB/railing				3.01	3.01	KN/m
Total Transverse wind				**7.64**	**7.64**	KN/m
				OK	**OK**	

Summary of Wind Load On Superstructure Span2

	WITH LL					WITH OUT LL					
Wind attack angle	**0**	**15**	**30**	**45**	**60**	**0**	**15**	**30**	**45**	**60**	
P_B Trans	0.0024	0.0021	0.0020	0.0016	0.0008	0.0024	0.0021	0.0020	0.0016	0.0008	MPa
P_B Long	0.0000	0.0003	0.0006	0.0008	0.0009	0.0000	0.0003	0.0006	0.0008	0.0009	MPa
P_D at H1 Trans	0.0027	0.0024	0.0022	0.0018	0.0009	0.0027	0.0024	0.0022	0.0018	0.0009	MPa
P_D at H2 Trans	0.0025	0.0022	0.0021	0.0017	0.0008	0.0025	0.0022	0.0021	0.0017	0.0008	MPa
P_D at H3 Trans	0.0028	0.0024	0.0023	0.0019	0.0009	0.0028	0.0024	0.0023	0.0019	0.0009	MPa
P_D at H1 Long	0.0000	0.0003	0.0007	0.0009	0.0010	0.0000	0.0003	0.0007	0.0009	0.0010	MPa
P_D at H2 Long	0.0000	0.0003	0.0006	0.0008	0.0009	0.0000	0.0003	0.0006	0.0008	0.0009	MPa
P_D at H3 Long	0.0000	0.0003	0.0007	0.0009	0.0010	0.0000	0.0003	0.0007	0.0009	0.0010	MPa
Long. wind Load at Sup.	0.00	2.41	4.83	6.43	7.24	0.00	4.29	8.58	11.44	12.87	KN
Long wind Load at CB/Railing	0.00	1.57	3.15	4.20	4.72	0.00	2.80	5.60	7.46	8.39	KN
Trans wind Load at Sup.	19.30	16.89	16.08	12.87	6.43	34.31	30.02	28.59	22.87	11.44	KN
Trans wind Load at CB/Railing	12.59	11.02	10.49	8.39	4.20	22.38	19.59	18.65	14.92	7.46	KN
Total wind force In Trans direc.	**31.89**	**27.90**	**26.58**	**21.26**	**10.63**	**56.70**	**49.61**	**47.25**	**37.80**	**18.90**	KN
Total wind force In Long direc.	**0.00**	**3.99**	**7.97**	**10.63**	**11.96**	**0.00**	**7.09**	**14.17**	**18.90**	**21.26**	KN
CG of Load acting from Fdn. Bot.(Trans	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	13.89	m
CG of Load acting from Fdn. Bot.(Long.)		13.89	13.89	13.89	13.89		13.89	13.89	13.89	13.89	m

Upward/ Downward Vertical wind Load = **137** KN

Wind Load Calculation For Substructures

		With LL	**With Out LL**	
For Pier Shaft				
Design wind pressure	=	0.0019	0.0019	Mpa
Exposed surface of pier shaft End face	=	14.81	14.81	m^2
Exposed surface of pier shaft Front face	=	14.81	14.81	m^2
Force in Long. Direction	=	**8.90**	**15.82**	KN
Force in Trans. Direction	=	**8.90**	**15.82**	KN
Cg of Force acting From Fdn Bottom	=	6.4	6.4	m
For Pier Cap				
Exposed surface of pier Cap End face	=	3.5	3.5	m^2
Exposed surface of pier Cap Front face	=	8.4	8.4	m^2
Force in Long. Direction	=	**5.0**	**9.0**	KN
Force in Trans. Direction	=	**2.1**	**3.7**	KN
Cg of Force acting From Fdn Bottom	=	11.23	11.23	m
Total long. Force in substructure	=	**13.95**	**24.80**	KN
Total Trans. Force in substructure	=	11.00	19.56	KN
Cg of Force acting From Fdn Bottom	=	7.333	7.333	m

Wind Load Calculation For LL (WL)

	For Span 1					For Span 2					
Wind attack angle	**0**	**15**	**30**	**45**	**60**	**0**	**15**	**30**	**45**	**60**	
Normal Component	1.46	1.28	1.20	0.96	0.50	1.46	1.28	1.20	0.96	0.50	KN/m
Parallel Component	0.00	0.18	0.35	0.47	0.55	0.00	0.18	0.35	0.47	0.55	KN/m
Design Wind load on LL in Long. Direc	0.00	2.34	4.55	6.11	7.15	0.00	2.38	4.62	6.20	7.26	KN
Design Wind load on LL in Trans. Direc	19.0	16.6	15.6	12.5	6.5	19.3	16.9	15.8	12.7	6.6	KN
Cg of Force acting From Fdn Bottom	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	m

CALCULATION OF WATER FORCES

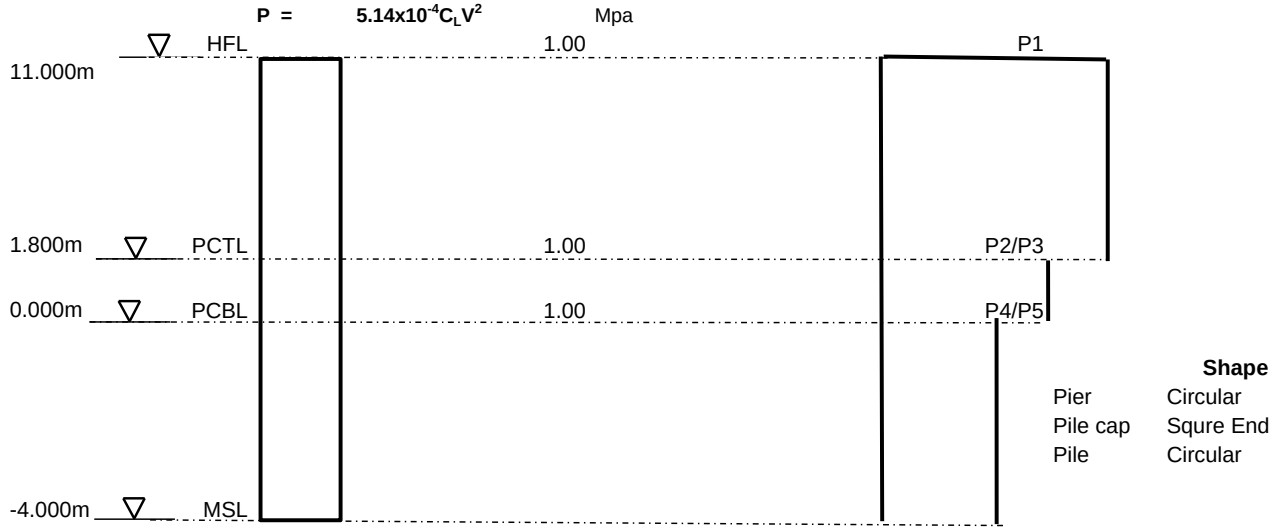
Highest Flood Level (HFL)	=	11.00	m
Pile Cap Top Level(PCTL)	=	1.80	m
Pile Cap Bottom Level(PCBL	=	0.00	m
Max Scour Level(MSL)	=	-4.00	m
Max.Mean Velocity(V)	=	1	m/s
Square of velocity at free Surface($V^2=2V^2$)	=	1	m^2/s^2
Square of velocity at Pile Cap top	=	1	m^2/s^2
Square of velocity at Pile Cap Bottom	=	1	m^2/s^2
Skew Angle of structure(α)	=	0	deg
Length of pier (In Long. Direction)	=	2.00	m
Width of pier (In Trans. Direction)	=	2.00	m
Dia. of Pile	=	1.2	
No. of Piles in Longitudinal Direction	=	2	Nos.
No. of Piles in Transverse Direction	=	2	Nos.
Width of Piles in Longitudinal Direction	=	1.06	m
Width of Piles in Transverse Direction	=	1.06	m
Dimension of Pile Cap in Longitudinal Direction	=	5.3	m
Dimension of Pile Cap in Transverse Direction	=	5.3	m

On Piers Parallel to the direction of the water current,The intensity of pressure shall be calculated from the below equation

P = $5.14x10^{-4}C_DV^2$ Mpa

On Piers Perpendicular to the direction of the water current,The intensity of pressure shall be calculated from the below equation

P = $5.14x10^{-4}C_LV^2$ Mpa



Velocity Distribution(m^2/s^2) **Pressure Distribution(KN/m^2)**

For ϕ = α+20 0+30 **30** deg

Component		Longitudinal direction C_L	Pressure ($KN/m^{2)}$	Force(KN)	Moment At fdn. Top(KN-m)	Moment At fdn. Bot(KN-m)	Transverse direction C_D	Pressure ($KN/m^{2)}$	Force(KN)	Moment At fdn. Top(KN-m)	Moment At fdn. Bot(KN-m)
Pier	Top(P1)	1.00	0.129	2.36	11	15	0.7	0.27	4.97	23	32
	Bottom(P2)	1.00	0.129				0.7	0.27			
Pile cap	Top(P3)	1.00	0.129	1.23	0	1	1.4	0.54	5.15	0	5
	Bottom(P4)	1.00	0.129				1.4	0.54			
Pile	Top(P5)	1.00	0.129	0.273			0.7	0.27	0.574		

Summary Of Water Current Forces:-

	Longitudinal Direction		Transverse Direction	
	HL(KN)	ML(KN-M)	HT(KN)	MT(KN-M)
Total Force & Moment at Pile cap top	**2**	**11**	**5**	**23**
Total Force & Moment at Pile cap Bottom	**4**	**16**	**10**	**36**

SEISMIC LOAD CALCULATION

Longitudianl direction-

Elatic Seismic response coefficient C_{sm} = $1.2xAS/T_m^{2/3}$ <=2.5A

C_{sm} = $A(0.8+4.0T_m$ <=2.0A (For Tm<=0.3 in Soil Profile III &IV)

C_{sm} = $3AS/T_m^{4/3}$ <=2.0A (For Tm>4 in Soil Profile III &IV)

Where, A= Acceleration Coefficient
S=Site Coefficient
T_m=Period of Vibration

Seismic Zone	=	**I**
Soil profile Zone	=	**II**
A	=	0.09
S	=	1.2

T_m = $2\pi\sqrt{w/gk}$

K1=Bridge Lateral stiffness = $\frac{P_0 L}{V_s Max}$

Total length of bridge	=	52.4	m	
P_0	=	1.000	N/m	
Distance of pier cap top where 1mm Deflection is required	=	10.125	m	
Moment of Inertia of substructue	=	0.515	m^4	
E	=	29440.09	Mpa	
Vsmax	=	1.2E-06	m	
K1	=	4.4E+07	N/m	
K2 (For Elastomeric Bearing)	=	13440000	N/m	
Total bridge stiffness K 1/(1/K1+1/K2)	=	**10286661**	N/m	

Contribution Mass Of Superstructure	M1	=	5243	KN
Contribution Mass Of Substructure	M2	=	1255	KN
Total	M	=	6498	KN
T_m		=	**1.594**	Sec
$C_{sm}L$		=	**0.095**	

Transverse direction-

Elatic Seismic response coefficient C_{sm} = $1.2xAS/T_m^{2/3}$ <=2.5A

C_{sm} = $A(0.8+4.0T_m$ <=2.0A (For Tm<=0.3 in Soil Profile III &IV)

C_{sm} = $3AS/T_m^{4/3}$ <=2.0A (For Tm>4 in Soil Profile III &IV)

Where, A= Acceleration Coefficient
S=Site Coefficient
T_m=Period of Vibration

Seismic Zone	=	**I**
Soil profile Zone	=	**II**
A	=	0.09
S	=	1.2

T_m = $2\pi\sqrt{w/gk}$

K1=Bridge Lateral stiffness = $\frac{P_0 L}{V_s Max}$

Total length of bridge	=	52.4	m
P_0	=	1.000	N/m
Distance of pier cap top where 1mm Deflection is required	=	10.125	m
Moment of Inertia of substructue	=	0.515	m^4
E	=	29440.087	Mpa
Vsmax	=	5.976E-07	m
K1	=	87686553	N/m
K2 (For Elastomeric Bearing)	=	13440000	N/m
Total bridge stiffness K 1/(1/K1+1/K2)	=	**11653787**	N/m

Contribution Mass Of Superstructure	M1	=	5243	KN
Contribution Mass Of Substructure	M2	=	1255	KN
Total	M	=	6498	KN
T_m		=	**1.498**	Sec
$C_{sm}T$		=	**0.099**	

LOAD COMBINATION FOR DESIGN OF FOUNDATION(NORMAL CASE)

Summary Of Verticle load Span 1

Dead Load

Total load	=	2007	KN
Long. Moment	=	1505	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to CB

Total load	=	325	KN
Long. Moment	=	244	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to Surfacing

Total load	=	272	KN
Long. Moment	=	204	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

FPLL

Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL1 Max. Reaction Case

Total load	=	579	KN
Long. Moment	=	434	KN-m
Trans. Moment	=	926.24	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	1.600	m

CWLL2 Max. Long. Moment Case

Total load	=	1031	KN
Long. Moment	=	773	KN-m
Trans. Moment	=	1650	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	1.600	m

CWLL3 Max. Trans. Moment Case

Total load	=	347	KN
Long. Moment	=	261	KN-m
Trans. Moment	=	1180.96	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	3.400	m

Summary Of Verticle load Span 2

Dead Load

Total load	=	2034	KN
Long. Moment	=	-1525	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to CB

Total load	=	330	KN
Long. Moment	=	-248	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to Surfacing

Total load	=	276	KN
Long. Moment	=	-207	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

FPLL

Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL1 Max. Reaction Case

Total load	=	722.54	KN
Long. Moment	=	-542	KN-m
Trans. Moment	=	1156.06	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	1.600	m

CWLL2 Max. Long. Moment Case

Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL3 Max. Trans. Moment Case

Total load	=	433.52	KN
Long. Moment	=	-325	KN-m
Trans. Moment	=	1473.98	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	3.400	m

Summary Of Horizontal load Span

Skew angle of Structure	=	0.0	Degree
Frictional/Shear Rating of Span 1	=	37.7	KN
Component in Long direction	=	37.740	KN
Component in Trans direction	=	0.000	KN
Braking force of Span 1	=	162.5	KN
Component in Long direction	=	162.500	KN
Component in Trans direction	=	0.000	KN

Centrifugal force of Span 1	=	0.1	KN
Component in Long direction	=	0.079	KN
Component in Trans direction	=	0.000	KN
Frictional/Shear Rating of Span 2	=	37.7	KN
Component in Long direction	=	37.740	KN
Component in Trans direction	=	0.000	KN
Braking force of Span 2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN
Centrifugal force of Span 2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN

Load of Substructure

Load of substructure	=	1305.23	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Load of Foundation

Load of substructure	=	1264.05	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Load of Soil on Foundation

Load of Soil	=	255.4531	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Summary of Loads at Bottom of Foundation :

Summary of Vertical Loads & its effect

Load Items	Load (KN)	e_L (m)	e_T (m)	Load Factor	Factored Load(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
DL of superstructure (Span1)	2007	0.75	0.00	1.00	2007	1505	0
DL of superstructure (Span1)	2034	-0.75	0.00	1.00	2034	-1525	0
SIDL CB(Span1)	325	0.75	0.00	1.00	325	244	0
SIDL CB(Span2)	330	-0.75	0.00	1.00	330	-248	0
SIDL Surfacing (Span1)	271.7	0.75	0.00	1.00	272	204	0
SIDL Surfacing (Span2)	275.9	-0.75	0.00	1.00	276	-207	0
FPLL Surfacing (Span1)	0.00	0.00	0.00	1.00	0	0	0
FPLL Surfacing (Span2)	0.00	0.00	0.00	1.00	0	0	0
CWLL1 (Span1)	579	0.75	1.60	1.00	579	434	926
CWLL1 (Span2)	723	-0.75	1.60	1.00	723	-542	1156
CWLL2(Span1)	1031	0.75	1.60	1.00	1031	773	1650
CWLL2 (Span2)	0.00	0.00	0.00	1.00	0	0	0
CWLL3 (Span1)	347	0.75	3.40	1.00	347	261	1181
CWLL3 (Span2)	434	-0.75	3.40	1.00	434	-325	1474
Wt. of Substructure	1305	0.00	0.00	1.00	1305	0	0
Wt. of Foundation	1264	0.00	0.00	1.00	1264	0	0
Wt. of soil on foundation (LWL)	255.45	0.00	0.00	1.00	255	0	0
Wt. of soil on foundation (HFL)	0	0.00	0.00	1.00	0	0	0
Wt. of Water	0	0.00	0.00	1.00	0	0	0
Buoyancy	-740	0.00	0.00	1.00	-740	0	0
Vertical wind force ('+'/'-')	272	0.00	2.70	1.00	272	0	733

Summary of Horizontal Loads & its effect

Load Items	Longitudinal Load H_L(KN)	Transverse Load H_T(KN)	Lever arm (m)	Load Factor	Factored Load H_L(KN)	Factored Load H_T(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
Shear Rating of Span1(Long.)	38		12.2	1.0	38		461	
Shear Rating of Span1(Trans.)		0	12.2	1.0		0		0
Braking force of Span1(Long.)	163		16.0	1.0	163		2602	
Braking force of Span1(Trans.)		0	16.0	1.0		0		0
CF force of Span1(Long.)	0		16.0	1.0	0		1	
CF force of Span1(Trans.)		0	16.0	1.0		0		0
Shear Rating of Span2(Long.)	38		12.2	1.0	38		461	
Shear Rating of Span2(Trans.)		0	12.2	1.0		0		0
Braking force of Span2(Long.)	0		16.0	1.0	0		0	
Braking force of Span2(Trans.)		0	16.0	1.0		0		0
CF force of Span2(Long.)	0		16.0	1.0	0		0	
CF force of Span2(Trans.)		0	16.0	1.0		0		0
WL on Sup.of Span1 with LL (Long)	4		13.9	1.0	4		55	
WL on Sup. of Span1 with LL (Trans)		27	13.9	1.0		27		382
WL on Sup.of Span2 with LL (Long)	4		13.9	1.0	4		55	
WL on Sup.of Span2 with LL (Trans)		28	13.9	1.0		28		388
WL on Sup.of Span1 (Long)	7		13.9	1.0	7		97	
WL on Sup. of Span1 (Trans)		49	13.9	1.0		49		679
WL on Sup.of Span2(Long)	7		13.9	1.0	7		98	
WL on Sup.of Span2(Trans)		50	13.9	1.0		50		689
WL on LL of Span1 (Long)	2		16.0	1.0	2		37	
WL on LL of Span1 (Trans)		17	16.0	1.0		17		266
WL on LL of Span2(Long)	2		16.0	1.0	2		38	
WL on LL of Span2(Trans)		17	16.0	1.0		17		271
WL on Substructure(Long) Without LL	25		7.3	1.0	25		182	
WL on Substructure(Trans) without LL		20	7.3	1.0		20		143
WL on Substructure(Long) with LL	14		7.3	1.0	14		102	
WL on Substructure(Trans) With LL		11	7.3	1.0		11		81
Water Load (Long)	4		4.5	1.0	4		16	
Water Load (trans)		10	3.6	1.0		10		36

Load Factor for Different Combination

Maximum case

Case		Load							
		DC	DW	LL/CE/BR/PL	WA	WS	WL	FR	TU
Strength-I		1.25	1.50	1.75	1.00	0.00	0.00	1.00	0.50
Strength-III		1.25	1.50	0.00	1.00	1.40	0.00	1.00	0.50
Strength-V		1.25	1.50	1.35	1.00	0.40	1.00	1.00	0.50
Service-I		1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00

Minimum case

Case		Load							
		DC	DW	LL/CE/BR/PL	WA	WS	WL	FR	TU
Strength-I		0.90	0.65	1.75	1.00	0.00	0.00	1.00	0.50
Strength-III		0.90	0.65	0.00	1.00	1.40	0.00	1.00	0.50
Strength-V		0.90	0.65	1.35	1.00	0.40	1.00	1.00	0.50
Service-I		1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00

LOAD COMBINATION

LWL CASE

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1a. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU	12499	4794	3644	322.3	0.00	322
2a. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU	12026	6335	2887	322.3	0.00	322
3a. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU	11588	4869	4646	322.3	0.00	322
4a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down	10602	427	3143	37.7	165	169
5a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up	9841	427	1089	37.7	165	169
6a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind	10222	955	2116	92	165	189
7a. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	11979	3956	3688	271	60	277
8a. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	11614	5145	3104	271	60	277
9a. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	11276	4014	4461	271	60	277
10a. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU	9369	3530	2874	249	53	255
11a. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU	9099	4411	2442	249	53	255
12a. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU	8849	3573	3447	249	53	255

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1b. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU	9402	4805	3644	322.3	0.00	322
2b. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU	8929	6347	2887	322.3	0.00	322
3b. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU	8491	4880	4646	322.3	0.00	322
4b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert.Wind Down	7504	438	3143	37.7	165	169
5b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert.Wind Up	6744	438	1089	37.7	165	169
6b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert. Wind	7124	966	2116	92	165	189
7b. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	8881	3967	3688	271	60	277
8b. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	8516	5156	3104	271	60	277
9b. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	8178	4025	4461	271	60	277
10b. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU	9369	3530	2874	249	53	255
11b. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU	9099	4411	2442	249	53	255
12b. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU	8849	3573	3447	249	53	255

HWL CASE

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1c. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU+1WA	11440	4810	3680	325.8	10.11	326
2c. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU+1WA	10967	6352	2924	325.8	10.11	326
3c. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU+1WA	10529	4885	4683	325.8	10.11	326
4c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down+1W	9543	443	3179	41.3	175	180
5c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up+1WA	8782	443	1126	41.3	175	180
6c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind+	9163	971	2152	96	175	200
7c. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	10920	3972	3725	274	70	283
8c. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	10555	5162	3141	274	70	283
9c. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	10217	4030	4498	274	70	283
10c. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU+1WA	8374	3546	2911	253	64	261
11c. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU+1WA	8104	4427	2478	253	64	261
12c. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU+1WA	7854	3589	3483	253	64	261

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1d. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU+1WA	8432	4821	3680	325.8	10.11	326
2d. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU+1WA	7959	6363	2924	325.8	10.11	326
3d. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU+1WA	7521	4896	4683	325.8	10.11	326
4d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down+1W	6535	454	3179	41.3	175	180
5d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up+1WA	5774	454	1126	41.3	175	180
6d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind+	6155	982	2152	96	175	200
7d. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	7911	3983	3725	274	70	283
8d. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	7547	5173	3141	274	70	283
9d. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	7209	4041	4498	274	70	283
10d. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU+1WA	8374	3546	2911	253	64	261
11d. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU+1WA	8104	4427	2478	253	64	261
12d. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU+1WA	7854	3589	3483	253	64	261

LOAD COMBINATION FOR DESIGN OF FOUNDATION(SEISMIC CASE)

Summary Of Seismic load

Skew angle of Structure	=	0.0	Degree
Long. Seismic Load Calculation			
Seismic force due to DL & SIDL Span1	=	497.9	KN
Component in Long direction	=	497.9	KN
Component in Trans direction	=	0.000	KN
Total Seismic Force in long. direction	=	**498**	
Seismic force due to DL & SIDL Span2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN
Long. Seismic Load Calculation For Substructure			
Weight of Substructre Below GL	=	1305	KN
Seismic Force	=	124	KN
Long. Seismic Load Calculation For Foundation	=		
Weight of Foundation	=	1264	KN
Seismic Force	=	120	KN
Trans. Seismic Load Calculation Span 1			
DL of Superstructure	=	2007	KN
Seismic Force	=	198.7	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	198.7	KN
SIDL Including Surfacing	=	597	KN
Seismic Force	=	59.1	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	59.1	KN
CWLL	=	0	KN
Seismic Force	=	0.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	0.0	KN
Trans seismic components for Dead load & SIDL including surfacing	=	0.0	KN
Trans. Seismic Load Calculation Span 2			
DL of Superstructure	=	2034	KN
Seismic Force	=	201.3	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	201.3	KN
SIDL Including Surfacing	=	606	KN
Seismic Force	=	60.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	60.0	KN
CWLL	=	0	KN
Seismic Force	=	0.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	0.0	KN
Trans seismic components for Dead load & SIDL including surfacing	=	0.0	KN
Long. Seismic Load Calculation For Substructure			
Weight of Substructre	=	1305	KN
Seismic Force	=	129	KN
Trans. Seismic Load Calculation For Foundation	=		
Weight of Foundation	=	1264	KN
Seismic Force	=	125	KN

Response Reduction Factor (R) Long = **1.0**

Response Reduction Factor (R) Trans = **1.0**

Summary of Loads at Bottom of Foundation :

Summary of Vertical Loads & its effect

Load Items	Load (KN)	e_L (m)	e_T (m)	Load Factor	Factored Load(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
DL of superstructure (Span1)	2007	0.75	0.00	1.00	2007	1505	0
DL of superstructure (Span1)	2034	-0.75	0.00	1.00	2034	-1525	0
SIDL CB(Span1)	325	0.75	0.00	1.00	325	244	0
SIDL CB(Span2)	330	-0.75	0.00	1.00	330	-248	0
SIDL Surfacing (Span1)	272	0.75	0.00	1.00	272	204	0
SIDL Surfacing (Span2)	276	-0.75	0.00	1.00	276	-207	0
FPLL Surfacing (Span1)	0	0.00	0.00	1.00	0	0	0
FPLL Surfacing (Span2)	0	0.00	0.00	1.00	0	0	0
CWLL1 (Span1)	579	0.75	1.60	1.00	579	434	926
CWLL1 (Span2)	723	-0.75	1.60	1.00	723	-542	1156
CWLL2(Span1)	1031	0.75	1.60	1.00	1031	773	1650
CWLL2 (Span2)	0	0.00	0.00	1.00	0	0	0
CWLL3 (Span1)	347	0.75	3.40	1.00	347	261	1181
CWLL3 (Span2)	434	-0.75	3.40	1.00	434	-325	1474
Wt. of Substructure	1305	0.00	0.00	1.00	1305	0	0
Wt. of Foundation	1264	0.00	0.00	1.00	1264	0	0
Wt. of soil on foundation (LWL)	255	0.00	0.00	1.00	255	0	0
Wt. of soil on foundation (HFL)	0	0.00	0.00	1.00	0	0	0
Wt. of Water	0	0.00	0.00	1.00	0	0	0
Buoyancy	-740	0.00	0.00	1.00	-740	0	0
Vertical wind force ('+'/'-')	0	0.00	2.70	1.00	0	0	0

Summary of Horizontal Loads & its effect

Load Items	Longitudinal Load H_L(KN)	TransverseLoad H_T(KN)	Lever arm (m)	Load Factor	Factored Load H_L(KN)	Factored Load H_T(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
Shear Rating of Span1(Long.)	38		12.2	1.0	38		461	
Shear Rating of Span1(Trans.)		0	12.2	1.0		0		0
Braking force of Span1(Long.)	163		16.0	1.0	163		2602	
Braking force of Span1(Trans.)		0	16.0	1.0		0		0
CF force of Span1(Long.)	0		16.0	1.0	0		1	
CF force of Span1(Trans.)		0	16.0	1.0		0		0
Shear Rating of Span2(Long.)	38		12.2	1.0	38		461	
Shear Rating of Span2(Trans.)		0	12.2	1.0		0		0
Braking force of Span2(Long.)	0		16.0	1.0	0		0	
Braking force of Span2(Trans.)		0	16.0	1.0		0		0
CF force of Span2(Long.)	0		16.0	1.0	0		0	
CF force of Span2(Trans.)		0	16.0	1.0		0		0
SL on DL Superstructure (Long.) - Span1	498		12.2	1.0	498		6087	
SL on DL Superstructure (Trans.) - Span1		199	13.6	1.0		199		2710
SL on SIDL (Trans.) - Span1		59	14.5	1.0		59		857
Trans compo. for Long. Seicmic force - Span1		0	13.6	1.0		0		0
SL on CWLL (Trans.) - Span1		0	15.7	1.0		0		0
SL on DL Superstructure (Long.) - Span2	0		12.2	1.0	0		0	
SL on DL Superstructure (Trans.) - Span2		201	13.6	1.0		201		2746
SL on SIDL (Trans.) - Span2		60	14.5	1.0		60		870
Trans compo. for Long. Seicmic force - Span2		0	13.6	1.0		0		0
SL on CWLL (Trans.) - Span2		0	15.7	1.0		0		0
SL on Substructure (Long.)	124		8.7	1.0	124		1079	
SL on Substructure (Trans.)		129	8.7	1.0		129		1125
SL on Foundation (Long.)	120		0.9	1.0	120		108	
SL on Foundation (Trans.)		125	0.9	1.0		125		113
Water Load (Long)	4		4.5	1.0	4		16	
Water Load (trans)		10	3.6	1.0		10		36

Load Factor for Different Combination

Maximum case

Case	Load							
	DC	DW	LL/CE/BR/PL	WA	FR	TU	SL	SL
Extreme Event-I	1.25	1.50	0.50	1.00	1.00	0.00	1.00	0.30

Minimum case

Case	Load							
	DC	DW	LL/CE/BR/PL	WA	FR	TU	SL	SL
Extreme Event-I	0.90	0.65	0.50	1.00	1.00	0.00	1.00	0.30

LOAD COMBINATION

LWL CASE

Longitudinal Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1La. EE-I 1.25DC+1.5DW+0.5LL1/CE/BR/PL+0TU+1SL	10872	8487	3567	823	232	855
2La. EE-I 1.25DC+1.5DW+0.5LL2/CE/BR/PL+0TU+1SL	10737	8927	3351	823	232	855
3La. Strength-I 1.25DC+1.5DW+0.5LL3/CE/BR/PL+0TU+1SL	10612	8508	3854	823	232	855

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Lb. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	7775	8498	3567	823	232	855
2Lb. EE-I 0.9DC+0.65DW+0.5LL2/CE/BR/PL+0TU+1SL	7640	8938	3351	823	232	855
3Lb. Strength-I 0.9DC+0.65DW+0.5LL3/CE/BR/PL+0TU+1SL	7515	8519	3854	823	232	855

Transverse Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Tc. EE-I 1.25DC+1.5DW+0.5LL1/CE/BR/PL+0TU+1SL	10872	3395	9462	304	773	831
2Tc. EE-I 1.25DC+1.5DW+0.5LL2/CE/BR/PL+0TU+1SL	10737	3836	9246	304	773	831
3Tc. Strength-I 1.25DC+1.5DW+0.5LL3/CE/BR/PL+0TU+1SL	10612	3417	9748	304	773	831

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Td. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	7775	3406	9462	304	773	831
2Td. EE-I 0.9DC+0.65DW+0.5LL2/CE/BR/PL+0TU+1SL	7640	3847	9246	304	773	831
3Td. Strength-I 0.9DC+0.65DW+0.5LL3/CE/BR/PL+0TU+1SL	7515	3428	9748	304	773	831

HWL CASE

Longitudinal Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Le. EE-I 1.25DC+1.5DW+0.5LL1/CE/BR/PL+0TU+1SL	9813	8503	3604	827	242	861
2Le. EE-I 1.25DC+1.5DW+0.5LL2/CE/BR/PL+0TU+1SL	9678	8943	3388	827	242	861
3Le. Strength-I 1.25DC+1.5DW+0.5LL3/CE/BR/PL+0TU+1SL	9553	8524	3890	827	242	861

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Lf. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6805	8514	3604	827	242	861
2Lf. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6670	8954	3388	827	242	861
3Lf. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6545	8535	3890	827	242	861

Transverse Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Tg. EE-I 1.25DC+1.5DW+0.5LL1/CE/BR/PL+0TU+1SL	9813	3411	9498	307	783	842
2Tg. EE-I 1.25DC+1.5DW+0.5LL2/CE/BR/PL+0TU+1SL	9678	3852	9282	307	783	842
3Tg. Strength-I 1.25DC+1.5DW+0.5LL3/CE/BR/PL+0TU+1SL	9553	3433	9785	307	783	842

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Th. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6805	3422	9498	307	783	842
2Th. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6670	3863	9282	307	783	842
3Th. EE-I 0.9DC+0.65DW+0.5LL1/CE/BR/PL+0TU+1SL	6545	3444	9785	307	783	842

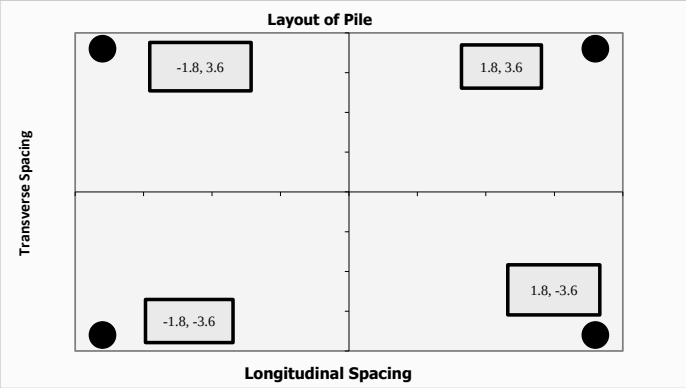
SUMMARY OF PILE REACTION

Verticle pile capacity *(Normal case)* = 3070 KN
Horizontal Pile Capacity = 120 KN

No. of Pile = 4 Nos.

∑xl2 = 13
∑xT2 = 52

Pile No	1	2	3	4	5	6
X_L (m)	1.80	1.80	-1.80	-1.80	0.00	0.00
X_T (m)	3.60	-3.60	3.60	-3.60	0.00	0.00



Summary of Pile Reactions (For Pile Capacity Check)

Maximum Reaction Case LWL

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1a	Strength-I	12499	4794	3644	4044	3538	2712	2206	4044	2206	322	80.6
2a	Strength-I	12026	6335	2887	4087	3686	2327	1926	4087	1926	322	80.6
3a	Strength-I	11588	4869	4646	3896	3251	2543	1898	3896	1898	322	80.6
4a	Strength-III	10602	427	3143	2928	2492	2809	2373	2928	2373	169	42.4
5a	Strength-III	9841	427	1089	2595	2444	2477	2325	2595	2325	169	42.4
6a	Strength-III	10222	955	2116	2835	2541	2570	2276	2835	2276	189	47.3
7a	Strength-V	11979	3956	3688	3800	3288	2701	2189	3800	2189	277	69.3
8a	Strength-V	11614	5145	3104	3834	3402	2404	1973	3834	1973	277	69.3
9a	Strength-V	11276	4014	4461	3686	3067	2571	1952	3686	1952	277	69.3
10a	Service-I	9369	3530	2874	3032	2633	2052	1652	3032	1652	255	63.7
11a	Service-I	9099	4411	2442	3057	2718	1832	1493	3057	1493	255	63.7
12a	Service-I	8849	3573	3447	2948	2469	1955	1477	2948	1477	255	63.7

Minimum Reaction Case LWL

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1b	Strength-I	9402	4805	3644	3271	2765	1936	1430	3271	1430	322	80.6
2b	Strength-I	8929	6347	2887	3314	2913	1551	1150	3314	1150	322	80.6
3b	Strength-I	8491	4880	4646	3123	2478	1768	1122	3123	1122	322	80.6
4b	Strength-III	7504	438	3143	2155	1719	2034	1597	2155	1597	169	42.4
5b	Strength-III	6744	438	1089	1822	1671	1701	1550	1822	1550	169	42.4
6b	Strength-III	7124	966	2116	2062	1768	1794	1500	2062	1500	189	47.3
7b	Strength-V	8881	3967	3688	3027	2515	1925	1413	3027	1413	277	69.3
8b	Strength-V	8516	5156	3104	3061	2630	1628	1197	3061	1197	277	69.3
9b	Strength-V	8178	4025	4461	2913	2294	1795	1176	2913	1176	277	69.3
10b	Service-I	9369	3530	2874	3032	2633	2052	1652	3032	1652	255	63.7
11b	Service-I	9099	4411	2442	3057	2718	1832	1493	3057	1493	255	63.7
12b	Service-I	8849	3573	3447	2948	2469	1955	1477	2948	1477	255	63.7

Maximum Reaction Case HWL

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1c	Strength-I	11440	4810	3680	3784	3272	2448	1936	3784	1936	326	81.5
2c	Strength-I	10967	6352	2924	3827	3421	2063	1657	3827	1657	326	81.5
3c	Strength-I	10529	4885	4683	3636	2986	2279	1629	3636	1629	326	81.5
4c	Strength-III	9543	443	3179	2668	2226	2545	2103	2668	2103	180	45.0
5c	Strength-III	8782	443	1126	2335	2179	2212	2056	2335	2056	180	45.0
6c	Strength-III	9163	971	2152	2575	2276	2305	2006	2575	2006	200	49.9
7c	Strength-V	10920	3972	3725	3540	3023	2437	1920	3540	1920	283	70.8
8c	Strength-V	10555	5162	3141	3574	3137	2140	1704	3574	1704	283	70.8
9c	Strength-V	10217	4030	4498	3426	2802	2307	1682	3426	1682	283	70.8
10c	Service-I	8374	3546	2911	2788	2384	1803	1399	2788	1399	261	65.2
11c	Service-I	8104	4427	2478	2813	2469	1583	1239	2813	1239	261	65.2
12c	Service-I	7854	3589	3483	2704	2220	1707	1223	2704	1223	261	65.2

Minimum Reaction Case HWL

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1d	Strength-I	8432	4821	3680	3033	2522	1694	1183	3033	1183	326	81.5
2d	Strength-I	7959	6363	2924	3077	2670	1309	903	3077	903	326	81.5
3d	Strength-I	7521	4896	4683	2885	2235	1525	875	2885	875	326	81.5
4d	Strength-III	6535	454	3179	1918	1476	1791	1350	1918	1350	180	45.0
5d	Strength-III	5774	454	1126	1585	1428	1459	1302	1585	1302	180	45.0
6d	Strength-III	6155	982	2152	1825	1526	1552	1253	1825	1253	200	49.9
7d	Strength-V	7911	3983	3725	2790	2272	1683	1166	2790	1166	283	70.8
8d	Strength-V	7547	5173	3141	2823	2387	1386	950	2823	950	283	70.8
9d	Strength-V	7209	4041	4498	2676	2051	1553	929	2676	929	283	70.8
10d	Service-I	8374	3546	2911	2788	2384	1803	1399	2788	1399	261	65.2
11d	Service-I	8104	4427	2478	2813	2469	1583	1239	2813	1239	261	65.2
12d	Service-I	7854	3589	3483	2704	2220	1707	1223	2704	1223	261	65.2

Maximum Reaction Case LWL Extreme Event

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1La	Ex.Event-I	10872	8487	3567	4145	3649	1787	1292	4145	1292	855	213.8
2La	Ex.Event-I	10737	8927	3351	4157	3691	1677	1212	4157	1212	855	213.8
3La	Ex.Event-I	10612	8508	3854	4102	3567	1739	1204	4102	1204	855	213.8
1Tc	Ex.Event-I	10872	3395	9462	3847	2533	2904	1589	3847	1589	831	207.7
2Tc	Ex.Event-I	10737	3836	9246	3859	2575	2794	1510	3859	1510	831	207.7
3Tc	Ex.Event-I	10612	3417	9748	3805	2451	2855	1502	3805	1502	831	207.7

Minimum Reaction Case LWL Extreme Event

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1Lb	Ex.Event-I	7775	8498	3567	3372	2876	1011	516	3372	516	855	213.8
2Lb	Ex.Event-I	7640	8938	3351	3384	2919	901	436	3384	436	855	213.8
3Lb	Ex.Event-I	7515	8519	3854	3329	2794	963	428	3329	428	855	213.8
1Td	Ex.Event-I	7775	3406	9462	3074	1760	2128	814	3074	814	831	207.7
2Td	Ex.Event-I	7640	3847	9246	3086	1802	2018	734	3086	734	831	207.7
3Td	Ex.Event-I	7515	3428	9748	3032	1678	2080	726	3032	726	831	207.7

Maximum Reaction Case HWL Extreme Event

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1Le	Ex.Event-I	9813	8503	3604	3885	3384	1523	1022	3885	1022	861	215.4
2Le	Ex.Event-I	9678	8943	3388	3897	3426	1413	942	3897	942	861	215.4
3Le	Ex.Event-I	9553	8524	3890	3842	3302	1474	934	3842	934	861	215.4
1Tg	Ex.Event-I	9813	3411	9498	3587	2268	2639	1320	3587	1320	842	210.4
2Tg	Ex.Event-I	9678	3852	9282	3599	2310	2529	1240	3599	1240	842	210.4
3Tg	Ex.Event-I	9553	3433	9785	3545	2186	2591	1232	3545	1232	842	210.4

Minimum Reaction Case HWL Extreme Event

Load Case	Comb.	P (KN)	ML (KN-m)	MT (KN-m)	R1 (KN)	R2 (KN)	R3 (KN)	R4 (KN)	Max. Load In pile(KN)	Min. Load In pile(KN)	HR (KN)	Horz. Per Pile(KN)
1Lf	Ex.Event-I	6805	8514	3604	3134	2634	769	269	3134	269	861	215.4
2Lf	Ex.Event-I	6670	8954	3388	3146	2676	659	189	3146	189	861	215.4
3Lf	Ex.Event-I	6545	8535	3890	3092	2552	721	181	3092	181	861	215.4
1Th	Ex.Event-I	6805	3422	9498	2836	1517	1886	566	2836	566	842	210.4
2Th	Ex.Event-I	6670	3863	9282	2849	1559	1776	486	2849	486	842	210.4
3Th	Ex.Event-I	6545	3444	9785	2794	1435	1837	478	2794	478	842	210.4

Summary

LWL

Comb.	MAX. Reaction (KN)	Min. Reaction (KN)	HL/Pile(KN)
Strength-I	4087	1122	81
Strength-III	2928	1500	47
Strength-V	3834	1176	69
Ex.Event-I	4157	428	214
Service-I	3057	1477	64

HWL

Comb.	MAX. Reaction (KN)	Min. Reaction (KN)	HL/Pile(KN)
Strength-I	3827	875	81
Strength-III	2668	1253	50
Strength-V	3574	929	71
Ex.Event-I	3897	181	215
Service-I	2813	1223	65

DESIGN OF PILE CAP

Bending along L-L axis

Dia. of pier		=	1.80	m
Side of Eq. Square of Pier		=	1.60	m
Dia. of Pile		=	1.20	m
Depth of pile cap		=	1.80	m
Width of Pile cap		=	12.7	m
Projection of pile cap		=	0.25	m
Edge of Pile Cap from C/L of Pile		=	0.85	m
Depth of soil over pile cap		=	0	m
Unit. Wt of Concrete		=	25	KN/m^3
Unit. Wt of Soil		=	20	KN/m^3
Load factor for soil		=	0.9	
Load factor for S/w		=	1.25	

LWL Case

Total reaction of Pile no. 1 to 2 =		=	7848	KN
Distance of the Max Reaction Force from face of Pier		=	1.002	m
Width Of Pile Cap		=	5.3	m
Max moment at face of support		=	1484	KN-m/m
Bending moment due to self wt. of pile cap		=	69.5	KN-m/m
Bending moment due to Soil. over pile cap		=	0.0	KN-m/m
Net Bending Moment		=	**1415**	KN-m/m

HWL Case

Total reaction of Pile no. 1 to 6 =		=	7323	KN
Distance of the Max Reaction Force from face of Pier		=	1.002	m
Width Of Pile Cap		=	5.3	m
Max moment at face of support		=	1385	KN-m/m
Bending moment due to self wt. of pile cap		=	69.5	KN-m/m
Bending moment due to Soil. over pile cap		=	0.0	KN-m/m
Net Bending Moment		=	**1316**	KN-m/m
Design Bending Moment For Pile Cap		=	**1415**	KN-m/m

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.0875	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	1702.5	mm
Width of Section (b)		=	1000	mm
Main Bar Dia. Provided		=	25	mm
Distribution Bar Dia. Provided		=	10	mm
Clear Cover Provided		=	75	mm

DESIGN OF PILE CAP

Area of steel is obtained by solving the quadratic equation:- **As =[B -√(B² - 4*Mn * A)]/(2 * A)**

Where, B = $d*f_y$ 715050
A = $(f_y^2)/(1.7 * f_c' * b)$ 3.46

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **22.22** cm^2/m

Provide	**25**	@	**140**	+	**0**	@	**240**	=	**35.06**	cm^2/m

(at tension face) **OK**

Neutral axis a = $A_s f_y/0.85 f_c b$ = 57.7 mm
c = a/β1 = 67.9 mm
ε_c = 0.003
ε_t = $(d-c)*\varepsilon_c/c$ = 0.072 **Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.31290881	Mpa
Gross Moment of Inertia	I_g		=	4.86E+11	mm^4
Distance from tension face to Cg	y_t		=	900	mm
Cracking Moment	Mcr		=	2868.97076	KN-m
1.2 times M_{cr}			=	3442.76491	KN-m
Reinforcement required for 1.2 Cracking moment			=	**54.96**	cm^2/m
1.33 times M_u			=	1881.8	KN-m
Reinforcement required for 1.33 M_u			=	**29.67**	cm^2/m
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**5.79**	cm^2/m
So Reqiured area of Reinforcement			=	**29.67**	cm^2/m

OK

Moment In service stage

LWL Case

Total reaction of Pile no. 1 to 2 =	=	5775	KN
Max moment at face of support	=	1092.21	KN-m/m
Bending moment due to self wt. of pile cap	=	77.2	KN-m/m
Bending moment due to Soil. over pile cap	=	0.0	KN-m/m
Net Bending moment in Service I	=	**1015.0**	KN-m/m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c)))$ = 1.072

γe 0.75

Modular ratio (n)	=	6.00	
Position of nutural axis from comp face (x)	=	247.431	mm
MOA of compression area	=	30610950.9	mm^2
MOA of Tension area	=	30610951	mm^2
Diff	=	0.000	
MOI of cracked section	=	4.959E+10	mm^4
Stress in tension reinf (f_{ss})	=	**178.69**	Mpa
Stress in Concrete (fcs)	=	**5.1**	Mpa
		OK	
Required Spacing of reinforcement s	=	**482**	mm
		OK	

Side face Reinforcement

Area of skin reinforcement A_{sk} = 9.425 cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

DESIGN OF PILE CAP

Check For One way Shear

Shear acts deff from face of column				=	1481	KN
Factored Wt. of Soil				=	0.0	KN
Factored Wt. of Pile cap				=	104.2	KN
Net facrored shear force			V_u	=	**1347**	KN
Effective Shear depth			d_v	=	1674	mm
Width of critical section			b_v	=	1000	mm
β				=	2	
θ				=	45	degree
Strength reduction factor ϕ for shear				=	0.9	
Shear stress on concrete			υ_u	=	0.89	Mpa
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$		V_c	=	1522	KN
					OK	
Capacity of shear steel	Vs =	$A_v.\ f_y.d_v.\cot\theta/s$			0	KN
Spacing of Stirrups			s	=	200	mm

Provide	**0**	dia.	**5 L**	Stirrups	**200**	Spacing	A_{sw} provided	=	**0**	mm^2

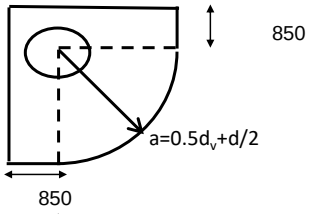
Max. spacing of transverse Reinforcement

If $\upsilon_u < 0.125f_c'$ then $S_{max}=0.8d_v<=600$	=	600	mm
If $\upsilon_u >= 0.125f_c'$ then $S_{max}=0.4d_v<=300$		**OK**	

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	1522	KN
	$V_n = 0.25fc'b_vd_v$	=	12552	KN

Check For Punching Shear around Cornar Pile

Maximum Rection In Cornar Pile = 4157 KN

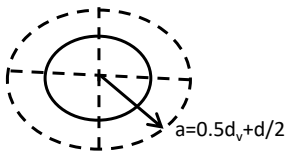


Nominal Shear resistance $Vn= Min(0.17+0.33/\beta_c)\sqrt{f_c'}b_0d_v, 0.33\sqrt{fc'}b_od_v$

a	=	1437	mm
β_c	=	1.0	
b_0	=	3957	mm
Vn1= $(0.17+0.33/bc)\sqrt{fc'}b_0d_v$	=	18136	KN
Vn2= $0.33\sqrt{fc'}b_odv$	=	11970	KN
Factored punching shear resistance V_n	=	10773	KN
		OK	

Check For Punching Shear around the Pier

Maximum Rection In Pier 12499



a	=	1737	mm
β_c	=	1.0	
b_0	=	10913	mm
Vn1= $(0.17+0.33/bc)\sqrt{fc'}b_0d_v$	=	50017	KN
Vn2= $0.33\sqrt{fc'}b_odv$	=	33012	KN
Factored punching shear resistance V_n	=	29710	KN
		OK	

STAAD Input For Design Of PILE

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 17-Jul-23
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0 -1 0; 3 0 -2 0; 4 0 -3 0; 5 0 -4 0; 6 0 -5 0; 7 0 -6 0; 8 0 -7 0;
9 0 -8 0; 10 0 -9 0; 11 0 -10 0; 12 0 -11 0; 13 0 -12 0; 14 0 -13 0;
15 0 -14 0; 16 0 -15 0; 17 0 -16 0; 18 0 -17 0; 19 0 -18 0; 20 0 -19 0;
21 0 -20 0; 22 0 -21 0; 23 0 -22 0; 24 0 -23 0; 25 0 -24 0; 26 0 -25 0;
**************************************************
*       STAAD.Pro Generated Comment        *
**************************************************
*REPEAT 26 1 1
**************************************************
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19;
19 19 20; 20 20 21; 21 21 22; 22 22 23; 23 23 24; 24 24 25; 25 25 26;
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 3.2308e+07
POISSON 0.2
DENSITY 25
ALPHA 1.2e-05
DAMP 0.05
END DEFINE MATERIAL
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 25
MEMBER PROPERTY INDIAN
1 TO 25 PRIS YD 1.2
********************************************************************************
SUPPORTS
1 FIXED BUT FX FY FZ
          6 FIXED BUT FY MX MY MZ KFX      449126 KFZ          449126
          7 FIXED BUT FY MX MY MZ KFX      575863 KFZ          575863
          8 FIXED BUT FY MX MY MZ KFX      650325 KFZ          650325
          9 FIXED BUT FY MX MY MZ KFX      709973 KFZ          709973
         10 FIXED BUT FY MX MY MZ KFX      760902 KFZ          760902
         11 FIXED BUT FY MX MY MZ KFX      805897 KFZ          805897
         12 FIXED BUT FY MX MY MZ KFX      846520 KFZ          846520
         13 FIXED BUT FY MX MY MZ KFX      883757 KFZ          883757
         14 FIXED BUT FY MX MY MZ KFX      918271 KFZ          918271
         15 FIXED BUT FY MX MY MZ KFX      950540 KFZ          950540
         16 FIXED BUT FY MX MY MZ KFX      980916 KFZ          980916
         17 FIXED BUT FY MX MY MZ KFX     1009669 KFZ         1009669
         18 FIXED BUT FY MX MY MZ KFX     1037013 KFZ         1037013
         19 FIXED BUT FY MX MY MZ KFX     1063118 KFZ         1063118
         20 FIXED BUT FY MX MY MZ KFX     1088124 KFZ         1088124
         21 FIXED BUT FY MX MY MZ KFX     1112147 KFZ         1112147
         22 FIXED BUT FY MX MY MZ KFX     1135282 KFZ         1135282
         23 FIXED BUT FY MX MY MZ KFX     1157614 KFZ         1157614
         24 FIXED BUT FY MX MY MZ KFX     1182697 KFZ         1182697
         25 FIXED BUT FY MX MY MZ KFX     1307186 KFZ         1307186
26 FIXED
********************************************************************************
LOAD 1 POINT LOAD
JOINT LOAD
1 FX 1
LOAD 2 WATER CURRENT
MEMBER LOAD
1 TO 25 UNI GX 10
********************************************************************************
PERFORM ANALYSIS
FINISH
```

DESIGN OF PILE

Diameter of Pile		=	1.20	m
Side of Eq. Square of Pile		=	1.06	m
No.of Pile		=	4	Nos.
Pile Length		=	25.0	m
Unit. Wt of Concrete		=	25	KN/m^3
Unit. Wt of Soil		=	20	KN/m^3
Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.0875	Mpa
Modulus of Elasticity of steel	E_s	=	200000	Mpa
Moment of Inertia of Pile	I_W	=	1.0179E+11	mm^4

Calculation of Surcharge load on pile

Height of Backfill Surcharge	=	0.0000	m
Depth of Pile Cap	=	1.8	m
Width of Pile Cap	=	5.3	m
Coefficient of Active Earth Pressure	=	0.279	
Earth Pressure at Pile cap Top	=	0	KN/m^2
Force on Pile Cap	=	0	KN

Force on Pile

Con. Load / Pile	=	0.0	KN
UDL on Pile	=	0.0	KN/m^2

Moment Co-efficient from STAAD

Bending moment coefficient for 1.0 KN horizontal load	=	3.50	KN-m/m
Bending moment coefficient for surcharge load on pile	=	77	KN-m
Strength Factor		1.00	
Service Fcaotr		1.00	

Calculation of Reinforcement :

Gross c/s area of the Pile			=	1.13	m^2
As min	0.008Ac	[CL-5.13-4.5-2]	=	9048	mm2
				OK	
As max	0.025Ac		=	28274	mm2
Clear cover			=	75	mm
Minimum Clear Spacing of Main Reinforcement			=	125	mm

Provide	**20 nos**	**25 Dia**	**Reinforcement.**
Provide	**0 nos**	**20 Dia**	**Reinforcement.**
Provide	**0 nos**	**20 Dia**	**Reinforcement.**

As Provided	=	**9813**	mm2
Percentage of Main Reinforcement	=	0.87%	
Spacing of Main Reinforcement	=	158	mm
		OK	

Provide 10 dia. 150 c/c as transverse ties

Area of Transverse steel	=	157	mm2
Minimum Transverse Reinforcement	=	194.832738	
Maximum Center to Center spacing	=	150	mm

Unsupported Length of Pile L_u	8	m	KL_u/R	**53.33**	**SLENDER**	
Radius of Gyration, $R=(I/A)^{1/2}$	0.3	m	$EI = E_c I_g/2.5(1+bd)$	1198654	KN-m^2	
K	2.00		$Pcr = \pi^2 EI/(KL_u)^2$	46212	KN	
Mo. Magnifier,$\delta = C_m/(1-P_u/\phi_k P_{cr})$			Cm	1.00	ϕ_k	0.75

Strength & Extreme Event Combination

Load Case	***P_{Max} (KN)***	***P_{Min} (KN)***	***H (KN)***	***Mu (KN-m)***	***Moment Magnifier*** δ Max	***Moment Magnifier*** δ Min	***Int Ratio1***	***Int Ratio2***	***Remarks***
1a	4044	2206	81	359	406	383	0.207	0.135	**OK**
2a	4087	1926	81	359	407	380	0.209	0.126	**OK**
3a	3896	1898	81	359	404	380	0.2	0.125	**OK**
4a	2928	2373	42	225	246	242	0.15	0.121	**OK**
5a	2595	2325	42	225	244	242	0.133	0.12	**OK**
6a	2835	2276	47	243	264	260	0.145	0.12	**OK**
7a	3800	2189	69	320	359	341	0.194	0.128	**OK**
8a	3834	1973	69	320	359	339	0.196	0.121	**OK**
9a	3686	1952	69	320	358	339	0.189	0.12	**OK**
1b	3271	1430	81	359	396	374	0.175	0.113	**OK**
2b	3314	1150	81	359	397	371	0.177	0.108	**OK**
3b	3123	1122	81	359	395	371	0.17	0.107	**OK**
4b	2155	1597	42	225	240	236	0.113	0.092	**OK**
5b	1822	1550	42	225	238	236	0.1	0.09	**OK**
6b	2062	1500	47	243	258	254	0.112	0.091	**OK**
7b	3027	1413	69	320	350	333	0.16	0.103	**OK**
8b	3061	1197	69	320	351	331	0.161	0.098	**OK**
9b	2913	1176	69	320	349	331	0.156	0.098	**OK**
1c	3784	1936	81	362	407	384	0.196	0.127	**OK**
2c	3827	1657	81	362	407	380	0.198	0.119	**OK**
3c	3636	1629	81	362	405	380	0.19	0.118	**OK**
4c	2668	2103	45	235	254	250	0.137	0.112	**OK**
5c	2335	2056	45	235	252	249	0.121	0.11	**OK**
6c	2575	2006	50	252	272	267	0.133	0.111	**OK**
7c	3540	1920	71	325	362	344	0.181	0.12	**OK**
8c	3574	1704	71	325	362	342	0.183	0.113	**OK**
9c	3426	1682	71	325	360	341	0.177	0.112	**OK**
1d	3033	1183	81	362	397	375	0.166	0.109	**OK**
2d	3077	903	81	362	398	372	0.168	0.105	**OK**
3d	2885	875	81	362	395	372	0.161	0.105	**OK**
4d	1918	1350	45	235	248	244	0.105	0.084	**OK**
5d	1585	1302	45	235	246	244	0.093	0.083	**OK**
6d	1825	1253	50	252	266	261	0.104	0.085	**OK**
7d	2790	1166	71	325	353	336	0.152	0.099	**OK**
8d	2823	950	71	325	354	334	0.153	0.096	**OK**
9d	2676	929	71	325	352	334	0.147	0.096	**OK**
1La	4145	1292	214	825	937	857	0.295	0.252	**OK**
2La	4157	1212	214	825	938	855	0.295	0.26	**OK**
3La	4102	1204	214	825	936	855	0.293	0.261	**OK**
1Tc	3847	1589	208	804	904	843	0.281	0.239	**OK**
2Tc	3859	1510	208	804	905	841	0.281	0.24	**OK**
3Tc	3805	1502	208	804	903	840	0.279	0.24	**OK**
1Lb	3372	516	214	825	914	838	0.273	0.363	**OK**
2Lb	3384	436	214	825	915	836	0.273	0.378	**OK**
3Lb	3329	428	214	825	913	836	0.272	0.379	**OK**
1Td	3074	814	208	804	882	823	0.26	0.3	**OK**

2Td	3086	734	208	804	883	821	0.261	0.312	**OK**
3Td	3032	726	208	804	881	821	0.259	0.313	**OK**
1Le	3885	1022	215	831	936	856	0.288	0.283	**OK**
2Le	3897	942	215	831	936	854	0.289	0.295	**OK**
3Le	3842	934	215	831	934	854	0.287	0.297	**OK**
1Tg	3587	1320	210	813	907	846	0.275	0.247	**OK**
2Tg	3599	1240	210	813	908	844	0.276	0.251	**OK**
3Tg	3545	1232	210	813	906	843	0.274	0.252	**OK**
1Lf	3134	269	215	831	913	837	0.269	0.413	**OK**
2Lf	3146	189	215	831	914	835	0.269	0.431	**OK**
3Lf	3092	181	215	831	912	835	0.268	0.431	**OK**
1Th	2836	566	210	813	886	827	0.258	0.347	**OK**
2Th	2849	486	210	813	886	825	0.258	0.362	**OK**
3Th	2794	478	210	813	885	825	0.257	0.364	**OK**

Shear Check of Pile Section :

Shear Co-efficient from STAAD

Shear coefficient for 1.0 KN horizontal load			=	1	KN/KN
Shear coefficient for surcharge load on pile			=	104	KN
Strength Factor				1	
Effective depth of setion	d_e	D/2+Dr/π	=	920	mm
Effective Shear depth	d_v		=	864	mm
Width of Critical section	b_v		=	1200	mm
β			=	2	
θ			=	45	degree
Strength reduction factor ϕ for shear			=	0.9	

Load Case	***H (KN)***	***Shear Stress Mpa υ_u***	***Vc (KN)***	***V_S (KN)***	***Vn1 V_c+V_s***	***Vn2 0.25fc'bvd v***	***Vn= ϕMin(Vn1,Vn2)***	***Status***
1a	185	0.20	943	380	1323	7776	1190	**Shear R/F Not Reqd.**
2a	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3a	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
4a	146	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
5a	146	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
6a	151	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
7a	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
8a	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
9a	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1b	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2b	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3b	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
4b	146	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
5b	146	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
6b	151	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
7b	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
8b	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
9b	173	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1c	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2c	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3c	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
4c	149	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
5c	149	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
6c	154	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
7c	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
8c	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
9c	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1d	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2d	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3d	185	0.20	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
4d	149	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
5d	149	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
6d	154	0.16	943	380	1323	7776	1323	**Shear R/F Not Reqd.**

7d	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
8d	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
9d	175	0.19	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1La	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2La	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3La	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Tc	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Tc	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Tc	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Lb	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Lb	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Lb	318	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Td	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Td	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Td	312	0.33	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Le	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Le	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Le	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Tg	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Tg	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Tg	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Lf	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Lf	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Lf	319	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
1Th	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
2Th	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**
3Th	314	0.34	943	380	1323	7776	1323	**Shear R/F Not Reqd.**

Service Combination

Load Case	***P_{Max} (KN)***	***P_{Min} (KN)***	***H (KN)***	***Mu (KN-m)***	***Moment Magnifier*** δ Max	
10a	3032	1652	63.74956	300	329	
11a	3057	1493	63.74956	300	329	
12a	2948	1477	63.74956	300	328	
10c	2788	1399	65.19725	305	332	
11c	2813	1239	65.19725	305	332	
12c	2704	1223	65.19725	305	331	

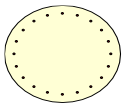
MIDAS OUTPUT

Property

1. Material

Concrete	None	
fc'	30000.00	kN/m²
Ec	29500000.00	kN/m²
Poisson's Ratio	0.20	
Weight Density	25.00	kN/m³
Nonlinear Property	Whitney Rectangular	
Rebar		
fy	420000.00	kN/m²
Es	200000000.00	kN/m²
Nonlinear Property	Bilinear Model	

2. Section



As	0.0098174	m²

Ⅰ. General

Area	1.125240191	m²
Shear Area (y)	0.965761567	m²
Shear Area (z)	0.965767319	m²
Ixx	0.201517942	m⁴
Iyy	0.100758769	m⁴
Izz	0.100758769	m⁴
Centroid (y)	0.599999994	m
Centroid (z)	0.599999994	m

Ⅱ. Section Modulus

Section Modulus (Top)	0.167931282	m³
Section Modulus (Bottom)	0.167931282	m³
Section Modulus (Right)	0.167931282	m³
Section Modulus (Left)	0.167931282	m³

Ⅲ. Principal Properties

Principal Angle	0	°
Iyy'	0.100758769	m⁴
Izz'	0.100758769	m⁴

Ⅳ. Plastic Properties

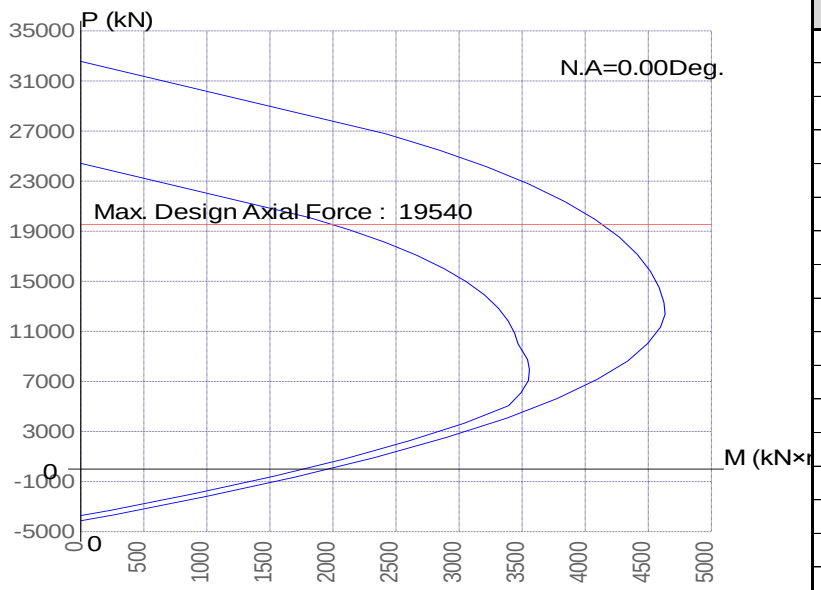
Plastic Modulus (Major axis)	0.285812316	m³
Plastic Modulus (Minor axis)	0.285812316	m³

3. Design Load Combination

No	Name	Pu(kN)	My(kN×m)	Mz(kN×m)	Vy(kN)	Vz(kN)	T(kN×m)
1	1a	2206.00	383.00	0.00	0.00	0.00	0.00
2	2a	1926.00	380.00	0.00	0.00	0.00	0.00
3	3a	1898.00	380.00	0.00	0.00	0.00	0.00
4	4a	2373.00	242.00	0.00	0.00	0.00	0.00
5	5a	2325.00	242.00	0.00	0.00	0.00	0.00
6	6a	2276.00	260.00	0.00	0.00	0.00	0.00
7	7a	2189.00	341.00	0.00	0.00	0.00	0.00
8	8a	1973.00	339.00	0.00	0.00	0.00	0.00
9	9a	1952.00	339.00	0.00	0.00	0.00	0.00
10	1b	1430.00	374.00	0.00	0.00	0.00	0.00
11	2b	1150.00	371.00	0.00	0.00	0.00	0.00
12	3b	1122.00	371.00	0.00	0.00	0.00	0.00
13	4b	1597.00	236.00	0.00	0.00	0.00	0.00
14	5b	1550.00	236.00	0.00	0.00	0.00	0.00
15	6b	1500.00	254.00	0.00	0.00	0.00	0.00
16	7b	1413.00	333.00	0.00	0.00	0.00	0.00
17	8b	1197.00	331.00	0.00	0.00	0.00	0.00
18	9b	1176.00	331.00	0.00	0.00	0.00	0.00
19	1c	1936.00	384.00	0.00	0.00	0.00	0.00
20	2c	1657.00	380.00	0.00	0.00	0.00	0.00
21	3c	1629.00	380.00	0.00	0.00	0.00	0.00
22	4c	2103.00	250.00	0.00	0.00	0.00	0.00
23	5c	2056.00	249.00	0.00	0.00	0.00	0.00
24	6c	2006.00	267.00	0.00	0.00	0.00	0.00
25	7c	1920.00	344.00	0.00	0.00	0.00	0.00
26	8c	1704.00	342.00	0.00	0.00	0.00	0.00
27	9c	1682.00	341.00	0.00	0.00	0.00	0.00
28	1d	1183.00	375.00	0.00	0.00	0.00	0.00
29	2d	903.00	372.00	0.00	0.00	0.00	0.00
30	3d	875.00	372.00	0.00	0.00	0.00	0.00
31	4d	1350.00	244.00	0.00	0.00	0.00	0.00
32	5d	1302.00	244.00	0.00	0.00	0.00	0.00
33	6d	1253.00	261.00	0.00	0.00	0.00	0.00
34	7d	1166.00	336.00	0.00	0.00	0.00	0.00
35	8d	950.00	334.00	0.00	0.00	0.00	0.00
36	9d	929.00	334.00	0.00	0.00	0.00	0.00
37	1La	1292.00	857.00	0.00	0.00	0.00	0.00
38	2La	1212.00	855.00	0.00	0.00	0.00	0.00
39	3La	1204.00	855.00	0.00	0.00	0.00	0.00
40	1Tc	1589.00	843.00	0.00	0.00	0.00	0.00
41	2Tc	1510.00	841.00	0.00	0.00	0.00	0.00
42	3Tc	1502.00	840.00	0.00	0.00	0.00	0.00
43	1Lb	516.00	838.00	0.00	0.00	0.00	0.00
44	2Lb	436.00	836.00	0.00	0.00	0.00	0.00
45	3Lb	428.00	836.00	0.00	0.00	0.00	0.00
46	1Td	814.00	823.00	0.00	0.00	0.00	0.00
47	2Td	734.00	821.00	0.00	0.00	0.00	0.00
48	3Td	726.00	821.00	0.00	0.00	0.00	0.00
49	1Le	1022.00	856.00	0.00	0.00	0.00	0.00
50	2Le	942.00	854.00	0.00	0.00	0.00	0.00
51	3Le	934.00	854.00	0.00	0.00	0.00	0.00
52	1Tg	1320.00	846.00	0.00	0.00	0.00	0.00
53	2Tg	1240.00	844.00	0.00	0.00	0.00	0.00
54	3Tg	1232.00	843.00	0.00	0.00	0.00	0.00
55	1Lf	269.00	837.00	0.00	0.00	0.00	0.00
56	2Lf	189.00	835.00	0.00	0.00	0.00	0.00
57	3Lf	181.00	835.00	0.00	0.00	0.00	0.00
58	1Th	566.00	827.00	0.00	0.00	0.00	0.00
59	2Th	486.00	825.00	0.00	0.00	0.00	0.00
60	3Th	478.00	825.00	0.00	0.00	0.00	0.00

P-M Curve

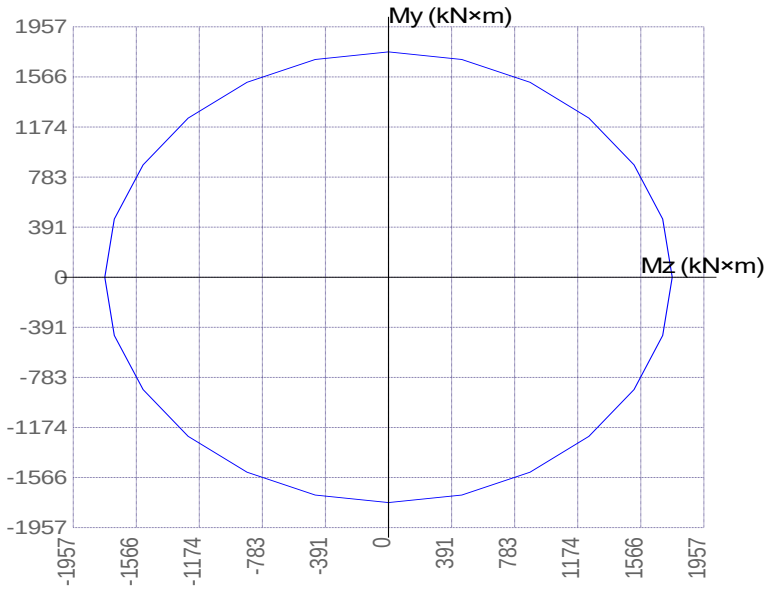
Mode : Angle = 0.00 °



Pu(kN)	Mn(kN×m)
24424.942	0.000
20091.285	1812.772
19118.396	2128.667
18125.632	2411.220
17084.348	2663.794
16022.890	2879.789
14976.719	3056.436
13913.731	3201.480
12865.413	3310.071
11844.169	3388.824
10884.287	3439.191
9983.820	3468.350
9361.881	3506.060
8743.534	3543.355
7948.323	3558.136
7071.337	3549.047
6109.065	3491.238
5069.037	3393.214
3674.583	3042.958
2273.625	2607.787
831.609	2098.537
-577.284	1527.831
-1953.487	901.543
-3295.392	233.237
-3710.977	0.000

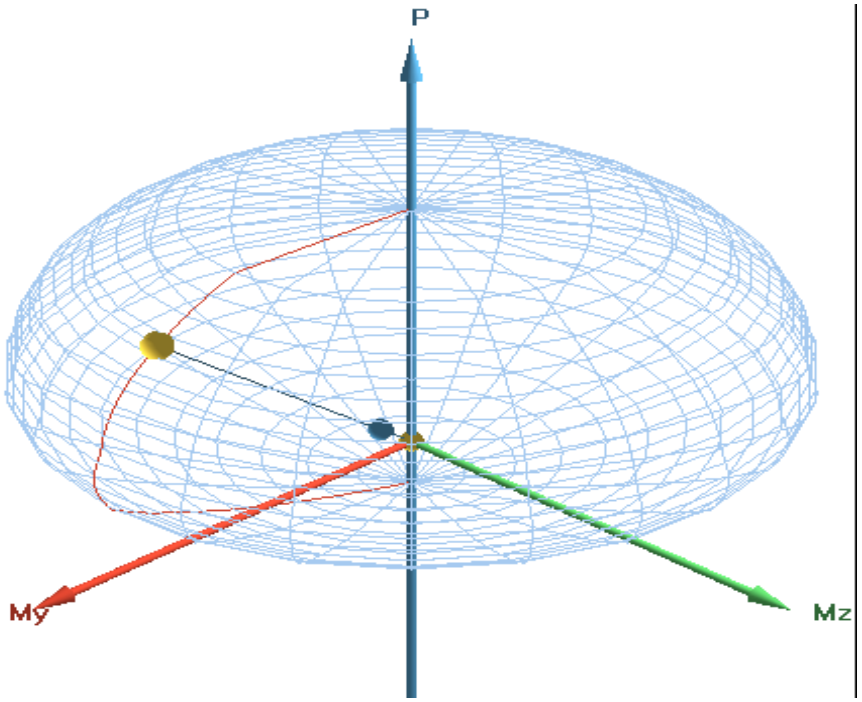
Mx-My Curve

Mode : Axial Force, P = 0.00 kN



My(kN×m)	Mz(kN×m)
1761.674	0.000
1702.562	455.619
1524.289	878.671
1244.249	1244.249
878.671	1524.289
455.619	1702.562
0.000	1761.674
-455.619	1702.562
-878.671	1524.289
-1244.249	1244.249
-1524.289	878.671
-1702.562	455.619
-1761.674	0.000
-1702.562	-455.619
-1524.289	-878.671
-1244.249	-1244.249
-878.671	-1524.289
-455.619	-1702.562
0.000	-1761.674
455.619	-1702.562
878.671	-1524.289
1244.249	-1244.249
1524.289	-878.671
1702.562	-455.619
1761.674	0.000

P-Mx-My Curve



Load Comb Name	Ratio		
	P-My	P-Mz	P-My/Mz
1a	0.135	0.000	0.135
2a	0.126	0.000	0.126
3a	0.125	0.000	0.125
4a	0.121	0.000	0.121
5a	0.120	0.000	0.120
6a	0.120	0.000	0.120
7a	0.128	0.000	0.128
8a	0.121	0.000	0.121
9a	0.120	0.000	0.120
1b	0.113	0.000	0.113
2b	0.108	0.000	0.108
3b	0.107	0.000	0.107
4b	0.092	0.000	0.092
5b	0.090	0.000	0.090
6b	0.091	0.000	0.091
7b	0.103	0.000	0.103
8b	0.098	0.000	0.098
9b	0.098	0.000	0.098
1c	0.127	0.000	0.127
2c	0.119	0.000	0.119
3c	0.118	0.000	0.118
4c	0.112	0.000	0.112
5c	0.110	0.000	0.110
6c	0.111	0.000	0.111
7c	0.120	0.000	0.120
8c	0.113	0.000	0.113
9c	0.112	0.000	0.112
1d	0.109	0.000	0.109
2d	0.105	0.000	0.105

3d	0.105	0.000	0.105
4d	0.084	0.000	0.084
5d	0.083	0.000	0.083
6d	0.085	0.000	0.085
7d	0.099	0.000	0.099
8d	0.096	0.000	0.096
9d	0.096	0.000	0.096
1La	0.252	0.000	0.252
2La	0.260	0.000	0.260
3La	0.261	0.000	0.261
1Tc	0.239	0.000	0.239
2Tc	0.240	0.000	0.240
3Tc	0.240	0.000	0.240
1Lb	0.363	0.000	0.363
2Lb	0.378	0.000	0.378
3Lb	0.379	0.000	0.379
1Td	0.300	0.000	0.300
2Td	0.312	0.000	0.312
3Td	0.313	0.000	0.313
1Le	0.283	0.000	0.283
2Le	0.295	0.000	0.295
3Le	0.297	0.000	0.297
1Tg	0.247	0.000	0.247
2Tg	0.251	0.000	0.251
3Tg	0.252	0.000	0.252
1Lf	0.413	0.000	0.413
2Lf	0.431	0.000	0.431
3Lf	0.431	0.000	0.431
1Th	0.347	0.000	0.347
2Th	0.362	0.000	0.362
3Th	0.364	0.000	0.364

LOAD COMBINATION FOR DESIGN OF PIER (NORMAL CASE)

Thickness of Foundation	=	1.8	m
Section from Pier Bottom	=	0.0	m

Summary Of Verticle load Span 1

Dead Load			
Total load	=	2007	KN
Long. Moment	=	1505	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to CB			
Total load	=	325	KN
Long. Moment	=	244	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to Surfacing			
Total load	=	272	KN
Long. Moment	=	204	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	0.000	m

FPLL			
Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL1 Max. Reaction Case			
Total load	=	579	KN
Long. Moment	=	434	KN-m
Trans. Moment	=	-260.51	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	-0.450	m

CWLL2 Max. Long. Moment Case			
Total load	=	1031	KN
Long. Moment	=	773	KN-m
Trans. Moment	=	1650	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	1.600	m

CWLL3 Max. Trans. Moment Case			
Total load	=	347	KN
Long. Moment	=	261	KN-m
Trans. Moment	=	1094.12	KN-m
Long. Eccentricity	=	0.750	m
Trans. Eccentricity	=	3.150	m

Summary Of Verticle load Span 2

Dead Load			
Total load	=	2034	KN
Long. Moment	=	-1525	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to CB			
Total load	=	330	KN
Long. Moment	=	-248	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

SIDL Due to Surfacing			
Total load	=	276	KN
Long. Moment	=	-207	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	0.000	m

FPLL			
Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL1 Max. Reaction Case			
Total load	=	722.5400	KN
Long. Moment	=	-542	KN-m
Trans. Moment	=	-325.14	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	-0.450	m

CWLL2 Max. Long. Moment Case			
Total load	=	0.0000	KN
Long. Moment	=	0	KN-m
Trans. Moment	=	0.00	KN-m
Long. Eccentricity	=	0.000	m
Trans. Eccentricity	=	0.000	m

CWLL3 Max. Trans. Moment Case			
Total load	=	433.52	KN
Long. Moment	=	-325	KN-m
Trans. Moment	=	1365.60	KN-m
Long. Eccentricity	=	-0.750	m
Trans. Eccentricity	=	3.150	m

Summary Of Horizontal load Span

Skew angle of Structure	=	0.0	Degree
Frictional/Shear Rating of Span 1	=	37.7	KN
Component in Long direction	=	37.740	KN
Component in Trans direction	=	0.000	KN
Braking force of Span 1	=	162.5	KN
Component in Long direction	=	162.500	KN
Component in Trans direction	=	0.000	KN

Centrifugal force of Span 1	=	0.1	KN
Component in Long direction	=	0.079	KN
Component in Trans direction	=	0.000	KN
Frictional/Shear Rating of Span 2	=	37.7	KN
Component in Long direction	=	37.740	KN
Component in Trans direction	=	0.000	KN
Braking force of Span 2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN
Centrifugal force of Span 2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN

Load of Substructure

Load of substructure	=	1305.23	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Load of Foundation

Load of substructure	=	0.00	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Load of Soil on Foundation

Load of Soil	=	0.00	KN
Long. Moment	=	0.000	KN-m
Trans. Moment	=	0.000	KN-m
Long. Eccentricity	=	0.0	m
Trans. Eccentricity	=	0.0	m

Summary of Loads at Bottom of Foundation :

Summary of Vertical Loads & its effect

Load Items	Load (KN)	e_L (m)	e_T (m)	Load Factor	Factored Load(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
DL of superstructure (Span1)	2007	0.75	0.00	1.00	2007	1505	0
DL of superstructure (Span1)	2034	-0.75	0.00	1.00	2034	-1525	0
SIDL CB(Span1)	325	0.75	0.00	1.00	325	244	0
SIDL CB(Span2)	330	-0.75	0.00	1.00	330	-248	0
SIDL Surfacing (Span1)	271.7	0.75	0.00	1.00	272	204	0
SIDL Surfacing (Span2)	275.9	-0.75	0.00	1.00	276	-207	0
FPLL Surfacing (Span1)	0.00	0.00	0.00	1.00	0	0	0
FPLL Surfacing (Span2)	0.00	0.00	0.00	1.00	0	0	0
CWLL1 (Span1)	579	0.75	-0.45	1.33	770	577	-346
CWLL1 (Span2)	723	-0.75	-0.45	1.33	961	-721	-432
CWLL2(Span1)	1031	0.75	1.60	1.33	1371	1029	2194
CWLL2 (Span2)	0.00	0.00	0.00	1.33	0	0	0
CWLL3 (Span1)	347	0.75	3.15	1.33	462	346	1455
CWLL3 (Span2)	434	-0.75	3.15	1.33	577	-432	1816
Wt. of Substructure	1305	0.00	0.00	1.00	1305	0	0
Wt. of Foundation	0	0.00	0.00	1.00	0	0	0
Wt. of soil on foundation (LWL)	0	0.00	0.00	1.00	0	0	0
Wt. of soil on foundation (HFL)	0	0.00	0.00	1.00	0	0	0
Wt. of Water	0	0.00	0.00	1.00	0	0	0
Buoyancy	-234	0.00	0.00	1.00	-234	0	0
Vertical wind force ('+'/'-')	272	0.00	2.70	1.00	272	0	733

Summary of Horizontal Loads & its effect

Load Items	Longitudinal Load H_L(KN)	Transverse Load H_T(KN)	Lever arm (m)	Load Factor	Factored Load H_L(KN)	Factored Load H_T(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
Shear Rating of Span1(Long.)	38		10.4	1.0	38		393	
Shear Rating of Span1(Trans.)		0	10.4	1.0		0		0
Braking force of Span1(Long.)	163		14.2	1.0	163		2309	
Braking force of Span1(Trans.)		0	14.2	1.0		0		0
CF force of Span1(Long.)	0		14.2	1.0	0		1	
CF force of Span1(Trans.)		0	14.2	1.0		0		0
Shear Rating of Span2(Long.)	38		10.4	1.0	38		393	
Shear Rating of Span2(Trans.)		0	10.4	1.0		0		0
Braking force of Span2(Long.)	0		14.2	1.0	0		0	
Braking force of Span2(Trans.)		0	14.2	1.0		0		0
CF force of Span2(Long.)	0		14.2	1.0	0		0	
CF force of Span2(Trans.)		0	14.2	1.0		0		0
WL on Sup.of Span1 with LL (Long)	4		12.1	1.0	4		47	
WL on Sup. of Span1 with LL (Trans)		27	12.1	1.0		27		332
WL on Sup.of Span2 with LL (Long)	4		12.1	1.0	4		48	
WL on Sup.of Span2 with LL (Trans)		28	12.1	1.0		28		337
WL on Sup.of Span1 (Long)	7		12.1	1.0	7		84	
WL on Sup. of Span1 (Trans)		49	12.1	1.0		49		591
WL on Sup.of Span2(Long)	7		12.1	1.0	7		86	
WL on Sup.of Span2(Trans)		50	12.1	1.0		50		600
WL on LL of Span1 (Long)	2		14.2	1.0	2		33	
WL on LL of Span1 (Trans)		17	14.2	1.0		17		236
WL on LL of Span2(Long)	2		14.2	1.0	2		34	
WL on LL of Span2(Trans)		17	14.2	1.0		17		240
WL on Substructure(Long) Without LL	25		5.5	1.0	25		137	
WL on Substructure(Trans) without LL		20	5.5	1.0		20		108
WL on Substructure(Long) with LL	14		5.5	1.0	14		77	
WL on Substructure(Trans) With LL		11	5.5	1.0		11		61
Water Load (Long)	2		4.6	1.0	2		11	
Water Load (trans)		5	4.6	1.0		5		23

Load Factor for Different Combination

Maximum case

Case		Load							
		DC	DW	LL/CE/BR/PL	WA	WS	WL	FR	TU
Strength-I		1.25	1.50	1.75	1.00	0.00	0.00	1.00	0.50
Strength-III		1.25	1.50	0.00	1.00	1.40	0.00	1.00	0.50
Strength-V		1.25	1.50	1.35	1.00	0.40	1.00	1.00	0.50
Service-I		1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00

Minimum case

Case		Load							
		DC	DW	LL/CE/BR/PL	WA	WS	WL	FR	TU
Strength-I		0.90	0.65	1.75	1.00	0.00	0.00	1.00	0.50
Strength-III		0.90	0.65	0.00	1.00	1.40	0.00	1.00	0.50
Strength-V		0.90	0.65	1.35	1.00	0.40	1.00	1.00	0.50
Service-I		1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00

LOAD COMBINATION

LWL CASE

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1a. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU	11351	4151	-1363	322.3	0.00	322
2a. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU	10722	6202	3840	322.3	0.00	322
3a. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU	10140	4252	5725	322.3	0.00	322
4a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down	8703	359	2845	37.7	165	169
5a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up	7942	359	792	37.7	165	169
6a. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind	8322	789	1819	92	165	189
7a. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	10659	3421	-283	271	60	277
8a. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	10174	5003	3731	271	60	277
9a. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	9724	3498	5185	271	60	277
10a. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU	8279	3046	-83	249	53	255
11a. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU	7920	4218	2890	249	53	255
12a. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU	7587	3103	3967	249	53	255

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1b. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU	8786	4162	-1363	322.3	0.00	322
2b. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU	8157	6213	3840	322.3	0.00	322
3b. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU	7574	4263	5725	322.3	0.00	322
4b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert.Wind Down	6137	370	2845	37.7	165	169
5b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert.Wind Up	5376	370	792	37.7	165	169
6b. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert. Wind	5757	800	1819	92	165	189
7b. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	8093	3432	-283	271	60	277
8b. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	7608	5014	3731	271	60	277
9b. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	7159	3509	5185	271	60	277
10b. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU	8279	3046	-83	249	53	255
11b. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU	7920	4218	2890	249	53	255
12b. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU	7587	3103	3967	249	53	255

HWL CASE

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1c. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU+1WA	11117	4162	-1340	324.6	4.97	325
2c. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU+1WA	10488	6213	3863	324.6	4.97	325
3c. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU+1WA	9906	4262	5748	324.6	4.97	325
4c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down+1W	8469	370	2868	40.1	170	175
5c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up+1WA	7708	370	815	40.1	170	175
6c. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind+1	8088	800	1841	95	170	195
7c. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	10425	3431	-260	273	65	281
8c. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	9940	5013	3754	273	65	281
9c. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	9490	3509	5208	273	65	281
10c. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU+1WA	8045	3057	-60	252	58	258
11c. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU+1WA	7686	4229	2913	252	58	258
12c. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU+1WA	7353	3114	3990	252	58	258

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1d. Strength-I 1.25DC+1.5DW+1.75LL1/CE/BR/PL+0.5TU+1WA	8552	4173	-1340	324.6	4.97	325
2d. Strength-I 1.25DC+1.5DW+1.75LL2/CE/BR/PL+0.5TU+1WA	7923	6224	3863	324.6	4.97	325
3d. Strength-I 1.25DC+1.5DW+1.75LL3/CE/BR/PL+0.5TU+1WA	7340	4274	5748	324.6	4.97	325
4d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Down+1W	5903	381	2868	40.1	170	175
5d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+Vert. Wind Up+1WA	5142	381	815	40.1	170	175
6d. Strength-III 1.25DC+1.5DW+1.4WS+0.5TU+ With out Vert.Wind+	5523	811	1841	95	170	195
7d. Strength-V 1.25DC+1.5DW+1.35LL1/CE/BR/PL+0.4WS+1WL+0.5	7859	3443	-260	273	65	281
8d. Strength-V 1.25DC+1.5DW+1.35LL2/CE/BR/PL+0.4WS+1WL+0.5	7374	5025	3754	273	65	281
9d. Strength-V 1.25DC+1.5DW+1.35LL3/CE/BR/PL+0.4WS+1WL+0.5	6925	3520	5208	273	65	281
10d. Service-I 1DC+1DW+1LL1/CE/BR/PL+0.3WS+1WL+1TU+1WA	8045	3057	-60	252	58	258
11d. Service-I 1DC+1DW+1LL2/CE/BR/PL+0.3WS+1WL+1TU+1WA	7686	4229	2913	252	58	258
12d. Service-I 1DC+1DW+1LL3/CE/BR/PL+0.3WS+1WL+1TU+1WA	7353	3114	3990	252	58	258

LOAD COMBINATION FOR DESIGN OF PIER(SEISMIC CASE)

Thickness of Foundation	=	1.8	m
Section from Pier Bottom	=	0.0	m
Summary Of Seismic load			
Skew angle of Structure	=	0.0	Degree
Long. Seismic Load Calculation			
Seismic force due to DL & SIDL Span1	=	497.9	KN
Component in Long direction	=	497.9	KN
Component in Trans direction	=	0.000	KN
Total Seismic Force in long. direction	=	**498**	
Seismic force due to DL & SIDL Span2	=	0.0	KN
Component in Long direction	=	0.000	KN
Component in Trans direction	=	0.000	KN
Long. Seismic Load Calculation For Substructure			
Weight of Substructre	=	1305	KN
Seismic Force	=	124	KN
Long. Seismic Load Calculation For Foundation	=		
Weight of Foundation	=	0	KN
Seismic Force	=	0	KN
Trans. Seismic Load Calculation Span 1			
DL of Superstructure	=	2007	KN
Seismic Force	=	198.7	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	198.7	KN
SIDL Including Surfacing	=	597	KN
Seismic Force	=	59.1	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	59.1	KN
CWLL	=	0	KN
Seismic Force	=	0.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	0.0	KN
Trans seismic components for Dead load & SIDL including surfacing	=	0.0	KN
Trans. Seismic Load Calculation Span 2			
DL of Superstructure	=	2034	KN
Seismic Force	=	201.3	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	201.3	KN
SIDL Including Surfacing	=	606	KN
Seismic Force	=	60.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	60.0	KN
CWLL	=	0	KN
Seismic Force	=	0.0	KN
Component of trans Seismic in Long direction	=	0.000	KN
Component of trans Seismic in Trans direction	=	0.0	KN
Trans seismic components for Dead load & SIDL including surfacing	=	0.0	KN
Long. Seismic Load Calculation For Substructure			
Weight of Substructre	=	1305	KN
Seismic Force	=	129	KN
Trans. Seismic Load Calculation For Foundation	=		
Weight of Foundation	=	0	KN
Seismic Force	=	0	KN

Response Reduction Factor (R) Long = **1.0**

Response Reduction Factor (R) Trans = **1.0**

Summary of Loads at Bottom of Foundation :

Summary of Vertical Loads & its effect

Load Items	Load (KN)	e_L (m)	e_T (m)	Load Factor	Factored Load(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
DL of superstructure (Span1)	2007	0.75	0.00	1.00	2007	1505	0
DL of superstructure (Span1)	2034	-0.75	0.00	1.00	2034	-1525	0
SIDL CB(Span1)	325	0.75	0.00	1.00	325	244	0
SIDL CB(Span2)	330	-0.75	0.00	1.00	330	-248	0
SIDL Surfacing (Span1)	272	0.75	0.00	1.00	272	204	0
SIDL Surfacing (Span2)	276	-0.75	0.00	1.00	276	-207	0
FPLL Surfacing (Span1)	0	0.00	0.00	1.00	0	0	0
FPLL Surfacing (Span2)	0	0.00	0.00	1.00	0	0	0
CWLL1 (Span1)	579	0.75	1.60	1.00	579	434	926
CWLL1 (Span2)	723	-0.75	1.60	1.00	723	-542	1156
CWLL2(Span1)	1031	0.75	1.60	1.00	1031	773	1650
CWLL2 (Span2)	0	0.00	0.00	1.00	0	0	0
CWLL3 (Span1)	347	0.75	3.40	1.00	347	261	1181
CWLL3 (Span2)	434	-0.75	3.40	1.00	434	-325	1474
Wt. of Substructure	1305	0.00	0.00	1.00	1305	0	0
Wt. of Foundation	1264	0.00	0.00	1.00	1264	0	0
Wt. of soil on foundation (LWL)	0	0.00	0.00	1.00	0	0	0
Wt. of soil on foundation (HFL)	0	0.00	0.00	1.00	0	0	0
Wt. of Water	0	0.00	0.00	1.00	0	0	0
Buoyancy	-234	0.00	0.00	1.00	-234	0	0
Vertical wind force ('+'/'-')	0	0.00	2.70	1.00	0	0	0

Summary of Horizontal Loads & its effect

Load Items	Longitudinal Load H_L(KN)	TransverseLoad H_T(KN)	Lever arm (m)	Load Factor	Factored Load H_L(KN)	Factored Load H_T(KN)	Factored Moment M_L(KN-m)	Factored Moment M_T(KN-m)
Shear Rating of Span1(Long.)	38		10.4	1.0	38		393	
Shear Rating of Span1(Trans.)		0	10.4	1.0		0		0
Braking force of Span1(Long.)	163		14.2	1.0	163		2309	
Braking force of Span1(Trans.)		0	14.2	1.0		0		0
CF force of Span1(Long.)	0		14.2	1.0	0		1	
CF force of Span1(Trans.)		0	14.2	1.0		0		0
Shear Rating of Span2(Long.)	38		10.4	1.0	38		393	
Shear Rating of Span2(Trans.)		0	10.4	1.0		0		0
Braking force of Span2(Long.)	0		14.2	1.0	0		0	
Braking force of Span2(Trans.)		0	14.2	1.0		0		0
CF force of Span2(Long.)	0		14.2	1.0	0		0	
CF force of Span2(Trans.)		0	14.2	1.0		0		0
SL on DL Superstructure (Long.) - Span1	498		10.4	1.0	498		5191	
SL on DL Superstructure (Trans.) - Span1		199	11.8	1.0		199		2352
SL on SIDL (Trans.) - Span1		59	12.7	1.0		59		751
Trans compo. for Long. Seicmic force - Span1		0	11.8	1.0		0		0
SL on CWLL (Trans.) - Span1		0	13.9	1.0		0		0
SL on DL Superstructure (Long.) - Span2	0		10.4	1.0	0		0	
SL on DL Superstructure (Trans.) - Span2		201	11.8	1.0		201		2384
SL on SIDL (Trans.) - Span2		60	12.7	1.0		60		762
Trans compo. for Long. Seicmic force - Span2		0	11.8	1.0		0		0
SL on CWLL (Trans.) - Span2		0	13.9	1.0		0		0
SL on Substructure (Long.)	124		6.9	1.0	124		856	
SL on Substructure (Trans.)		129	6.9	1.0		129		892
SL on Foundation (Long.)	0		0.9	1.0	0		0	
SL on Foundation (Trans.)		0	0.9	1.0		0		0
Water Load (Long)	2		4.6	1.0	2		11	
Water Load (trans)		5	4.6	1.0		5		23

Load Factor for Different Combination

Maximum case

Case	Load							
	DC	DW	LL/CE/BR/PL	WA	FR	TU	SL	SL
Extreme Event-I	1.25	1.50	0.20	1.00	1.00	0.00	1.00	0.30

Minimum case

Case	Load							
	DC	DW	LL/CE/BR/PL	WA	FR	TU	SL	SL
Extreme Event-I	0.90	0.65	0.20	1.00	1.00	0.00	1.00	0.30

LOAD COMBINATION

LWL CASE

Longitudinal Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1La. EE-I 1.25DC+1.5DW+0.2LL1/CE/BR/PL+0TU+1SL	10163	6452	2559	654	194	683
2La. EE-I 1.25DC+1.5DW+0.2LL2/CE/BR/PL+0TU+1SL	10109	6628	2472	654	194	683
3La. Strength-I 1.25DC+1.5DW+0.2LL3/CE/BR/PL+0TU+1SL	10059	6461	2673	654	194	683

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Lb. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	7155	6463	2559	654	194	683
2Lb. EE-I 0.9DC+0.65DW+0.2LL2/CE/BR/PL+0TU+1SL	7101	6639	2472	654	194	683
3Lb. Strength-I 0.9DC+0.65DW+0.2LL3/CE/BR/PL+0TU+1SL	7050	6472	2673	654	194	683

Transverse Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Tc. EE-I 1.25DC+1.5DW+0.2LL1/CE/BR/PL+0TU+1SL	10163	2220	7558	219	648	684
2Tc. EE-I 1.25DC+1.5DW+0.2LL2/CE/BR/PL+0TU+1SL	10109	2396	7471	219	648	684
3Tc. Strength-I 1.25DC+1.5DW+0.2LL3/CE/BR/PL+0TU+1SL	10059	2228	7672	219	648	684

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Td. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	7155	2231	7558	219	648	684
2Td. EE-I 0.9DC+0.65DW+0.2LL2/CE/BR/PL+0TU+1SL	7101	2407	7471	219	648	684
3Td. Strength-I 0.9DC+0.65DW+0.2LL3/CE/BR/PL+0TU+1SL	7050	2239	7672	219	648	684

HWL CASE

Longitudinal Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Le. EE-I 1.25DC+1.5DW+0.2LL1/CE/BR/PL+0TU+1SL	9929	6463	2582	657	199	686
2Le. EE-I 1.25DC+1.5DW+0.2LL2/CE/BR/PL+0TU+1SL	9875	6639	2495	657	199	686
3Le. Strength-I 1.25DC+1.5DW+0.2LL3/CE/BR/PL+0TU+1SL	9824	6472	2696	657	199	686

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Lf. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6920	6474	2582	657	199	686
2Lf. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6866	6650	2495	657	199	686
3Lf. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6816	6483	2696	657	199	686

Transverse Seismic Case

Maximum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Tg. EE-I 1.25DC+1.5DW+0.2LL1/CE/BR/PL+0TU+1SL	9929	2231	7581	221	653	690
2Tg. EE-I 1.25DC+1.5DW+0.2LL2/CE/BR/PL+0TU+1SL	9875	2407	7494	221	653	690
3Tg. Strength-I 1.25DC+1.5DW+0.2LL3/CE/BR/PL+0TU+1SL	9824	2239	7695	221	653	690

Minimum Reaction Case

LOAD COMBINATION	P (KN)	ML (KN-m)	MT (KN-m)	HL (KN)	HT (KN)	HR (KN)
1Th. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6920	2242	7581	657	199	686
2Th. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6866	2418	7494	657	199	686
3Th. EE-I 0.9DC+0.65DW+0.2LL1/CE/BR/PL+0TU+1SL	6816	2250	7695	657	199	686

DESIGN OF PIER

Diameter of Pier			=	1.80	m
Side of Eq. Square of Pier			=	1.60	m
Pier Length			=	8.7	m
Unit. Wt of Concrete			=	25	KN/m^3
Unit. Wt of Soil			=	20	KN/m^3
Grade of Concrete		$f_{c'}$	=	C30	Mpa
Grade of Reinforcement		f_y	=	420	Mpa
Modulus of Elasticity of Concrete		E_{con}	=	29440.0875	Mpa
Modulus of Elasticity of steel		E_s	=	200000	Mpa
Moment of Inertia of Pier		I_w	=	5.153E+11	mm^4

Calculation of Reinforcement :

Gross c/s area of the Pier			=	2.54	m^2
As min	$0.135A_cf_c'/f_y$	[Eq-5.7-4.2-2]	=	**24538**	mm2
				OK	
As max	0.08Ac	[Eq-5.7-4.2-1]	=	**203575**	mm2
Clear cover			=	50	mm
Minimum Clear Spacing of Main Reinforcement			=	75	mm

Provide	**52 nos**	**25 Dia**	**Reinforcement.**
Provide	**0 nos**	**25 Dia**	**Reinforcement.**
Provide	**0 nos**	**20 Dia**	**Reinforcement.**

As Provided	=	**25513**	mm2
Percentage of Main Reinforcement	=	1.00%	
Spacing of Main Reinforcement	=	100	mm
		OK	

Provide 12 dia. 100 c/c as transverse ties

Area of Transverse steel		=	226	mm2
Minimum Transverse Reinforcement		=	194.832738	
Maximum Center to Center spacing		=	300	mm
			OK	
Minimum clear soacing of Transverse Reinforcement		=	**25**	mm
Dsp		=	**1688**	mm
Ratio of spiral reinforcement to volume of concrete core ρ	= 4Asp/Dsp/SL	=	0.005	**OK**
Area of core		=	2269801	mm^2

Unsupported Length of Pile L_u	8.7	m	KL_u/R	**29.08**	**SLENDER**	
Radius of Gyration, $R=(I/A)^{1/2}$	0.45	m	$EI = E_c I_g/2.5(1+bd)$	6068188	$KN-m^2$	
K	1.50		$Pcr = \pi^2 EI/(KL_u)^2$	349639	KN	
Mo. Magnifier, $\delta = C_m/(1-P_u/\phi_k P_{cr})$			C_m	1.00	ϕ_k	0.75

Strength & Extreme Event Combination

Load Case	***P (KN)***	***M_L (KN-M)***	***M_T (KN-M)***	***M_R (KN-m)***	***Moment Magnifier*** δ Max	***Int. Ratio***	***Remarks***
1a	11351	4151	-1363	4369	4567	0.39	**OK**
2a	10722	6202	3840	7295	7606	0.58	**OK**
3a	10140	4252	5725	7131	7418	0.57	**OK**
4a	8703	359	2845	2868	2966	0.27	**OK**
5a	7942	359	792	869	896	0.18	**OK**
6a	8322	789	1819	1982	2047	0.22	**OK**
7a	10659	3421	-283	3432	3578	0.32	**OK**
8a	10174	5003	3731	6241	6493	0.50	**OK**
9a	9724	3498	5185	6255	6496	0.50	**OK**
1b	8786	4162	-1363	4380	4532	0.36	**OK**
2b	8157	6213	3840	7304	7538	0.58	**OK**
3b	7574	4263	5725	7138	7350	0.57	**OK**
4b	6137	370	2845	2869	2938	0.24	**OK**
5b	5376	370	792	874	892	0.12	**OK**
6b	5757	800	1819	1987	2031	0.18	**OK**
7b	8093	3432	-283	3443	3553	0.29	**OK**
8b	7608	5014	3731	6250	6436	0.49	**OK**
9b	7159	3509	5185	6261	6437	0.49	**OK**
1c	11117	4162	-1340	4373	4566	0.38	**OK**
2c	10488	6213	3863	7316	7621	0.58	**OK**
3c	9906	4262	5748	7156	7437	0.57	**OK**
4c	8469	370	2868	2892	2988	0.26	**OK**
5c	7708	370	815	895	922	0.17	**OK**
6c	8088	800	1841	2008	2071	0.21	**OK**
7c	10425	3431	-260	3441	3584	0.32	**OK**
8c	9940	5013	3754	6263	6510	0.50	**OK**
9c	9490	3509	5208	6280	6516	0.50	**OK**
1d	8552	4173	-1340	4383	4531	0.36	**OK**
2d	7923	6224	3863	7325	7553	0.58	**OK**
3d	7340	4274	5748	7162	7369	0.57	**OK**
4d	5903	381	2868	2893	2960	0.24	**OK**
5d	5142	381	815	899	917	0.12	**OK**
6d	5523	811	1841	2012	2055	0.18	**OK**
7d	7859	3443	-260	3452	3559	0.29	**OK**
8d	7374	5025	3754	6272	6453	0.49	**OK**
9d	6925	3520	5208	6286	6457	0.50	**OK**
1La	10163	6452	2559	6941	7221	0.55	**OK**
2La	10109	6628	2472	7074	7358	0.56	**OK**
3La	10059	6461	2673	6992	7271	0.55	**OK**
1Tc	10163	2220	208	2229	2319	0.26	**OK**
2Tc	10109	2396	208	2405	2501	0.26	**OK**
3Tc	10059	2228	208	2238	2327	0.25	**OK**
1Lb	7155	6463	2559	6951	7146	0.56	**OK**
2Lb	7101	6639	2472	7085	7282	0.57	**OK**
3Lb	7050	6472	2673	7002	7196	0.56	**OK**
1Td	7155	2231	7558	7880	8101	0.66	**OK**

2Td	7101	2407	7471	7850	8068	0.66	**OK**
3Td	7050	2239	7672	7993	8213	0.67	**OK**
1Le	9929	6463	2582	6960	7233	0.55	**OK**
2Le	9875	6639	2495	7093	7370	0.56	**OK**
3Le	9824	6472	2696	7011	7284	0.55	**OK**
1Tg	9929	2231	7581	7902	8213	0.63	**OK**
2Tg	9875	2407	7494	7871	8179	0.62	**OK**
3Tg	9824	2239	7695	8014	8326	0.64	**OK**
1Lf	6920	6474	2582	6970	7159	0.56	**OK**
2Lf	6866	6650	2495	7103	7294	0.57	**OK**
3Lf	6816	6483	2696	7021	7208	0.57	**OK**
1Th	6920	2242	7581	7905	8120	0.67	**OK**
2Th	6866	2418	7494	7875	8086	0.67	**OK**
3Th	6816	2250	7695	8018	8232	0.69	**OK**

Shear Check of Pier Section :

Effective depth of setion	d_e	$D/2+Dr/\pi$	=	1426	mm
Effective Shear depth	d_v		=	1296	mm
Width of Critical section	b_v		=	1800	mm
β			=	1	
θ			=	45	degree
Strength reduction factor ϕ for shear			=	0.9	

Load Case	***H_R (KN)***	***Shear Stress Mpa υ_u***	***Vc (KN)***	***V_S (KN)***	***Vn1 V_c+V_s***	***Vn2 0.25fc'bvdv***	***Vn= ϕMin(Vn1,Vn2)***	***Status***
1a	322	0.15	1061	1231	2292	17496	2063	**Shear R/F Not Reqd.**
2a	322	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
3a	322	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
4a	169	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
5a	169	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
6a	189	0.09	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
7a	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
8a	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
9a	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
1b	322	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
2b	322	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
3b	322	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
4b	169	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
5b	169	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
6b	189	0.09	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
7b	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
8b	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
9b	277	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
1c	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
2c	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
3c	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
4c	175	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
5c	175	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
6c	195	0.09	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
7c	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
8c	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
9c	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
1d	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
2d	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
3d	325	0.15	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
4d	175	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
5d	175	0.08	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
6d	195	0.09	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**

7d	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
8d	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
9d	281	0.13	1061	1231	2292	17496	2292	**Shear R/F Not Reqd.**
1La	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2La	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3La	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Tc	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Tc	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Tc	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Lb	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Lb	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Lb	683	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Td	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Td	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Td	684	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Le	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Le	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Le	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Tg	690	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Tg	690	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Tg	690	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Lf	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Lf	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Lf	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
1Th	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
2Th	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**
3Th	686	0.33	1061	1231	2292	17496	2292	**SF REQD,OK**

Service Combination

Load Case	***P (KN)***	***M_L (KN-M)***	***M_T (KN-M)***	***M_R (KN-m)***	***Moment Magnifier*** δ Max	
10a	8279	3046	-83	3047	3146	
11a	7920	4218	2890	5113	5272	
12a	7587	3103	3967	5037	5187	
10c	8045	3057	-60	3057	3154	
11c	7686	4229	2913	5135	5290	
12c	7353	3114	3990	5061	5207	

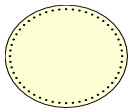
MIDAS OUTPUT

Property

1. Material

Concrete	None	
fc'	30000.00	kN/m²
Ec	29500000.00	kN/m²
Poisson's Ratio	0.20	
Weight Density	25.00	kN/m³
Nonlinear Property	Whitney Rectangular	
Rebar		
fy	420000.00	kN/m²
Es	200000000.00	kN/m²
Nonlinear Property	Bilinear Model	

2. Section



As	0.02552524	m²

Ⅰ. General

Area	2.53179043	m²
Shear Area (y)	2.17293269	m²
Shear Area (z)	2.172935958	m²
Ixx	1.020184581	m⁴
Iyy	0.51009127	m⁴
Izz	0.51009127	m⁴
Centroid (y)	0.899999978	m
Centroid (z)	0.899999978	m

Ⅱ. Section Modulus

Section Modulus (Top)	0.566768078	m³
Section Modulus (Bottom)	0.566768078	m³
Section Modulus (Right)	0.566768078	m³
Section Modulus (Left)	0.566768078	m³

Ⅲ. Principal Properties

Principal Angle	0	°
Iyy'	0.51009127	m⁴
Izz'	0.51009127	m⁴

Ⅳ. Plastic Properties

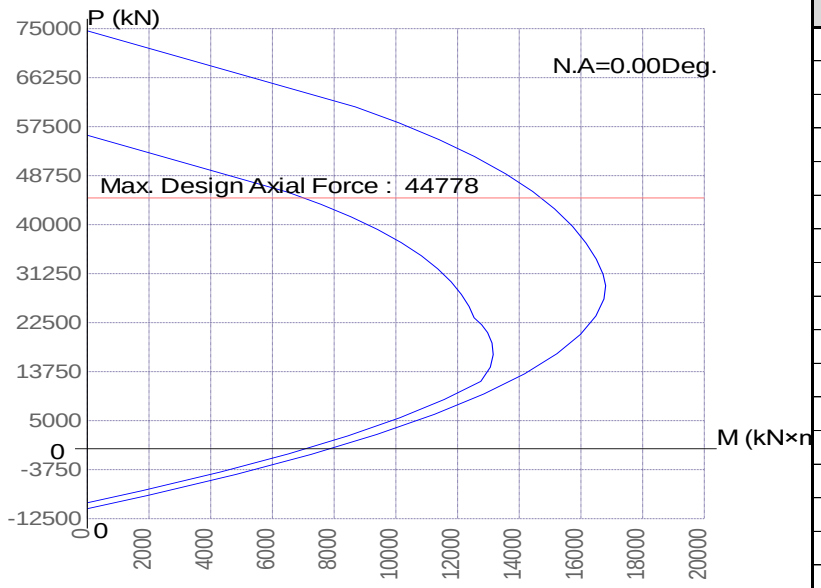
Plastic Modulus (Major axis)	0.964616568	m³
Plastic Modulus (Minor axis)	0.964616568	m³

3. Design Load Combination

No	Name	Pu(kN)	My(kN×m)	Mz(kN×m)	Vy(kN)	Vz(kN)	T(kN×m)
1	1a	11351.00	4567.00	0.00	0.00	0.00	0.00
2	2a	10722.00	7606.00	0.00	0.00	0.00	0.00
3	3a	10140.00	7418.00	0.00	0.00	0.00	0.00
4	4a	8703.00	2966.00	0.00	0.00	0.00	0.00
5	5a	7942.00	896.00	0.00	0.00	0.00	0.00
6	6a	8322.00	2047.00	0.00	0.00	0.00	0.00
7	7a	10659.00	3578.00	0.00	0.00	0.00	0.00
8	8a	10174.00	6493.00	0.00	0.00	0.00	0.00
9	9a	9724.00	6496.00	0.00	0.00	0.00	0.00
10	1b	8786.00	4532.00	0.00	0.00	0.00	0.00
11	2b	8157.00	7538.00	0.00	0.00	0.00	0.00
12	3b	7574.00	7350.00	0.00	0.00	0.00	0.00
13	4b	6137.00	2938.00	0.00	0.00	0.00	0.00
14	5b	5376.00	892.00	0.00	0.00	0.00	0.00
15	6b	5757.00	2031.00	0.00	0.00	0.00	0.00
16	7b	8093.00	3553.00	0.00	0.00	0.00	0.00
17	8b	7608.00	6436.00	0.00	0.00	0.00	0.00
18	9b	7159.00	6437.00	0.00	0.00	0.00	0.00
19	1c	11117.00	4566.00	0.00	0.00	0.00	0.00
20	2c	10488.00	7621.00	0.00	0.00	0.00	0.00
21	3c	9906.00	7437.00	0.00	0.00	0.00	0.00
22	4c	8469.00	2988.00	0.00	0.00	0.00	0.00
23	5c	7708.00	922.00	0.00	0.00	0.00	0.00
24	6c	8088.00	2071.00	0.00	0.00	0.00	0.00
25	7c	10425.00	3584.00	0.00	0.00	0.00	0.00
26	8c	9940.00	6510.00	0.00	0.00	0.00	0.00
27	9c	9490.00	6516.00	0.00	0.00	0.00	0.00
28	1d	8552.00	4531.00	0.00	0.00	0.00	0.00
29	2d	7923.00	7553.00	0.00	0.00	0.00	0.00
30	3d	7340.00	7369.00	0.00	0.00	0.00	0.00
31	4d	5903.00	2960.00	0.00	0.00	0.00	0.00
32	5d	5142.00	917.00	0.00	0.00	0.00	0.00
33	6d	5523.00	2055.00	0.00	0.00	0.00	0.00
34	7d	7859.00	3559.00	0.00	0.00	0.00	0.00
35	8d	7374.00	6453.00	0.00	0.00	0.00	0.00
36	9d	6925.00	6457.00	0.00	0.00	0.00	0.00
37	1La	10163.00	7221.00	0.00	0.00	0.00	0.00
38	2La	10109.00	7358.00	0.00	0.00	0.00	0.00
39	3La	10059.00	7271.00	0.00	0.00	0.00	0.00
40	1Tc	10163.00	2319.00	0.00	0.00	0.00	0.00
41	2Tc	10109.00	2501.00	0.00	0.00	0.00	0.00
42	3Tc	10059.00	2327.00	0.00	0.00	0.00	0.00
43	1Lb	7155.00	7146.00	0.00	0.00	0.00	0.00
44	2Lb	7101.00	7282.00	0.00	0.00	0.00	0.00
45	3Lb	7050.00	7196.00	0.00	0.00	0.00	0.00
46	1Td	7155.00	8101.00	0.00	0.00	0.00	0.00
47	2Td	7101.00	8068.00	0.00	0.00	0.00	0.00
48	3Td	7050.00	8213.00	0.00	0.00	0.00	0.00
49	1Le	9929.00	7233.00	0.00	0.00	0.00	0.00
50	2Le	9875.00	7370.00	0.00	0.00	0.00	0.00
51	3Le	9824.00	7284.00	0.00	0.00	0.00	0.00
52	1Tg	9929.00	8213.00	0.00	0.00	0.00	0.00
53	2Tg	9875.00	8179.00	0.00	0.00	0.00	0.00
54	3Tg	9824.00	8326.00	0.00	0.00	0.00	0.00
55	1Lf	6920.00	7159.00	0.00	0.00	0.00	0.00
56	2Lf	6866.00	7294.00	0.00	0.00	0.00	0.00
57	3Lf	6816.00	7208.00	0.00	0.00	0.00	0.00
58	1Th	6920.00	8120.00	0.00	0.00	0.00	0.00
59	2Th	6866.00	8086.00	0.00	0.00	0.00	0.00
60	3Th	6816.00	8232.00	0.00	0.00	0.00	0.00

P-M Curve

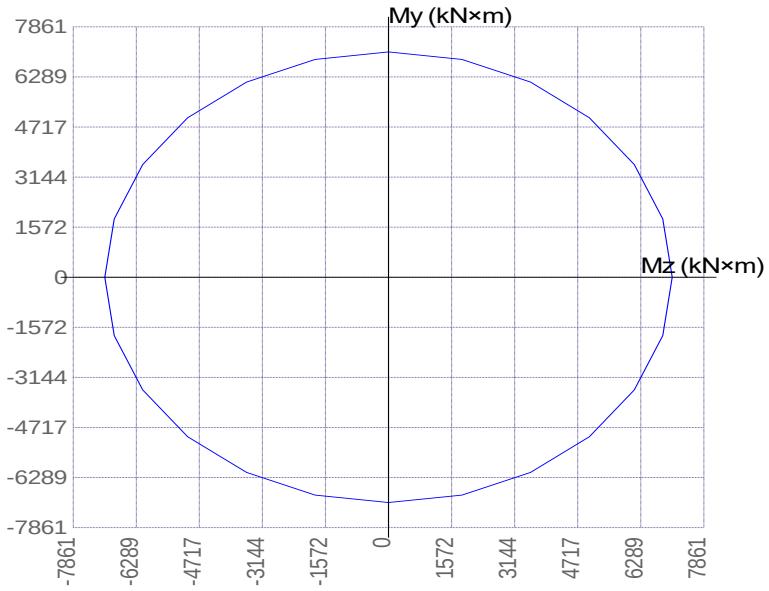
Mode : Angle = 0.00 °



Pu(kN)	Mn(kN×m)
55972.772	0.000
45779.248	6517.774
43645.046	7572.725
41403.800	8549.715
39126.868	9414.593
36808.629	10174.285
34457.322	10828.168
32124.389	11359.932
29801.253	11795.494
27550.225	12125.673
25387.700	12374.221
23375.081	12539.107
22146.398	12780.320
20730.410	12976.507
18863.891	13120.220
16800.282	13156.811
14557.641	13065.360
12030.353	12756.599
8793.130	11569.829
5516.523	10125.882
2287.975	8444.421
-916.266	6526.515
-4131.849	4334.486
-7504.153	1767.817
-9648.541	0.000

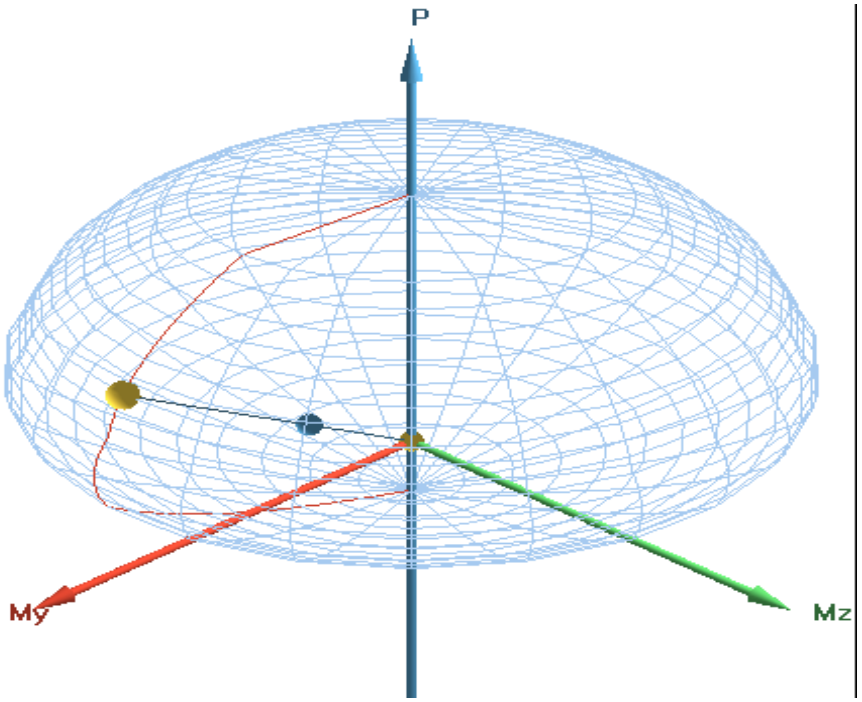
Mx-My Curve

Mode : Axial Force, P = 0.00 kN



My(kN×m)	Mz(kN×m)
7074.948	0.000
6839.123	1832.199
6129.536	3539.551
5008.926	5008.926
3539.551	6129.536
1832.199	6839.123
0.000	7074.948
-1832.199	6839.123
-3539.551	6129.536
-5008.926	5008.926
-6129.536	3539.551
-6839.123	1832.199
-7074.948	0.000
-6839.123	-1832.199
-6129.536	-3539.551
-5008.926	-5008.926
-3539.551	-6129.536
-1832.199	-6839.123
0.000	-7074.948
1832.199	-6839.123
3539.551	-6129.536
5008.926	-5008.926
6129.536	-3539.551
6839.123	-1832.199
7074.948	0.000

P-Mx-My Curve



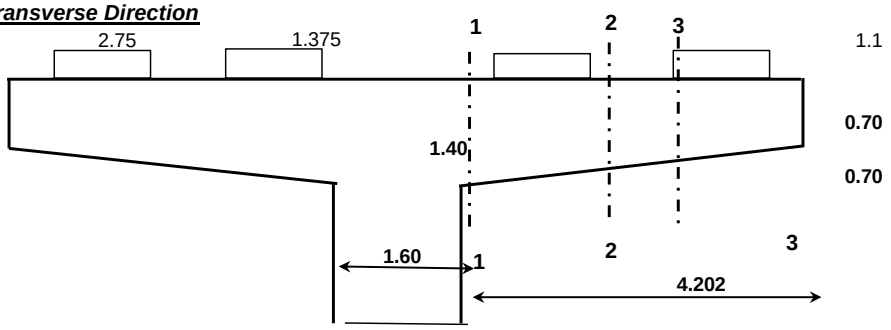
Load Comb Name	Ratio		
	P-My	P-Mz	P-My/Mz
1a	0.385	0.000	0.385
2a	0.579	0.000	0.579
3a	0.565	0.000	0.565
4a	0.265	0.000	0.265
5a	0.177	0.000	0.177
6a	0.215	0.000	0.215
7a	0.322	0.000	0.322
8a	0.499	0.000	0.499
9a	0.497	0.000	0.497
1b	0.363	0.000	0.363
2b	0.579	0.000	0.579
3b	0.569	0.000	0.569
4b	0.238	0.000	0.238
5b	0.122	0.000	0.122
6b	0.179	0.000	0.179
7b	0.293	0.000	0.293
8b	0.491	0.000	0.491
9b	0.493	0.000	0.493
1c	0.383	0.000	0.383
2c	0.580	0.000	0.580
3c	0.566	0.000	0.566
4c	0.263	0.000	0.263
5c	0.172	0.000	0.172
6c	0.212	0.000	0.212
7c	0.319	0.000	0.319
8c	0.499	0.000	0.499
9c	0.497	0.000	0.497
1d	0.362	0.000	0.362
2d	0.583	0.000	0.583

3d	0.573	0.000	0.573
4d	0.238	0.000	0.238
5d	0.119	0.000	0.119
6d	0.178	0.000	0.178
7d	0.292	0.000	0.292
8d	0.493	0.000	0.493
9d	0.497	0.000	0.497
1La	0.550	0.000	0.550
2La	0.560	0.000	0.560
3La	0.554	0.000	0.554
1Tc	0.255	0.000	0.255
2Tc	0.261	0.000	0.261
3Tc	0.253	0.000	0.253
1Lb	0.555	0.000	0.555
2Lb	0.568	0.000	0.568
3Lb	0.561	0.000	0.561
1Td	0.656	0.000	0.656
2Td	0.655	0.000	0.655
3Td	0.674	0.000	0.674
1Le	0.551	0.000	0.551
2Le	0.561	0.000	0.561
3Le	0.554	0.000	0.554
1Tg	0.626	0.000	0.626
2Tg	0.623	0.000	0.623
3Tg	0.635	0.000	0.635
1Lf	0.559	0.000	0.559
2Lf	0.572	0.000	0.572
3Lf	0.565	0.000	0.565
1Th	0.669	0.000	0.669
2Th	0.667	0.000	0.667
3Th	0.687	0.000	0.687

DESIGN OF PIER CAP ***(Maximum Torsion Case)***

Diameter of pier	=	1.8	m
Length of Pier	=	1.60	m
Width of Pier	=	1.60	m
Depth of Pier Cap at Edge	=	0.7	m
Depth of Pier Cap at Junction	=	1.4	m
Width of Pier Cap of Span 1 Side	=	1.25	m
Width of Pier Cap of Span 2 Side	=	1.25	m
Total Width of Pier Cap	=	2.5	m
Length of Pier Cap	=	10	
No. of Bearings (Span 1)	=	2	Nos
Long. Distance of Span 1 Bearings from c/l of Pier	=	0.75	m
Width of Pier Cap Wall of Span 1 Side	=	0.00	m
Height of Pier Cap Wall of Span 1 Side	=	0.00	m
Length of Pier Cap Wall of Span 1 Side	=	0.00	m
Ht of Brg + Pedestal (Span 1)	=	0.3	m
Dynamic Factor (Span 1)	=	1.33	
Multiple Presence Factor(Span 1)	=	1.20	
	=		
No. of Bearings (Span 2)	=	2	Nos
Long. Distance of Span 2 Bearings from c/l of Pier	=	0.75	m
Width of Pier Cap Wall of Span 2 Side	=	0.70	m
Height of Pier Cap Wall of Span 2 Side	=	0.50	m
Length of Pier Cap Wall of Span 2 Side	=	10.00	m
Ht of Brg + Pedestal (Span 2)	=	0.3	m
Dynamic Factor (Span 2)	=	1.33	
Multiple Presence Factor(Span 2)	=	1.00	
Dynamic Factor For Fatigue Vehicle	=	1.15	
Pedestal Size (tentative)	=	0.7	
Distance of Pier Cap edge from Pedestal face	=	0.5	
Unit. Wt of Concrete	=	25	KN/m^3

Design Of Pier Cap In Transverse Direction



Distance of cg of brg. from face of equivalent square (a)	=	3.3	m
Effective Depth at Junction (de)	=	1.318	m
a/d_e	=	2.52	

Design as Cantilever

Dead load Calculation Of Pier Cap

Item	Sec 1-1	Sec 2-2	Sec 3-3	Unit
Depth of Pier Cap at Edge	0.70	0.70	0.70	m
Depth of Pier Cap at Junction	1.40	1.18	0.88	m
Span1				
Length of Cantilever (Span 1 Side)	4.20	2.88	1.10	m
Self weight of Pier Cap (Span 1 Side)	**137.9**	**84.7**	**27.2**	KN
Long. Ecc.	0.63	0.6	0.6	m
Trans. Ecc.	1.868	1.319	0.529	m
Self weight of Pier Cap Wall (Span 1 Side)	**0.0**	**0.0**	**0.0**	KN
Long. Ecc.	0.75	0.75	0.75	m
Trans. Ecc.	2.1	1.4	0.6	m
Total (Span 1)				
Total Weight of Pier Cap	**137.9**	**84.7**	**27.2**	m
Moment	**257.5**	**111.8**	**14.4**	KN-m
Torsion	**86.2**	**53.0**	**17.0**	KN-m
Span2				

Length of Cantilever (Span 2 Side)	4.20	2.88	1.10	m
Self weight of Pier Cap (Span 2 Side)	**137.89**	**84.75**	**27.21**	KN
Long. Ecc.	-0.63	-0.63	-0.63	m
Trans. Ecc.	1.87	1.32	0.53	m
Self weight of Pier Cap Wall (Span 2 Side)	**36.77**	**25.24**	**9.63**	KN
Long. Ecc.	-0.75	-0.75	-0.75	m
Trans. Ecc.	2.10	1.44	0.55	m
Total (Span2)				
Total Weight of Pier Cap	**174.66**	**109.99**	**36.84**	m
Moment	**334.81**	**148.22**	**19.68**	KNm
Torsion	**-113.76**	**-71.90**	**-24.23**	KNm

Load Factor	***Strength-I***	***Strength-III***	***Strength-V***	***Service-I***	***Fatigue***
DC(Max)	1.25	1.25	1.25	1.0	0.0
DW (Max)	1.5	1.5	1.5	1.0	0.0
DC(Min)	0.9	0.9	0.9	1.0	0.0
DW (Min)	0.65	0.65	0.65	1.0	0.0
LL/CF/BR/PL	1.75	0	1.35	1.0	1.50
WS	0	1.4	0.4	0.3	0.0
FR	1	1	1	1.0	0.0
TU	0.5	0.5	0.5	1.0	0.0

Summary of Unfactored Load on Bearings Section-1

Load Items	**SPAN 1**				**SPAN 2**			
	Bearing 1	**Bearing 2**	**Bearing 3**	**Bearing 4**	**Bearing 1**	**Bearing 2**	**Bearing 3**	**Bearing 4**
Trans. Distance from face	3.33	0.58	0.00	0.00	3.33	0.58	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	534	0.00	0.00	496	534	0.00	0.00
SIDL CB	135	61	0.00	0.00	135	61	0.00	0.00
SIDL WC	64	79	0.00	0.00	64	79	0.00	0.00
FPLL	0	0	0.00	0.00	0	0	0.00	0.00
CWLL (With Out DA)	209	96	0.00	0.00	0	0	0.00	0.00
CW Lane Load	81	45	0.00	0.00	0.00	0.00	0.00	0.00
CF	0	0	0.00	0.00	0	0	0.00	0.00
Verticle Wind	36	36	0.00	0.00	28	28	0.00	0.00
Trans. Wind Laod	0	0	0.00	0.00	0	0	0.00	0.00
Friction Force (Horz. Forces)	9.43	9.43	0.00	0.00	9.43	9.43	0.00	0.00
Long. Wind (Horz. Forces)	5.23	5.23	0.00	0.00	5.23	5.23	0.00	0.00
Braking (Horz. Forces)	0	0	0.00	0.00	0	0	0.00	0.00
Fatigue LL	203	95	0.00	0.00	0	0	0.00	0.00

Summary of Unfactored Moment, Shear & Torsion-Section -1

Load Items	**SPAN 1**			**SPAN 2**		
	Moment	**Shear**	**Torsion**	**Moment**	**Shear**	**Torsion**
Dead Load	1959	1030	773	1959	1030	-773
SIDL CB	484	196	147	484	196	-147
SIDL WC	259	143	107	259	143	-107
FPLL	0	0	0	0	0	0
CWLL (With DA)	1553	638	478	0	0	0
CF	0	0	0	0	0	0
Verticle Wind	141	72	54	109	56	-42
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			19			28
Long. Wind (Horz. Forces)			10			16
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	258	138	86	335	175	-114
Fatigue load	840	343	257	0	0	0

Summary of factored Moment, Shear & Torsion-Section -1

Combination	***Details***	**Moment KN-m**	**Shear KN**	**Torsion KN-m**
1	Strength I Maximum load effect	10342	5001	826
2	Strength I Minimum load effect	9267	4451	1239
3	Strength III Maximum load effect	7974	4064	43
4	Strength III Minimum load effect	6899	3513	456
5	Strength V Maximum load effect	9820	4797	650
6	Strength V Minimum load effect	8746	4247	1063
7	Service I	7624	3727	510

8	Fatigue	1260	514	386

Summary of Unfactored Load on Bearings Section-2

Load Items	SPAN 1				SPAN 2			
	Bearing 1	Bearing 2	Bearing 3	Bearing 4	Bearing 1	Bearing 2	Bearing 3	Bearing 4
Trans. Distance from face	2.01	0.00	0.00	0.00	2.01	0.00	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	0	0	0	496	0	0	0
SIDL CB	135	0	0	0	135	0	0	0
SIDL WC	64	0	0	0	64	0	0	0
FPLL	0	0	0	0	0	0	0	0
CWLL (With Out DA)	209	0	0	0	0	0	0	0
CW Lane Load	81	0	0	0	0	0	0	0
CF	0	0	0	0	0	0	0	0
Verticle Wind	36	0	0	0	28	0	0	0
Trans. Wind Laod	0	0	0	0	0	0	0	0
Friction Force (Horz. Forces)	9	0	0	0	9	0	0	0
Long. Wind (Horz. Forces)	5	0	0	0	5	0	0	0
Braking (Horz. Forces)	0	0	0	0	0	0	0	0
Fatigue LL	203	0	0	0	0	0	0	0

Summary of Unfactored Moment, Shear & Torsion-Section -2

Load Items	SPAN 1			SPAN 2		
	Moment	Shear	Torsion	Moment	Shear	Torsion
Dead Load	997	496	372	997	496	-372
SIDL CB	271	135	101	271	135	-101
SIDL WC	129	64	48	129	64	-48
FPLL	0	0	0	0	0	0
CWLL (With DA)	866	431	323	0	0	0
CF	0	0	0	0	0	0
Verticle Wind	72	36	27	56	28	-21
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			8			13
Long. Wind (Horz. Forces)			5			7
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	112	85	53	148	110	-72
Fatigue load	469	233	175	0	0	0

Summary of factored Moment, Shear & Torsion-Section -2

Combination	*Details*	Moment KN-m	Shear KN	Torsion KN-m
1	Strength I Maximum load effect	5395	2767	552
2	Strength I Minimum load effect	4842	2492	759
3	Strength III Maximum load effect	4061	2103	12
4	Strength III Minimum load effect	3508	1827	219
5	Strength V Maximum load effect	5101	2620	430
6	Strength V Minimum load effect	4548	2345	637
7	Service I	3957	2035	331
8	Fatigue	704	350	263

Summary of Unfactored Load on Bearings Section-3

Load Items	SPAN 1				SPAN 2			
	Bearing 1	Bearing 2	Bearing 3	Bearing 4	Bearing 1	Bearing 2	Bearing 3	Bearing 4
Trans. Distance from face	0.35	0.00	0.00	0.00	0.35	0.00	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	0	0	0	496	0	0	0
SIDL CB	135	0	0	0	135	0	0	0
SIDL WC	64	0	0	0	64	0	0	0
FPLL	0	0	0	0	0	0	0	0
CWLL (With Out DA)	209	0	0	0	0	0	0	0
CW Lane Load	81	0	0	0	0	0	0	0
CF	0	0	0	0	0	0	0	0
Verticle Wind	36	0	0	0	28	0	0	0
Trans. Wind Laod	0	0	0	0	0	0	0	0
Friction Force (Horz. Forces)	9	0	0	0	9	0	0	0
Long. Wind (Horz. Forces)	5	0	0	0	5	0	0	0
Braking (Horz. Forces)	0	0	0	0	0	0	0	0
Fatigue LL	203	0	0	0	0	0	0	0

Summary of Unfactored Moment, Shear & Torsion-Section -3

Load Items	SPAN 1			SPAN 2		
	Moment	Shear	Torsion	Moment	Shear	Torsion
Dead Load	174	496	372	174	496	-372
SIDL CB	47	135	101	47	135	-101
SIDL WC	22	64	48	22	64	-48
FPLL	0	0	0	0	0	0
CWLL (With Out DA)	151	431	323	0	0	0
CF	0	0	0	0	0	0
Verticle Wind	13	36	27	10	28	-21
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			7			12
Long. Wind (Horz. Forces)			4			6
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	14	27	17	20	37	-24
Fatigue load	82	233	175	0	0	0

Summary of factored Moment, Shear & Torsion-Section -3

Combination	*Details*	Moment KN-m	Shear KN	Torsion KN-m
1	Strength I Maximum load effect	926	2603	566
2	Strength I Minimum load effect	829	2328	772
3	Strength III Maximum load effect	693	1939	23
4	Strength III Minimum load effect	597	1664	230
5	Strength V Maximum load effect	874	2457	443
6	Strength V Minimum load effect	778	2181	649
7	Service I	678	1904	339
8	Fatigue	123	350	263

Design of Pier Cap for Bending & Torsion in Section 1

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.08747	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	1318	mm
Width of Section (b)		=	2500	mm
Main Bar Dia. Provided		=	32	mm
Shear R/F. Provided		=	16	mm
Clear Cover Provided		=	50	mm
Maximum moment in section		=	**10342**	KN-m

Area of steel is obtained by solving the quadratic equation:- $\mathbf{As = [\, B - \sqrt{(B^2 - 4 * M_n * A)}\,] / (\, 2 * A\,)}$

Where, B = $d*f_y$ 553560

A = $(f_y^2)/(1.7 * f_c' * b)$ 1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **219.64** cm^2

Provide						=		
	23	Nos.	**32**	Dia.	(at tension face)		**257.23**	cm^2
	23	Nos.	**20**	Dia.				**OK**

Spacing of Reinforcement = 106 mm

Neutral axis a = $A_s f_y/0.85 f_c b$ = 169.5 mm

c = a/β1 = 199.4 mm

ε_c = 0.003

ε_t = $(d-c)*\varepsilon_c/c$ = 0.017 **Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.312908808 Mpa
Gross Moment of Inertia	I_g		=	5.71667E+11 mm^4
Distance from tension face to Cg	y_t		=	700 mm
Cracking Moment	Mcr		=	4338.88 KN-m
1.2 times M_{cr}			=	5206.650632 KN-m
Reinforcement required for 1.2 Cracking moment			=	**107.39** cm^2
1.33 times M_u			=	13754.4 KN-m
Reinforcement required for 1.33 M_u			=	**298.32** cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**20.19** cm^2
So Reqiured area of Reinforcement			=	**219.64** cm^2 **OK**
As required from Cpmpression side			=	**20.19** cm^2

Provide						=		
	23	Nos.	**16**	Dia.	(at Comp. face)		**46.24**	cm^2
	0	Nos.	**0**	Dia.				**OK**

Moment In service stage

Net Bending moment in Service I = **7624** KN-m/m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c))$ = 1.069

γe 0.75

Modular ratio (n)			=	6.00
Position of nutural axis from comp face (x)			=	346.366 mm
MOA of compression area			=	149962091.3 mm^2
MOA of Tension area			=	149962091 mm^2
Diff			=	0.000
MOI of cracked section			=	1.80336E+11 mm^4
Stress in tension reinf	f_{ss}		=	**246.5** Mpa
Stress in Concrete	f_{cs}		=	**14.6** Mpa **OK**
Required Spacing of reinforcement s			=	**350** mm **OK**

Side face Reinforcement

Area of skin reinforcement A_{sk} = **5.58** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	5996 +1,259.793	=	7256 KN-m
Minimum Moment	5996 +0.003	=	5996 KN-m

Modular ratio (n)						=	6.00	
Position of nutural axis from comp face (x)						=	346.366	mm
MOA of compression area						=	149962091.3	mm^2
MOA of Tension area						=	149962091	mm^2
Diff						=	0.000	
MOI of cracked section						=	1.80336E+11	mm^4
Stress in tension reinf(For Max. Moment)		f_{ssMax}				=	**234.6**	Mpa
Stress in tension reinf(For Min. Moment)		f_{ssMin}				=	**193.8**	Mpa
Stress range in the R/F						=	**40.73**	Mpa
		f_f=166-0.33f_{min}				=	**102**	Mpa
							OK	

Check For Shear & Torsion for Section -1

A_{cp}						=	3500000	mm^2
P_c						=	7800	mm
A_0						=	3061056	mm^2
P_h						=	7336	mm
Factored Torsional Moment				T_u		=	**1239**	KN-m
Torsional Cracking Moment		$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$		T_{cr}		=	2821	KN-m
				$0.25\phi T_{cr}$		=	635	KN-m
							Torsion Check Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : **16** **dia.** **2 L** **Stirrups** **100** **Spacing**

A_t						=	201.1	mm^2
Nominal torsional resistance		$2A_0A_tf_y\cot\theta/s$		T_n		=	**5170**	KN-m
Factored torsional resistance				$T_r=\phi T_n$		=	**4653**	KN-m
							OK	

Longitudianl Reinforcement

A_sf_y						=	12746	KN
$M_u/\phi d_v$						=	9317	KN
$\cot\theta sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$						=	2802	KN
							OK	

Maximum shear force on the section						=	**5001**	KN
Corresponding Torsion				T_u		=	**826**	KN-m
Equivalent shear force		$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$				=	5080	KN
Total facrored shear force				V_u		=	**5080**	KN
Effective Shear depth				d_v		=	1233	mm
Width of critical section				b_v		=	2500	mm
β						=	2	
θ						=	45	degree
Strength reduction factor ϕ for shear & torsion						=	0.9	
Shear stress on concrete				υ_u		=	1.83	Mpa
Shear strength provided by concrete		$0.083*\beta*\sqrt{f_c'}*b_v*d_v$		V_c		=	2803	KN
							SF.REQD	
Capacity of shear steel	Vs	=	$A_v.\ f_y.d_v.\cot\theta/s$				5598	KN
Spacing of Stirrups				s		=	100	mm

Provide	**16**	dia.	**2 L**	Stirrups	**100**	Spacing	A_{sw} provided	=	**1081**	mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing				

Max. spacing of transverse Reinforcement

If υ_u <0.125f_c' then S_{max}=0.8d_v<=600	=	600	mm
If υ_u >=0.125f_c' then S_{max}=0.4d_v<=300		**OK**	

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	8401	KN
	V_n =0.25fc'b_vd_v	=	23124	KN
Factored Shear Resistance	V_r =ϕVn	=	**7561**	KN
			OK	

Check for minimum Shear Reinforcement

A_v>0.083$\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2
		OK	

Design of Pier Cap for Bending & Torsion Section 2

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa	
Grade of Reinforcement	f_y	=	C420	Mpa	
Modulus of Elasticity of Concrete	E_{con}	=	29440.08747	Mpa	
Modulus of elasticity of steel	E_s	=	200000	Mpa	
deff provd.	d	=	1098	mm	
Width of Section (b)		=	2500	mm	
Main Bar Dia. Provided		=	32	mm	
Shear R/F. Provided		=	16	mm	
Clear Cover Provided		=	50	mm	
Maximum moment in section		=	**5395**	KN-m	

Area of steel is obtained by solving the quadratic equation:- **As =[B -$\sqrt{}$(B^2 - 4*M_n * A)]/(2 * A)**

Where, B = $d*f_y$ = 461353

A = $(f_y^2)/(1.7 * f_c' * b)$ = 1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **135.44** cm^2

Provide **23** Nos. **32** Dia. (at tension face) = **257.23** cm^2 **OK**

23 Nos. **20** Dia.

Spacing of Reinforcement = 106 mm

Neutral axis a = $A_s f_y/0.85 f_c b$ = 169.5 mm

c = a/β1 = 199.4 mm

ε_c = 0.003

ε_t = $(d-c)*\varepsilon_c/c$ = 0.014 **Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.312908808	Mpa
Gross Moment of Inertia	I_g		=	3.42698E+11	mm^4
Distance from tension face to Cg	y_t		=	590	mm
Cracking Moment	Mcr		=	3085	KN-m
1.2 times M_{cr}			=	3702	KN-m
Reinforcement required for 1.2 Cracking moment			=	**91.67**	cm^2
1.33 times M_u			=	7175.9	KN-m
Reinforcement required for 1.33 M_u			=	**182.85**	cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**18.04**	cm^2
So Reqiured area of Reinforcement			=	**135.44**	cm^2 **OK**

Moment In service stage

Net Bending moment in Service I = **3957** KN-m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c))) =$ 1.082

γe 0.75

Modular ratio (n)		=	6.00	
Position of nutural axis from comp face (x)		=	311.681	mm
MOA of compression area		=	121431366.5	mm^2
MOA of Tension area		=	121431366	mm^2
Diff		=	0.000	
MOI of cracked section		=	1.20771E+11	mm^4
Stress in tension reinf	f_{ss}	=	**154.7**	Mpa
Stress in Concrete	f_{cs}	=	**10.2**	Mpa
			OK	
Required Spacing of reinforcement s		=	**551**	mm
			OK	

Side face Reinforcement

Area of skin reinforcement A_{sk} = **3.38** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	3053 +703.643	=	3757	KN-m
Minimum Moment	3053 +0.003	=	3053	KN-m

Modular ratio (n)		=	6.00	
Position of nutural axis from comp face (x)		=	311.681	mm
MOA of compression area		=	121431366.5	mm^2
MOA of Tension area		=	121431366	mm^2
Diff		=	0.000	
MOI of cracked section		=	1.20771E+11	mm^4
Stress in tension reinf(For Max. Moment)	f_{ssMax}	=	**146.8**	Mpa
Stress in tension reinf(For Min. Moment)	f_{ssMin}	=	**119.3**	Mpa
Stress range in the R/F		=	**27.50**	Mpa
	$f_f=166-0.33f_{min}$	=	**127**	Mpa
			OK	

Check For Shear & Torsion for Section -2

A_{cp}		=	2951146	mm^2
P_c		=	7361	mm
A_0		=	2537669	mm^2
P_h		=	6897	mm
Factored Torsional Moment	T_u	=	**759**	KN-m
Torsional Cracking Moment	$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$ T_{cr}	=	2126	KN-m
	$0.25\phi T_{cr}$	=	478	KN-m
			Torsion Check Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : 16 dia. 2 L Stirrups 100 Spacing

A_t			=	201.1	mm^2	
Nominal torsional resistance	$2A_0A_tf_y\cot\theta/s$	T_n	=	**4286**	KN-m	
Factored torsional resistance		$T_r=\phi T_n$	=	**3857**	KN-m	
				OK		
Longitudianl Reinforcement						
A_sf_y			=	10804	KN	
$M_u/\phi d_v$			=	5914	KN	
$\cot\theta\,sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$			=	860	KN	
				OK		
Maximum shear force on the section			=	**2767**	KN	
Corresponding Torsion		T_u	=	**552**	KN-m	
Equivalent shear force	$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$		=	2848	KN	
Total facrored shear force		V_u	=	**2848**	KN	
Effective Shear depth		d_v	=	1014	mm	
Width of critical section		b_v	=	2500	mm	
β			=	2		
θ			=	45	degree	
Strength reduction factor ϕ for shear & torsion			=	0.9		
Shear stress on concrete		υ_u	=	1.25	Mpa	
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$	V_c	=	2304	KN	
				SF.REQD		
Capacity of shear steel	Vs = $A_v.\,f_y.d_v.\cot\theta/s$			4601	KN	
Spacing of Stirrups		s	=	100	mm	

Provide							A_{sw} provided	=		
	16	dia.	**2 L**	Stirrups	**100**	Spacing			**1081**	mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing				

Max. spacing of transverse Reinforcement

If υ_u <0.125f_c' then S_{max}=0.8d_v<=600	=	600	mm	
If υ_u >=0.125f_c' then S_{max}=0.4d_v<=300		**OK**		

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	6906	KN	
	$V_n = 0.25fc'b_vd_v$	=	19007	KN	
Factored Shear Resistance	$V_r = \phi Vn$	=	**6215**	KN	
			OK		

Check for minimum Shear Reinforcement

$A_v>0.083\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2	
		OK		

Design of Pier Cap for Bending & Torsion Section 3

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.08747	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	801	mm
Width of Section (b)		=	2500	mm
Main Bar Dia. Provided		=	32	mm
Shear R/F. Provided		=	16	mm
Clear Cover Provided		=	50	mm
Maximum moment in section		=	**926**	KN-m

Area of steel is obtained by solving the quadratic equation:- **As =[B -$\sqrt{(B^2 - 4*M_n * A)}$]/(2 * A)**

Where,	B	=	$d*f_y$	336516
	A	=	$(f_y^2)/(1.7 * f_c' * b)$	1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **30.96** cm^2

Provide **23** Nos. **32** Dia. (at tension face) = **257.23** cm^2

23 Nos. **20** Dia. **OK**

Spacing of Reinforcement = 106 mm

Neutral axis a = $A_s f_y / 0.85 f_c b$ = 169.5 mm

c = a/β1 = 199.4 mm

ε_c = 0.003

ε_t = $(d-c)*\varepsilon_c/c$ = 0.009 **Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.312908808 Mpa
Gross Moment of Inertia	I_g		=	1.43542E+11 mm^4
Distance from tension face to Cg	y_t		=	442 mm
Cracking Moment	Mcr		=	1727 KN-m
1.2 times M_{cr}			=	2072 KN-m
Reinforcement required for 1.2 Cracking moment			=	**70.46** cm^2
1.33 times M_u			=	1231.3 KN-m
Reinforcement required for 1.33 M_u			=	**41.36** cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**14.68** cm^2
So Reqiured area of Reinforcement			=	**41.36** cm^2 **OK**

Moment In service stage

Net Bending moment in Service I = **678** KN-m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c)))$ = 1.111

γe 0.75

Modular ratio (n)		=	6.00
Position of nutural axis from comp face (x)		=	258.796 mm
MOA of compression area		=	83719177.84 mm^2
MOA of Tension area		=	83719178 mm^2
Diff		=	0.000
MOI of cracked section		=	59856156416 mm^4
Stress in tension reinf	f_{ss}	=	**36.9** Mpa
Stress in Concrete	f_{cs}	=	**2.9** Mpa **OK**
Required Spacing of reinforcement s		=	**2252** mm **OK**

Side face Reinforcement

Area of skin reinforcement A_{sk} = **0.41** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	521 +122.563	=	643 KN-m
Minimum Moment	521 +0.003	=	521 KN-m

Modular ratio (n)		=	6.00
Position of nutural axis from comp face (x)		=	258.796 mm
MOA of compression area		=	83719177.84 mm^2
MOA of Tension area		=	83719178 mm^2
Diff		=	0.000
MOI of cracked section		=	59856156416 mm^4
Stress in tension reinf(For Max. Moment)	f_{ssMax}	=	**35.0** Mpa
Stress in tension reinf(For Min. Moment)	f_{ssMin}	=	**28.3** Mpa

Stress range in the R/F				=	**6.66**	Mpa
	$f_f=166-0.33f_{min}$			=	**157**	Mpa
					OK	

Check For Shear & Torsion for Section -3

A_{cp}				=	2208072	mm^2
P_c				=	6766	mm
A_0				=	1829073	mm^2
P_h				=	6302	mm
Factored Torsional Moment		T_u		=	**772**	KN-m
Torsional Cracking Moment	$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$	T_{cr}		=	1294	KN-m
		$0.25\phi T_{cr}$		=	291	KN-m
					Torsion Check Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : 16 dia. 2 L Stirrups 100 Spacing

A_t				=	201.1	mm^2
Nominal torsional resistance	$2A_0A_tf_y\cot\theta/s$	T_n		=	**3089**	KN-m
Factored torsional resistance		$T_r=\phi T_n$		=	**2780**	KN-m
					OK	

Longitudianl Reinforcement

A_sf_y				=	10804	KN
$M_u/\phi d_v$				=	1426	KN
$\cot\theta sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$				=	1347	KN
					OK	

Maximum shear force on the section				=	**2603**	KN
Corresponding Torsion		T_u		=	**566**	KN-m
Equivalent shear force	$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$			=	2747	KN
Total facrored shear force		V_u		=	**2747**	KN
Effective Shear depth		d_v		=	721	mm
Width of critical section		b_v		=	2500	mm
β				=	2	
θ				=	45	degree
Strength reduction factor ϕ for shear & torsion				=	0.9	
Shear stress on concrete		υ_u		=	1.69	Mpa
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$	V_c		=	1639	KN
					SF.REQD	
Capacity of shear steel	Vs =	$A_v.\ f_y.d_v.\cot\theta/s$			3273	KN
Spacing of Stirrups		s		=	100	mm

Provide	**16**	dia.	**2 L**	Stirrups	**100**	Spacing	A_{sw} provided	=	**1081** mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing			

Max. spacing of transverse Reinforcement

If $\upsilon_u<0.125f_c'$ then $S_{max}=0.8d_v<=600$	=	576.8847474	mm
If $\upsilon_u>=0.125f_c'$ then $S_{max}=0.4d_v<=300$		**OK**	

Nomninal Shear Resistance	$V_n=V_c+V_s$	=	4912	KN
	$V_n=0.25fc'b_vd_v$	=	13521	KN
Factored Shear Resistance	$V_r=\phi Vn$	=	**4421**	KN
			OK	

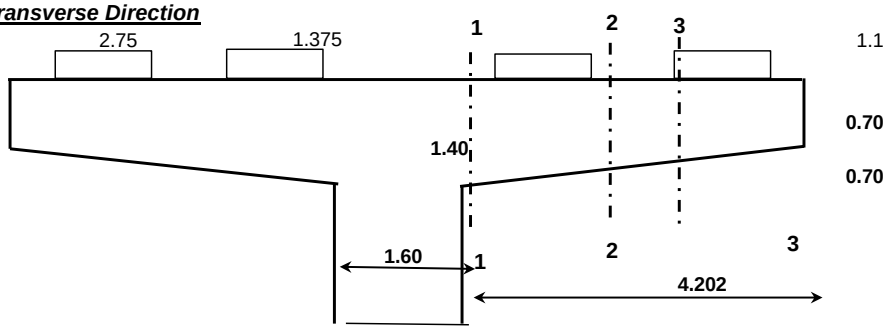
Check for minimum Shear Reinforcement

$A_v>0.083\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2
		OK	

DESIGN OF PIER CAP ***(Maximum Moment Case)***

Diameter of pier	=	1.8	m
Length of Pier	=	1.60	m
Width of Pier	=	1.60	m
Depth of Pier Cap at Edge	=	0.7	m
Depth of Pier Cap at Junction	=	1.4	m
Width of Pier Cap of Span 1 Side	=	1.25	m
Width of Pier Cap of Span 2 Side	=	1.25	m
Total Width of Pier Cap	=	2.5	m
Length of Pier Cap	=	10	
No. of Bearings (Span 1)	=	2	Nos
Long. Distance of Span 1 Bearings from c/l of Pier	=	0.75	m
Width of Pier Cap Wall of Span 1 Side	=	0.00	m
Height of Pier Cap Wall of Span 1 Side	=	0.00	m
Length of Pier Cap Wall of Span 1 Side	=	0.00	m
Ht of Brg + Pedestal (Span 1)	=	0.30	m
Dynamic Factor (Span 1)	=	1.33	
Multiple Presence Factor(Span 1)	=	1.20	
	=		
No. of Bearings (Span 2)	=	2	Nos
Long. Distance of Span 2 Bearings from c/l of Pier	=	0.75	m
Width of Pier Cap Wall of Span 2 Side	=	0.70	m
Height of Pier Cap Wall of Span 2 Side	=	0.50	m
Length of Pier Cap Wall of Span 2 Side	=	10.00	m
Ht of Brg + Pedestal (Span 2)	=	0.30	m
Dynamic Factor (Span 2)	=	1.33	
Multiple Presence Factor(Span 2)	=	1.00	
Dynamic Factor For Fatigue Vehicle	=	1.15	
Pedestal Size (tentative)	=	0.70	
Distance of Pier Cap edge from Pedestal face	=	0.50	
Unit. Wt of Concrete	=	25	KN/m^3

Design Of Pier Cap In Transverse Direction



Distance of cg of brg. from face of equivalent square (a)	=	3.3	m
Effective Depth at Junction (de)	=	1.318	m
a/d_e	=	2.52	

Design as Cantilever

Dead load Calculation Of Pier Cap

Item	Sec 1-1	Sec 2-2	Sec 3-3	Unit
Depth of Pier Cap at Edge	0.70	0.70	0.70	m
Depth of Pier Cap at Junction	1.40	1.18	0.88	m
Span1				
Length of Cantilever (Span 1 Side)	4.20	2.88	1.10	m
Self weight of Pier Cap (Span 1 Side)	**137.9**	**84.7**	**27.2**	KN
Long. Ecc.	0.63	0.6	0.6	m
Trans. Ecc.	1.868	1.319	0.529	m
Self weight of Pier Cap Wall (Span 1 Side)	**0.0**	**0.0**	**0.0**	KN
Long. Ecc.	0.75	0.75	0.75	m
Trans. Ecc.	2.1	1.4	0.6	m
Total (Span 1)				
Total Weight of Pier Cap	**137.9**	**84.7**	**27.2**	m
Moment	**257.5**	**111.8**	**14.4**	KN-m
Torsion	**86.2**	**53.0**	**17.0**	KN-m
Span2				

Length of Cantilever (Span 2 Side)	4.20	2.88	1.10	m
Self weight of Pier Cap (Span 2 Side)	**137.89**	**84.75**	**27.21**	KN
Long. Ecc.	-0.63	-0.63	-0.63	m
Trans. Ecc.	1.87	1.32	0.53	m
Self weight of Pier Cap Wall (Span 2 Side)	**36.77**	**25.24**	**9.63**	KN
Long. Ecc.	-0.75	-0.75	-0.75	m
Trans. Ecc.	2.10	1.44	0.55	m
Total (Span2)				
Total Weight of Pier Cap	**174.66**	**109.99**	**36.84**	m
Moment	**334.81**	**148.22**	**19.68**	KNm
Torsion	**-113.76**	**-71.90**	**-24.23**	KNm

Load Factor	***Strength-I***	***Strength-III***	***Strength-V***	***Service-I***	***Fatigue***
DC(Max)	1.25	1.25	1.25	1.0	0.0
DW (Max)	1.5	1.5	1.5	1.0	0.0
DC(Min)	0.9	0.9	0.9	1.0	0.0
DW (Min)	0.65	0.65	0.65	1.0	0.0
LL/CF/BR/PL	1.75	0	1.35	1.0	1.50
WS	0	1.4	0.4	0.3	0.0
FR	1	1	1	1.0	0.0
TU	0.5	0.5	0.5	1.0	0.0

Summary of Unfactored Load on Bearings Section-1

Load Items	**SPAN 1**				**SPAN 2**			
	Bearing 1	**Bearing 2**	**Bearing 3**	**Bearing 4**	**Bearing 1**	**Bearing 2**	**Bearing 3**	**Bearing 4**
Trans. Distance from face	3.33	0.58	0.00	0.00	3.33	0.58	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	534	0	0	496	534	0	0
SIDL CB	135	61	0	0	135	61	0	0
SIDL WC	64	79	0	0	64	79	0	0
FPLL	0	0	0	0	0	0	0	0
CWLL (With Out DA)	127	43	0.00	0.00	94	34	0.00	0.00
CW Lane Load	81	45	0.00	0.00	81.00	45.00	0.00	0.00
CF	0	0	0.00	0.00	0	0	0.00	0.00
Verticle Wind	36	36	0.00	0.00	28	28	0.00	0.00
Trans. Wind Laod	0	0	0.00	0.00	0	0	0.00	0.00
Friction Force (Horz. Forces)	9.43	9.43	0.00	0.00	9.43	9.43	0.00	0.00
Long. Wind (Horz. Forces)	5.23	5.23	0.00	0.00	5.23	5.23	0.00	0.00
Braking (Horz. Forces)	0	0	0.00	0.00	0	0	0.00	0.00
Fatigue LL	122	42	0.00	0.00	93	34	0.00	0.00

Summary of Unfactored Moment, Shear & Torsion-Section -1

Load Items	**SPAN 1**			**SPAN 2**		
	Moment	**Shear**	**Torsion**	**Moment**	**Shear**	**Torsion**
Dead Load	1959	1030	773	1959	1030	-773
SIDL CB	484	196	147	484	196	-147
SIDL WC	259	143	107	259	143	-107
FPLL	0	0	0	0	0	0
CWLL (With DA)	1069	423	317	885	355	-267
CF	0	0	0	0	0	0
Verticle Wind	141	72	54	109	56	-42
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			19			28
Long. Wind (Horz. Forces)			10			16
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	258	138	86	335	175	-114
Fatigue load	495	189	141	378	146	-110

Summary of factored Moment, Shear & Torsion-Section -1

Combination	***Details***	**Moment KN-m**	**Shear KN**	**Torsion KN-m**
1	Strength I Maximum load effect	11043	5246	77
2	Strength I Minimum load effect	9968	4696	490
3	Strength III Maximum load effect	7974	4064	43
4	Strength III Minimum load effect	6899	3513	456
5	Strength V Maximum load effect	10362	4986	72
6	Strength V Minimum load effect	9287	4436	485
7	Service I	8025	3867	81

8	Fatigue	1310	502	48

Summary of Unfactored Load on Bearings Section-2

Load Items	SPAN 1				SPAN 2			
	Bearing 1	Bearing 2	Bearing 3	Bearing 4	Bearing 1	Bearing 2	Bearing 3	Bearing 4
Trans. Distance from face	2.01	0.00	0.00	0.00	2.01	0.00	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	0	0	0	496	0	0	0
SIDL CB	135	0	0	0	135	0	0	0
SIDL WC	64	0	0	0	64	0	0	0
FPLL	0	0	0	0	0	0	0	0
CWLL (With Out DA)	127	0	0	0	94	0	0	0
CW Lane Load	81	0	0	0	81	0	0	0
CF	0	0	0	0	0	0	0	0
Verticle Wind	36	0	0	0	28	0	0	0
Trans. Wind Laod	0	0	0	0	0	0	0	0
Friction Force (Horz. Forces)	9	0	0	0	9	0	0	0
Long. Wind (Horz. Forces)	5	0	0	0	5	0	0	0
Braking (Horz. Forces)	0	0	0	0	0	0	0	0
Fatigue LL	122	0	0	0	93	0	0	0

Summary of Unfactored Moment, Shear & Torsion-Section -2

Load Items	SPAN 1			SPAN 2		
	Moment	Shear	Torsion	Moment	Shear	Torsion
Dead Load	997	496	372	997	496	-372
SIDL CB	271	135	101	271	135	-101
SIDL WC	129	64	48	129	64	-48
FPLL	0	0	0	0	0	0
CWLL (With DA)	603	300	225	497	247	-185
CF	0	0	0	0	0	0
Verticle Wind	72	36	27	56	28	-21
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			8			13
Long. Wind (Horz. Forces)			5			7
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	112	85	53	148	110	-72
Fatigue load	282	140	105	215	107	-80

Summary of factored Moment, Shear & Torsion-Section -2

Combination	*Details*	Moment KN-m	Shear KN	Torsion KN-m
1	Strength I Maximum load effect	5805	2970	56
2	Strength I Minimum load effect	5251	2695	263
3	Strength III Maximum load effect	4061	2103	12
4	Strength III Minimum load effect	3508	1827	219
5	Strength V Maximum load effect	5416	2777	48
6	Strength V Minimum load effect	4863	2502	254
7	Service I	4191	2151	47
8	Fatigue	745	371	38

Summary of Unfactored Load on Bearings Section-3

Load Items	SPAN 1				SPAN 2			
	Bearing 1	Bearing 2	Bearing 3	Bearing 4	Bearing 1	Bearing 2	Bearing 3	Bearing 4
Trans. Distance from face	0.35	0.00	0.00	0.00	0.35	0.00	0.00	0.00
Long. Distance from C/L Brg	0.75	0.75	0.75	0.75	-0.75	-0.75	-0.75	-0.75
Dead Load	496	0	0	0	496	0	0	0
SIDL CB	135	0	0	0	135	0	0	0
SIDL WC	64	0	0	0	64	0	0	0
FPLL	0	0	0	0	0	0	0	0
CWLL (With Out DA)	127	0	0	0	94	0	0	0
CW Lane Load	81	0	0	0	81	0	0	0
CF	0	0	0	0	0	0	0	0
Verticle Wind	36	0	0	0	28	0	0	0
Trans. Wind Laod	0	0	0	0	0	0	0	0
Friction Force (Horz. Forces)	9	0	0	0	9	0	0	0
Long. Wind (Horz. Forces)	5	0	0	0	5	0	0	0
Braking (Horz. Forces)	0	0	0	0	0	0	0	0
Fatigue LL	122	0	0	0	93	0	0	0

Summary of Unfactored Moment, Shear & Torsion-Section -3

Load Items	SPAN 1			SPAN 2		
	Moment	Shear	Torsion	Moment	Shear	Torsion
Dead Load	174	496	372	174	496	-372
SIDL CB	47	135	101	47	135	-101
SIDL WC	22	64	48	22	64	-48
FPLL	0	0	0	0	0	0
CWLL (With Out DA)	105	300	225	87	247	-185
CF	0	0	0	0	0	0
Verticle Wind	13	36	27	10	28	-21
Trans. Wind Laod	0	0	0	0	0	0
Friction Force (Horz. Forces)			7			12
Long. Wind (Horz. Forces)			4			6
Braking (Horz. Forces)			0			0
Dead Load of Pier Cap	14	27	17	20	37	-24
Fatigue load	49	140	105	37	107	-80

Summary of factored Moment, Shear & Torsion-Section -3

Combination	*Details*	Moment KN-m	Shear KN	Torsion KN-m
1	Strength I Maximum load effect	997	2807	69
2	Strength I Minimum load effect	901	2532	276
3	Strength III Maximum load effect	693	1939	23
4	Strength III Minimum load effect	597	1664	230
5	Strength V Maximum load effect	929	2614	60
6	Strength V Minimum load effect	833	2339	267
7	Service I	719	2020	56
8	Fatigue	130	371	38

Design of Pier Cap for Bending & Torsion in Section 1

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	C420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.0875	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	1318	mm
Width of Section (b)		=	2500	mm
Main Bar Dia. Provided		=	32	mm
Shear R/F. Provided		=	16	mm
Clear Cover Provided		=	50	mm
Maximum moment in section		=	**11043**	KN-m

Area of steel is obtained by solving the quadratic equation:- $\mathbf{As = [\ B - \sqrt{(B^2 - 4*M_n * A)}\ ]/(\ 2 * A\ )}$

Where, B = $d*f_y$ 553560

A = $(f_y^2)/(1.7 * f_c' * b)$ 1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **235.52** cm^2

Provide **23** Nos. **32** Dia. (at tension face) = **257.23** cm^2

23 Nos. **20** Dia. **OK**

Spacing of Reinforcement = 106 mm

Neutral axis a = $A_s f_y/0.85 f_c b$ = 169.5 mm

c = a/β1 = 199.4 mm

ε_c = 0.003

ε_t = $(d-c)*\varepsilon_c/c$ = 0.017

Tension Cont.

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.31290881	Mpa
Gross Moment of Inertia	I_g		=	5.7167E+11	mm^4
Distance from tension face to Cg	y_t		=	700	mm
Cracking Moment	Mcr		=	4338.88	KN-m
1.2 times M_{cr}			=	5206.65063	KN-m
Reinforcement required for 1.2 Cracking moment			=	**107.39**	cm^2
1.33 times M_u			=	14687.3	KN-m
Reinforcement required for 1.33 M_u			=	**320.47**	cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**20.19**	cm^2
So Reqiured area of Reinforcement			=	**235.52**	cm^2
				OK	
As required from Cpmpression side			=	**20.19**	cm^2

Provide **23** Nos. **16** Dia. (at Comp. face) = **46.24** cm^2

0 Nos. **0** Dia. **OK**

Moment In service stage

Net Bending moment in Service I = **8025** KN-m/m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c)))$ = 1.069

γe 0.75

Modular ratio (n)	=	6.00	
Position of nutural axis from comp face (x)	=	346.366	mm
MOA of compression area	=	149962091	mm^2
MOA of Tension area	=	149962091	mm^2
Diff	=	0.000	
MOI of cracked section	=	1.8034E+11	mm^4
Stress in tension reinf (f_{ss})	=	**259.4**	Mpa
Stress in Concrete (f_{cs})	=	**15.4**	Mpa
		OK	
Required Spacing of reinforcement s	=	**333**	mm
		OK	

Side face Reinforcement

Area of skin reinforcement A_{sk} = **5.58** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	5996 +1,309.743	=	7306 KN-m
Minimum Moment	5996 +0.003	=	5996 KN-m

Modular ratio (n)				=	6.00	
Position of nutural axis from comp face (x)				=	346.366	mm
MOA of compression area				=	149962091	mm^2
MOA of Tension area				=	149962091	mm^2
Diff				=	0.000	
MOI of cracked section				=	1.8034E+11	mm^4
Stress in tension reinf(For Max. Moment)	f_{ssMax}			=	**236.2**	Mpa
Stress in tension reinf(For Min. Moment)	f_{ssMin}			=	**193.8**	Mpa
Stress range in the R/F				=	**42.34**	Mpa
	$f_f=166-0.33f_{min}$			=	**102**	Mpa
					OK	

Check For Shear & Torsion for Section -1

A_{cp}				=	3500000	mm^2
P_c				=	7800	mm
A_0				=	3061056	mm^2
P_h				=	7336	mm
Factored Torsional Moment			T_u	=	**490**	KN-m
Torsional Cracking Moment	$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$		T_{cr}	=	2821	KN-m
			$0.25\phi T_{cr}$	=	635	KN-m
					Torsion Check Not Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : 16 dia. 2 L Stirrups 100 Spacing

A_t				=	201.1	mm^2
Nominal torsional resistance	$2A_0A_tf_y\cot\theta/s$		T_n	=	**5170**	KN-m
Factored torsional resistance			$T_r=\phi T_n$	=	**4653**	KN-m
					OK	

Longitudianl Reinforcement

A_sf_y				=	14097	KN
$M_u/\phi d_v$				=	9949	KN
$\cot\theta\,sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$				=	3031	KN
					OK	

Maximum shear force on the section				=	**5246**	KN
Corresponding Torsion			T_u	=	**77**	KN-m
Equivalent shear force	$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$			=	5247	KN
Total facrored shear force			V_u	=	**5247**	KN
Effective Shear depth			d_v	=	1233	mm
Width of critical section			b_v	=	2500	mm
β				=	2	
θ				=	45	degree
Strength reduction factor ϕ for shear & torsion				=	0.9	
Shear stress on concrete			υ_u	=	1.89	Mpa
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$		V_c	=	2803	KN
					SF.REQD	
Capacity of shear steel	Vs =	$A_v.\ f_y.d_v.\cot\theta/s$			5598	KN
Spacing of Stirrups			s	=	100	mm

Provide	**16**	dia.	**2 L**	Stirrups	**100**	Spacing	A_{sw} provided	=	**1081** mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing			

Max. spacing of transverse Reinforcement

If $\upsilon_u<0.125f_c'$ then $S_{max}=0.8d_v<=600$	=	600	mm
If $\upsilon_u>=0.125f_c'$ then $S_{max}=0.4d_v<=300$		**OK**	

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	8401	KN
	$V_n = 0.25fc'b_vd_v$	=	23124	KN
Factored Shear Resistance	$V_r = \phi Vn$	=	**7561**	KN
			OK	

Check for minimum Shear Reinforcement

$A_v>0.083\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2
		OK	

Design of Pier Cap for Bending & Torsion Section 2

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa	
Grade of Reinforcement	f_y	=	C420	Mpa	
Modulus of Elasticity of Concrete	E_{con}	=	29440.0875	Mpa	
Modulus of elasticity of steel	E_s	=	200000	Mpa	
deff provd.	d	=	1098	mm	
Width of Section (b)		=	2500	mm	
Main Bar Dia. Provided		=	32	mm	
Shear R/F. Provided		=	16	mm	
Clear Cover Provided		=	50	mm	
Maximum moment in section		=	**5805**	KN-m	

Area of steel is obtained by solving the quadratic equation:- **$As = [B - \sqrt{(B^2 - 4*M_n * A)}]/(2 * A)$**

Where, B = $d*f_y$ = 461353

A = $(f_y^2)/(1.7 * f_c' * b)$ = 1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration					=	**146.21**	cm^2
Provide	**23**	Nos.	**32**	Dia.	(at tension face) =	**257.23**	cm^2 **OK**
	23	Nos.	**20**	Dia.			
Spacing of Reinforcement					=	106	mm
Neutral axis a =			$A_sf_y/0.85f_cb$		=	169.5	mm
c =	a/β1				=	199.4	mm
ε_c					=	0.003	
ε_t =	$(d-c)*\varepsilon_c/c$				=	0.014	**Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.31290881	Mpa
Gross Moment of Inertia	I_g		=	3.427E+11	mm^4
Distance from tension face to Cg	y_t		=	590	mm
Cracking Moment	Mcr		=	3085	KN-m
1.2 times M_{cr}			=	3702	KN-m
Reinforcement required for 1.2 Cracking moment			=	**91.67**	cm^2
1.33 times M_u			=	7720.1	KN-m
Reinforcement required for 1.33 M_u			=	**197.64**	cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	=	**18.04**	cm^2
So Reqiured area of Reinforcement			=	**146.21**	cm^2 **OK**

Moment In service stage

Net Bending moment in Service I = **4191** KN-m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000*\gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c))$ = 1.082

γe 0.75

Modular ratio (n)		=	6.00	
Position of nutural axis from comp face (x)		=	311.681	mm
MOA of compression area		=	121431366	mm^2
MOA of Tension area		=	121431366	mm^2
Diff		=	0.000	
MOI of cracked section		=	1.2077E+11	mm^4
Stress in tension reinf	f_{ss}	=	**163.8**	Mpa
Stress in Concrete	f_{cs}	=	**10.8**	Mpa
			OK	
Required Spacing of reinforcement s		=	**520**	mm
			OK	

Side face Reinforcement

Area of skin reinforcement A_{sk} = **3.38** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	3053 +745.233	=	3798	KN-m
Minimum Moment	3053 +0.003	=	3053	KN-m

Modular ratio (n)		=	6.00	
Position of nutural axis from comp face (x)		=	311.681	mm
MOA of compression area		=	121431366	mm^2
MOA of Tension area		=	121431366	mm^2
Diff		=	0.000	
MOI of cracked section		=	1.2077E+11	mm^4
Stress in tension reinf(For Max. Moment)	f_{ssMax}	=	**148.5**	Mpa
Stress in tension reinf(For Min. Moment)	f_{ssMin}	=	**119.3**	Mpa
Stress range in the R/F		=	**29.13**	Mpa
	$f_f=166-0.33f_{min}$	=	**127**	Mpa
			OK	

Check For Shear & Torsion for Section -2

A_{cp}			=	2951146	mm^2
P_c			=	7361	mm
A_0			=	2537669	mm^2
P_h			=	6897	mm
Factored Torsional Moment		T_u	=	**263**	KN-m
Torsional Cracking Moment	$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$	T_{cr}	=	2126	KN-m
		$0.25\phi T_{cr}$	=	478	KN-m
				Torsion Check Not Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : 16 dia. 2 L Stirrups 100 Spacing

A_t			=	201.1	mm^2	
Nominal torsional resistance	$2A_0A_tf_y\cot\theta/s$	T_n	=	**4286**	KN-m	
Factored torsional resistance		$T_r=\phi T_n$	=	**3857**	KN-m	
				OK		
Longitudianl Reinforcement						
A_sf_y			=	10804	KN	
$M_u/\phi d_v$			=	6362	KN	
$\cot\theta\,sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$			=	1001	KN	
				OK		
Maximum shear force on the section			=	**2970**	KN	
Corresponding Torsion		T_u	=	**56**	KN-m	
Equivalent shear force	$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$		=	2971	KN	
Total facrored shear force		V_u	=	**2971**	KN	
Effective Shear depth		d_v	=	1014	mm	
Width of critical section		b_v	=	2500	mm	
β			=	2		
θ			=	45	degree	
Strength reduction factor ϕ for shear & torsion			=	0.9		
Shear stress on concrete		υ_u	=	1.30	Mpa	
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$	V_c	=	2304	KN	
				SF.REQD		
Capacity of shear steel	Vs = $A_v.\,f_y.d_v.\cot\theta/s$			4601	KN	
Spacing of Stirrups		s	=	100	mm	

Provide							A_{sw} provided	=		
	16	dia.	**2 L**	Stirrups	**100**	Spacing			**1081**	mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing				

Max. spacing of transverse Reinforcement

If υ_u <0.125f_c' then $S_{max}=0.8d_v$<=600	=	600	mm	
If υ_u >=0.125f_c' then $S_{max}=0.4d_v$<=300		**OK**		

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	6906	KN
	$V_n = 0.25fc'b_vd_v$	=	19007	KN
Factored Shear Resistance	$V_r = \phi Vn$	=	**6215**	KN
			OK	

Check for minimum Shear Reinforcement

$A_v>0.083\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2
		OK	

Design of Pier Cap for Bending & Torsion Section 3

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.0875	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	801	mm
Width of Section (b)		=	2500	mm
Main Bar Dia. Provided		=	32	mm
Shear R/F. Provided		=	16	mm
Clear Cover Provided		=	50	mm
Maximum moment in section		=	**997**	KN-m

Area of steel is obtained by solving the quadratic equation:- **As =[B -$\sqrt{(B^2 - 4*M_n * A)}$]/(2 * A)**

Where,	B	=	$d*f_y$	336516
	A	=	$(f_y^2)/(1.7 * f_c' * b)$	1.38

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **33.38** cm^2

Provide **23** Nos. **32** Dia. (at tension face) = **257.23** cm^2

23 Nos. **20** Dia. **OK**

Spacing of Reinforcement = 106 mm

Neutral axis a = $A_s f_y / 0.85 f_c b$ = 169.5 mm

c = a/β1 = 199.4 mm

ε_c = 0.003

ε_t = (d-c)*ε_c/c = 0.009 **Tension Cont.**

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	= 5.31290881	Mpa
Gross Moment of Inertia	I_g		= 1.4354E+11	mm^4
Distance from tension face to Cg	y_t		= 442	mm
Cracking Moment	Mcr		= 1727	KN-m
1.2 times M_{cr}			= 2072	KN-m
Reinforcement required for 1.2 Cracking moment			= **70.46**	cm^2
1.33 times M_u			= 1326.0	KN-m
Reinforcement required for 1.33 M_u			= **44.60**	cm^2
Mininum Reinforcement from Shrinkage & Temperature Criteria		(CL-5.10.8)	= **14.68**	cm^2
So Reqiured area of Reinforcement			= **44.60**	cm^2

OK

Moment In service stage

Net Bending moment in Service I = **719** KN-m

Control of Cracking by distribution of Reinforcement:-

(as per Clause 5.7.3.4 of AASHTO LRFD)

The spacing s of reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{123000 * \gamma_e}{\beta s * f_{ss}}$$

Where βs $1+(d_c/(0.7*(h-d_c))$ = 1.111

γe 0.75

Modular ratio (n)		= 6.00	
Position of nutural axis from comp face (x)		= 258.796	mm
MOA of compression area		= 83719177.8	mm^2
MOA of Tension area		= 83719178	mm^2
Diff		= 0.000	
MOI of cracked section		= 5.9856E+10	mm^4
Stress in tension reinf	f_{ss}	= **39.1**	Mpa
Stress in Concrete	f_{cs}	= **3.1**	Mpa
		OK	
Required Spacing of reinforcement s		= **2124**	mm
		OK	

Side face Reinforcement

Area of skin reinforcement A_{sk} = **0.41** cm^2/m

*Total area of longitudianl skin Reinforcement (per face) need not exceed one fourth of required flexural R/F

*Spacing shall not excedd either $d_e/6$ or 300

Design for Fatigue

Maximum Moment	521 +129.813	= 650	KN-m
Minimum Moment	521 +0.003	= 521	KN-m
Modular ratio (n)		= 6.00	
Position of nutural axis from comp face (x)		= 258.796	mm
MOA of compression area		= 83719177.8	mm^2
MOA of Tension area		= 83719178	mm^2
Diff		= 0.000	
MOI of cracked section		= 5.9856E+10	mm^4
Stress in tension reinf(For Max. Moment)	f_{ssMax}	= **35.4**	Mpa
Stress in tension reinf(For Min. Moment)	f_{ssMin}	= **28.3**	Mpa
Stress range in the R/F		= **7.06**	Mpa

	$f_f=166-0.33f_{min}$		=	**157**	Mpa
				OK	
Check For Shear & Torsion for Section -3					
A_{cp}			=	2208072	mm^2
P_c			=	6766	mm
A_0			=	1829073	mm^2
P_h			=	6302	mm
Factored Torsional Moment		T_u	=	**276**	KN-m
Torsional Cracking Moment	$0.328*\sqrt{f_c'}\ A_{cp}^2/p_c$	T_{cr}	=	1294	KN-m
		$0.25\phi T_{cr}$	=	291	KN-m
				Torsion Check Not Reqd.	

Calculation for Torsion Ring :

Provide torsion ring : 16 dia. 2 L Stirrups 100 Spacing

A_t			=	201.1	mm^2
Nominal torsional resistance	$2A_0A_tf_y\cot\theta/s$	T_n	=	**3089**	KN-m
Factored torsional resistance		$T_r=\phi T_n$	=	**2780**	KN-m
				OK	

Longitudianl Reinforcement

A_sf_y		=	10804	KN
$M_u/\phi d_v$		=	1536	KN
$\cot\theta\,sqrt(V_u/\phi-0.5V_s)^2+(0.45P_hT_u/2A_0\phi)^2$		=	1484	KN
			OK	

Maximum shear force on the section			=	**2807**	KN
Corresponding Torsion		T_u	=	**69**	KN-m
Equivalent shear force	$\sqrt{(V_u^2+(0.9P_hT_u/2A_0)^2)}$		=	2809	KN
Total facrored shear force		V_u	=	**2809**	KN
Effective Shear depth		d_v	=	721	mm
Width of critical section		b_v	=	2500	mm
β			=	2	
θ			=	45	degree
Strength reduction factor ϕ for shear & torsion			=	0.9	
Shear stress on concrete		υ_u	=	1.73	Mpa
Shear strength provided by concrete	$0.083*\beta*\sqrt{f_c'}*b_v*d_v$	V_c	=	1639	KN
				SF.REQD	
Capacity of shear steel	Vs = $A_v.\ f_y.d_v.\cot\theta/s$			3273	KN
Spacing of Stirrups		s	=	100	mm

Provide	**16**	dia.	**2 L**	Stirrups	**100**	Spacing	A_{sw} provided	=	**1081**	mm^2
	12	dia.	**6 L**	Stirrups	**100**	Spacing				

Max. spacing of transverse Reinforcement

If $\upsilon_u<0.125f_c'$ then $S_{max}=0.8d_v<=600$		=	576.884747	mm
If $\upsilon_u>=0.125f_c'$ then $S_{max}=0.4d_v<=300$			**OK**	

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	4912	KN
	$V_n = 0.25fc'b_vd_v$	=	13521	KN
Factored Shear Resistance	$V_r = \phi Vn$	=	**4421**	KN
			OK	

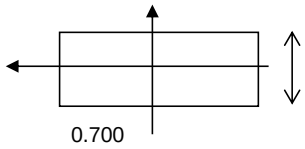
Check for minimum Shear Reinforcement

$A_v>0.083\sqrt{f_c'}b_vs/f_y$	=	270.6	mm^2
		OK	

DESIGN OF SEISMIC ARRESTER

Dimensional Details

Thickness of Stopper	=	0.400	m
Width of Stoper	=	0.700	m
Height of Stopper	=	0.600	m
Length of Stopper	=	0.450	m



Load Calculation

Dead load Reaction	=	2007	KN
SIDL Reaction	=	597	KN
LL Reaction	=	650.72	KN
Total Load	=	3254	KN
Seismic coefficient	=	0.15	
Forcre per Stopper		488.135	KN
Lever arm	=	0.450	m
Net bending Moment	=	219.661	KN-m
a	=	0.450	
a/d	=	1.42857	
		Design as a cantilever	

CALCULATION OF DESIGN PARAMETERS

Grade of Concrete	$f_{c'}$	=	C30	Mpa
Grade of Reinforcement	f_y	=	420	Mpa
Modulus of Elasticity of Concrete	E_{con}	=	29440.1	Mpa
Modulus of elasticity of steel	E_s	=	200000	Mpa
deff provd.	d	=	315	mm
Width of Section (b)		=	700	mm
Main Bar Dia. Provided		=	20	mm
Distribution Bar Dia. Provided		=	0	mm
Clear Cover Provided		=	75	mm

Area of steel is obtained by solving the quadratic equation:- **As =[B -$\sqrt{}$(B^2 - 4*M_n * A)]/(2 * A)**

Where, B = $d*f_y$ 132300

A = $(f_y^2)/(1.7 * f_c' * b)$ 4.94

If M_u = Factored Moment, then $M_n = M_u / \phi$ where ϕ =strength reduction factor 0.9

As required from Bending Consideration = **19.93** cm^2

Provide	**20**	Dia	**7 Nos**	+	**0**	Dia	**2 Nos**	=	**21.99**	cm^2
(at tension face)										**OK**

Neutral axis a =	$A_sf_y/0.85f_c'b$	=	51.7	mm
c =	a/β1	=	60.9	mm
ε_c		=	0.003	
ε_t =	$(d-c)*\varepsilon_c/c$	=	0.013	
				Tension Cont.

CHECK FOR MINIMUM REINFORCEMENT

Modulus of Rupture of concrete	f_r	(CL-5.4.2.6)	=	5.31291	Mpa
Gross Moment of Inertia	I_g		=	3.7E+09	mm^4
Distance from tension face to Cg	y_t		=	200	mm
Cracking Moment	Mcr		=	99.1743	KN-m
1.2 times M_{cr}			=	119.009	KN-m
Reinforcement required for 1.2 Cracking moment			=	**10.40**	cm^2/m
1.33 times M_u			=	292.1	KN-m

DESIGN OF SEISMIC ARRESTER

Reinforcement required for 1.33 M_u			=	**27.32**	cm^2/m	
Mininum Reinforcement from Shrinkage & Temperature Criteria	(CL-5.10.8)		=	**2.29**	cm^2/m	
So Reqiured area of Reinforcement			=	**19.93**	cm^2/m	
						OK
	≤					

Check For One way Shear

Net facrored shear force	V_u	=	**488**	KN	
Effective Shear depth	d_v	=	289	mm	
Width of critical section	b_v	=	700	mm	
β		=	2		
θ		=	45	degree	
Strength reduction factor ϕ for shear		=	0.9		
Shear stress on concrete	υ_u	=	2.68	Mpa	
Shear strength provided by concrete $0.083*\beta*\sqrt{f_c'}*b_v*d_v$	V_c	=	184	KN	
			SF.REQD		
Capacity of shear steel Vs = $A_v.f_y.d_v.\cot\theta/s$			439	KN	
Spacing of Stirrups	s	=	125	mm	

Provide	**12**	dia.	**4 L**	Stirrups	**125**	Spacing	A_{sw} provided	=	**452**	mm^2

Max. spacing of transverse Reinforcement

If $\upsilon_u < 0.125f_c'$ then $S_{max}=0.8d_v<=600$	=	231.302	mm	
If $\upsilon_u >= 0.125f_c'$ then $S_{max}=0.4d_v<=300$		**OK**		

Nomninal Shear Resistance	$V_n = V_c+V_s$	=	623	KN	
	$V_n = 0.25fc'b_vd_v$	I =	1518	KN	
Factored Shear Resistance	$Vr = \phi Vn$	=	**561**	KN	
			OK		

Check for minimum Shear Reinforcement

$A_v > 0.083\sqrt{f_c'}b_vs/f_y$	=	94.7	mm2	
		OK		