GOVERNMENT OF ORISSA

WORKS DEPARTMENT

ORISSA STATE ROAD PROJECT

FINAL DETAILED ENGINEERING REPORT
FOR PHASE-I ROADS
DESIGN REPORT OF BRIDGES
(BERHAMPUR TO BANGI JUNCTION)
(0.0 TO 41.0)

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INTRODUCTION

The work of detailed feasibility & preparation of Detailed Project Report has been awarded to M/S Consulting Engineers Group by client, The Chief Engineer (World Bank), Project Implementation Units & OWD, Bhubaneswar, vide agreement No.1 of 2005-06 dated 28/10/05

This report presents the detailed design report for bridges from Berhampur-Rayagada road on SH-17.from 0.0 km to 41.0 km. In this stretch, there are 10 nos of existing bridges. The final recommendation after analyzing the data based on detailed inventory & condition assessment, NDT testing, horizontal alignment, hydrological study, geo-technical investigations, vertical alignment & decision taking during site visit with PIU officials.

Geo-technical report, NDT reports & site visit report has already been submitted. The final recommendations after incorporating the above report are summarized in **Table** appended in next page.

The overall width of new construction has been kept as 12.0 m with or without footpath with clear carriageway width of 11.0 m in case of bridge without footpath and 7.5 m in case of bridges with footpath. Footpath has been provided for all major bridge and minor bridge lying in village vicinity.

The design of superstructure for RCC girder type bridge has been given as per STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C. T-Beam and Slab superstructure).

The design of superstructure for Solid slab type bridge has been given as per “STANDARD DRAWINGS FOR ROAD BRIDGES (R.C.C. Solid slab superstructure (15 and 30 skew).span 4.0 m to 10.0 m (With and without footpath)”

The substructure has been design using in-house software in Standard excel sheet.

The following code of practice has been referred in the designs.

1. IRC : 21-2000
2. IRC : 6-2004
3. IRC : 78-2000
4. SP : 13-2004
5. IRC : 89-1997
6. IRC : 83-1987
7. Specification for Road and Bridges works (MOST Book)

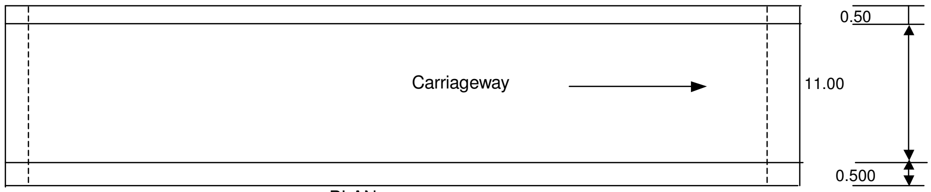
Sl No	Location/ Chainage	Existing Span Arrangement	Existing Carriage way width	Type of Bridge	Recommended for	Proposed Span Arrangement	Type of foundation	Type of substructure	Type of superstructure	Remarks
1	1/915	2 x 6.6	7.0	High Level	Rehabilitation	-	Open foundation	PCC wall type	RCC solid slab	Widening required due to change of alignment
2	4/400	3 x 6.75	7.5	High Level	Repair/Rehabilitation	-	Open foundation	PCC wall type	RCC solid slab	Good, Rehabilitation required
3	11/270	1 x 6.35	5.0	High Level	Reconstruction	1 x 10.8 Solid Slab	Open foundation	RCC wall type	RCC solid slab	Reconstruction due to poor condition
4	11/660	3 x 6.8	7.5	High Level	Repair/Rehabilitation	-	Open foundation	PCC wall type	RCC solid slab	Rehabilitation required
5	15/185	2 x 6.8	7.5	High Level	Repair/Rehabilitation	-	Open foundation	RCC wall type	RCC solid slab	Good, Rehabilitation required
6	15/680	4 X 6.8	7.5	High Level	Repair/Rehabilitation	-	Open foundation	PCC wall type	RCC solid slab	Good, Rehabilitation required
7	17/900	4 x 6.8	7.5	High Level	Repair/Rehabilitation	-	Open foundation	RCC wall type	RCC solid slab	Good, Rehabilitation required
8	21/850	3 x 10.8	7.5	High Level	Repair/Rehabilitation	-	Open foundation	PCC wall type	RCC solid slab	Good, Rehabilitation required
9	29/230	3 x 42.2	7.5	High Level	Nothing to do	-	Well foundation	RCC wall type	PSC T-beam girder	Good, Nothing to do
10	29/500	2 x 7.0	5.5	High Level	Reconstruction	1 x 21.6 RCC T-beam Girder	Pile foundation	RCC wall type	RCC T-beam girder	Reconstruction due to narrow in width and realignment as per proposed alignment

MINOR BRIDGE AT CH:11/270

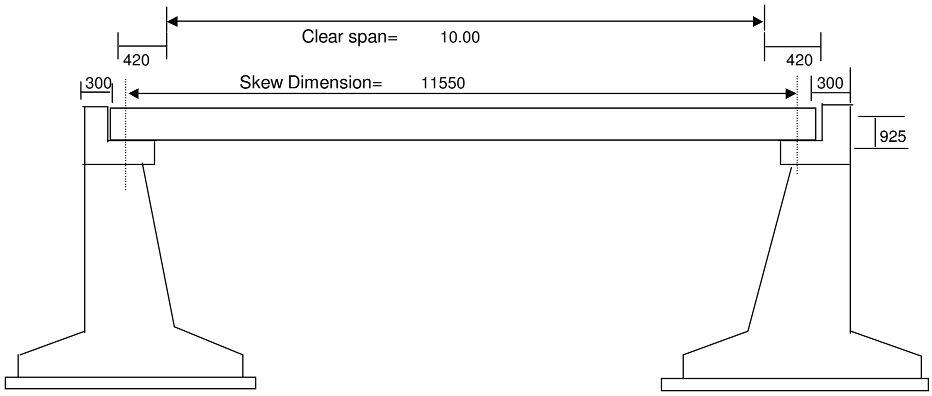
DESIGN OF SUBSTRUCTURE

Design of RCC abutment

a) General arrangement
The General arrangement is as follows



PLAN



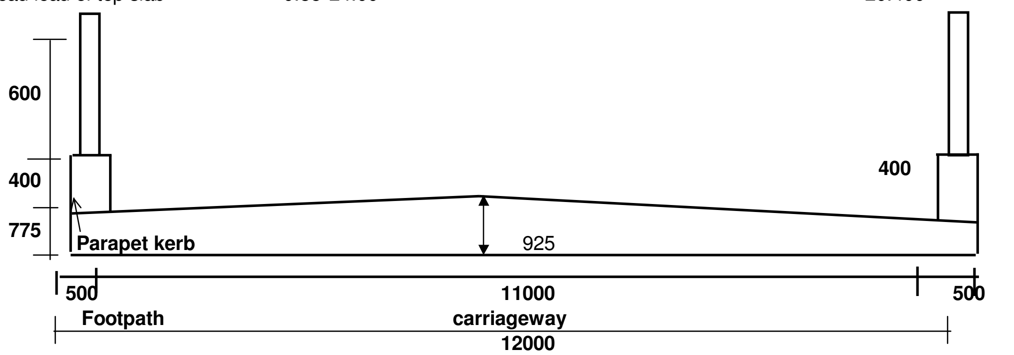
ELEVATION

Basic parameter

Unit weight of RCC		=	24.000	kN/m^3
Unit weight of wearing coat		=	22.000	kN/m^3
skew angle		=	30.000	degree
skew angle in radian	30/180*3.1416	=	0.524	radian
Thickness of wearing coat		=	0.056	M

Dead load calculation

Thickness of slab deck center		=	925.000	mm
Thickness of slab at edge		=	775.000	mm
Average Thickness of slab		=	850.000	mm
Dead load of top slab	0.85*24.00	=	20.400	kN/m^2



Total width of bridge		=	12.000	meter
Span c/c of bearing skew			11.550	meter
Width of crash barrier at top		=	0.225	meter
Width of crash barrier at bottom		=	0.500	meter
Height of crash barrier		=	1.000	meter
Crash barrier load intensity				
	(0.225+0.50)*0.5) x (1.000*24)	=	8.700	KN/m
Thickness of wearing coat		=	0.056	meter
Weight of wearing coat including provision for one additional coat				
	0.056*22*2	=	2.500	kN/m^2
Total pressure due to dead loads				
Crash barrier	8.70*11.55/2*2	=	100.485	kN
Top slab		=	20.400	kN/m^2
Total		=	22.900	kN/m^2

Dead load of slab at center of bearing of abutment (stadd out put)		**1530.000**	KN
Super Inposed Dead load at center of bearing of abutment (stadd out put)		**274.330**	KN
Total (Dead+SIDL) load at center of bearing of abutment (stadd out put)	=	**1804.330**	KN

ii) IRC 70R tracked and Class A loading

<u>Reaction due to Class 70R Tracked Vehicle</u>

Maximum Reaction on Abutment (STADD OUT PUT)

= 726.530 KN

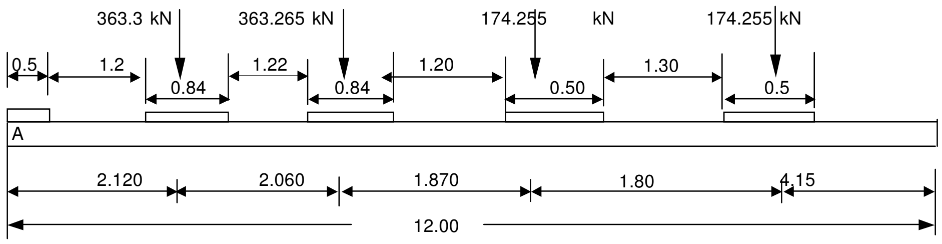
<u>Reaction due to Class A Vehicle</u>

Reaction due to Class A Loading

Maximum Reaction on Abutment (STADD OUTPUT)	=	348.510	KN
Total Reaction	=	1075.040	KN

<u>Transverse position</u>

Width of Track		=	0.840	meter
width of vehicle		=	2.900	meter
Clearance from kerb		=	1.200	meter
CG of loads from left end A		=	4.382	meter
Transverse eccentricity	12.00/2-4.38	=	1.618	meter
Transverse moment	1,075.04*1.62	=	**1739.526**	KN-m

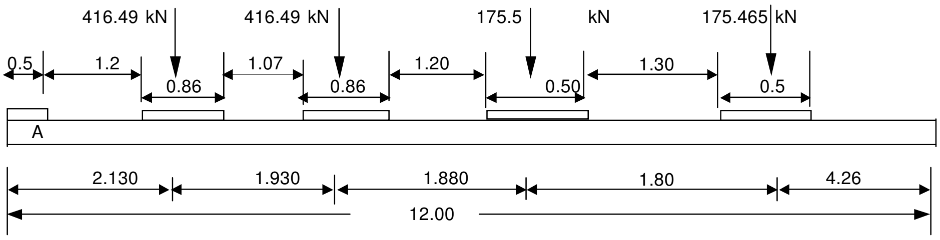


iii) IRC 70R Wheeled and Class A loading

Reaction due to Class A Loading (STADD OUT PUT)	=	350.930	KN
Reaction due to 70R wheel Loading (STADD OUT PUT)	=	832.980	KN
Total Reaction due to Class 70R wheel loading and class A loading	=	**1183.910**	KN

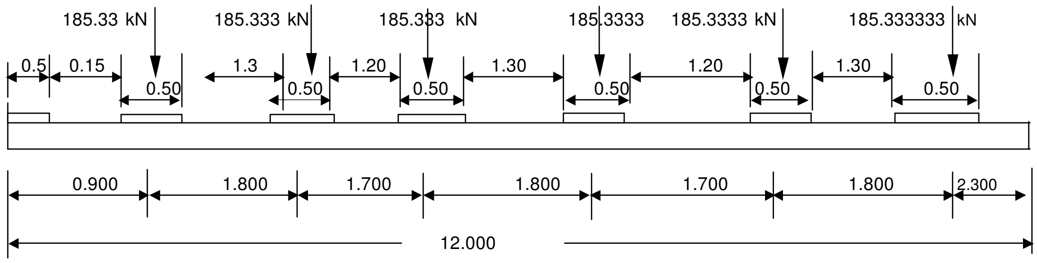
Transverse position

Width of Wheel		=	0.860	meter
width of vehicle		=	2.790	meter
Clearance in vehicles		=	1.200	
Clearance from kerb		=	1.200	meter
CG of loads from left end A		=	4.205	meter
Transverse eccentricity	12.00/2-4.21	=	1.795	meter
Transverse moment	1,183.91*1.79	=	2125.026	



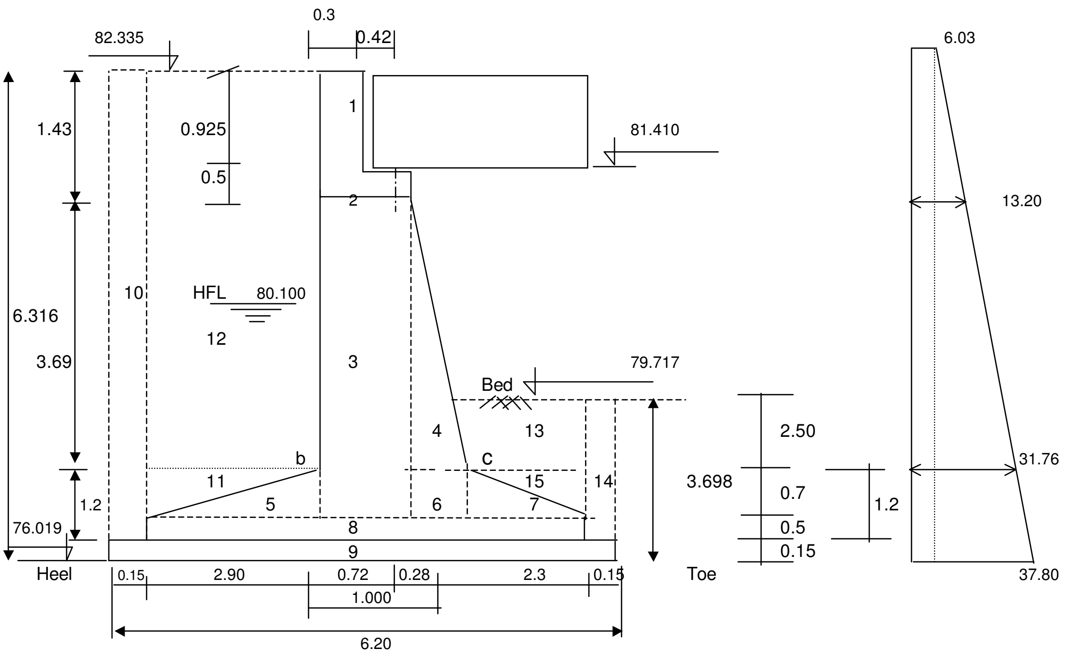
iv) Three lane of IRC Class A loading

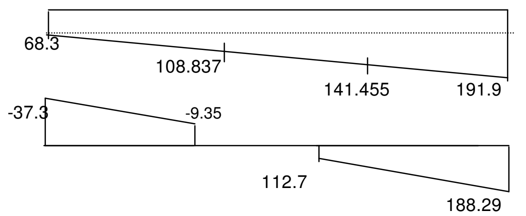
Total Reaction due to three lane of class A loading (STADD OUT PUT)		=	1112.000	KN
Transverse position				
Width of Wheel		=	0.500	meter
width of vehicle		=	1.800	meter
Clearance in vehicles		=	0.150	meter
Clearance from kerb		=	1.200	meter
CG of loads from left end A		=	5.300	meter
Transverse eccentricity	12.00/2-5.30	=	0.700	meter
Transverse moment	1,112.00*0.70	=	**778.400**	meter



Summary for different load conditions from superstructure

Load	P (KN)	Transvers M (KN-m)
Dead load	1804	0.000
IRC 70R tracked +class A	1075.04	1739.53
IRC 70R Wheeled +class A	1183.91	2125.03
Three lane of IRC A	1112.00	778.40





Grade of Concrete for sub Structure "M"	=	20.000	N/mm2
Grade of Reinforcement for sub Structure "Fe"	=	415.000	N/mm2
Unit weight of soil	=	18.000	kN/m^3
Unit weight of RCC	=	24.000	kN/m^3
Unit weight of PCC	=	22.000	kN/m^3
Density of back fill	=	18.000	kN/m^3
Submerge weight of soil	=	10.000	kN/m^3
Net Safe bearing capacity of soil		178.600	kN/m^2
Gross Safe bearing capacity of soil	=	215.580	kN/m^2
Depth of foundation	=	3.698	meter
Angle of internal friction of soil for active Earth pressure (ϕ)		30.000	Degree
(30/180*3.1416	=	0.524	Radians
Angle of internal friction of soil for passive Earth pressure (ϕ)		30.000	Degree
	=	0.524	Radians

Cross sectional detail

Deck level	=	82.335	m
Foundation level	=	76.019	m
HFL at site	=	80.100	m
Average bed level	=	79.717	m
Width of dirt wall	=	0.300	m
Thickness of slab	=	0.925	m
Total Height of dirt wall	=	1.425	m
Width of bearing including Exp joint	=	0.420	m
Thickness of vertical wall at top	=	0.720	m
Thickness of vertical wall at base	=	1.000	m
Projection of PCC	=	0.150	m
Thickness of PCC	=	0.150	m
Front projection in foundation slab	=	2.300	m
Back projection in foundation slab	=	2.900	m
Thickness of Foundation slab at Junction	=	1.200	m
Thickness of Abutment cap	=	0.500	m
Thickness of toe slab at edge	=	0.500	m
Thickness of heel slab at edge	=	0.500	m
Total base width of PCC	=	6.500	m

Earth Pressure

Active earth pressure co-efficient as per coulomb's theory

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta \cos(\delta+\theta)\left(1+\sqrt{\dfrac{\sin(\delta+\phi)\sin(\phi-\alpha)}{\cos(\delta+\theta)\cos(\theta-\alpha)}}\right)^2}$$

Where

ϕ = Angle of internal friction between of soil
θ = Angle of inclination of back of wall with vertical
δ = Angle of internal friction between retaining wall & earth
α = Angle of surcharge

ϕ =	30.000	deg.	=	0.524	Radian
θ =	0.000	deg.	=	0.000	Radian
δ =	20.000	deg.	=	0.349	Radian
α =	0.000	deg.	=	0.000	Radian

k_a = 0.2973

Therefore, Horizontal coefficient of Active earth pressure = $\mathbf{K_a Cos(\delta+\theta)}$

$\mathbf{K_{ha}}$ = **0.2794**

Therefore, Vertical coefficient of Active earth pressure = $\mathbf{K_a Sin(\delta+\theta)}$

$\mathbf{K_{va}}$ = **0.1017**

Horizontal coefficient of Active earth pressure =		0.279	m
vertical coefficient of Active earth pressure =		0.102	m
Coeff. of active earth pressure		0.279	
Coeff. of passive earth pressure Kp = (1+sinϕ)/(1-sinϕ)	=	3.000	
Coefficient of friction between base of wall & soil (μ)	=	0.500	
Additional surcharge Height due to load on embankment	=	1.200	m
Height of earth from heel of wall	=	6.316	m
Equivalent height of Active pressure	=	7.516	m
Equivalent height of Passive pressure	=	3.698	m
Passive earth pressure & wt. to be considered or not (y/n)		n	

Self wt of abutment

w_n	Particular	Description	Load/m	Length	Load kN
1	Weight of dirt wall	0.30*0.925*24	6.66	13.86	92.3
2	Weight of abutment cap	0.50*0.72*24	8.64	13.86	119.7
3	weight of rectangular	3.69*0.72*24	63.78	13.86	883.8
4	weight of Triangular	3.69*0.28*24*0.5	12.40	13.86	171.8
5	weight of Triangular	2.90*0.70*24*0.5	24.36	13.86	337.5
6	weight of rectangular	0.28*0.70*24	2.35	13.86	32.6
7	weight of Triangular	2.30*0.70*24*0.5	19.32	13.86	267.7
8	weight of rectangular	5.90*0.50*24	5.98	13.86	82.8
9	weight of rectangular pcc	6.20*0.15*22	20.46	13.86	283.5
10	Rectan of active earth	6.17*0.15*18.0	16.65	13.86	230.7
11	weight of Triangular	2.90*0.70*18.0*0.5	18.27	13.86	253.2
12	weight of rectangular	5.12*2.90*18.0	259	13.86	3591.9
13	rectan of passive earth	2.30*2.50*18.0	0.00	13.86	0
14	rectan of passive earth	3.55*0.15*18.0	0.00	13.86	0
15	rectan of passive earth	2.30*0.70*18.0*0.5	0.00	13.86	0

Dead load moment of the Forces about toe

S.no	Part description	vertical forces	Horizontal force (kN)	Lever arm (m)	Resisting Moment (kN.m)	Over turning Moment (kN.m)	Transverse moment (kN-m)
A	Active earth pressure (surcharge)		457.381	3.158		1444.409	
B	Active earth pressure		1203.674	2.653		3193.011	
1	Weight of dirt wall	92.3		3.300	304.536		
2	Weight of abutment cap	119.7		3.090	369.933		
3	weight of rectangular	883.8		3.090	2730.844		
4	weight of Triangular	171.8		2.637	453.095		
5	weight of Triangular	337.5		4.417	1490.811		
6	weight of rectangular	32.6		2.590	84.409		
7	weight of Triangular	267.7		1.683	450.638		
8	weight of rectangular	82.8		3.100	256.655		
9	weight of rectangular pcc	283.5		3.100	878.856		
10	Rectan of active earth	230.7		6.425	1482.146		
11	weight of Triangular	253.2		5.383	1362.826		
12	weight of rectangular	3591.9		4.900	17600.46		
13	rectan of passive earth	0.0		1.300	0.000		
14	rectangular of passive earth	0.0		0.075	0.000		
15	rectangular of passive earth	0.0		0.917	0.000		
16	Vertical load at bearing	1804.3		3.000	5412.990		0.0
	Total	8151.8	1661.055		32878.19	4637.41981	0.0

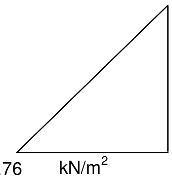
C.G of abutment structure from toe = 4.03 meter

i) Earth pressure on abutment

Angle of internal friction of soil for active Earth pressure (f) 30 Degree
(30/180*3.1416) = 0.52 Radians
Coeff. of active earth pressure Ka = = 0.2794

Max height of active pressure = 6.32 meter
Horizontal pressure at top 0.28*0*18.00 = 0.0 kN/m^2
Horizontal earth pressure at bottom 0.28*18.00*6.32 = 31.76 kN/m^2

0.0 kN/m^2

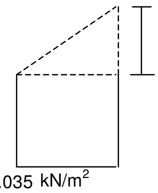


Lateral earth pressure

31.76 kN/m^2

Total horizontal pressure 0.28*18.00*6.32^2/2 = 100.31 kN/m
Height of action of horizontal pressure (As per IRC :6-2000 cl 217.1) = 0.42 Ht
Total horizontal pressure act at distance from base = 2.653 meter

Height of surcharge (As per IRC :78-1983, clu 714.4) = 1.2 meter
Surcharge horizontal pressure 0.28*18.00*1.20 = 6.035 kN/m^2



1.2 m

Surcharge pressure

6.035 kN/m^2

Total horizontal surcharge pressure = 38.12 kN/m
Surcharge pressure act at distance from base 3.16 meter

i) Check for the bottom of base slab

Total width of base slab = 6.20 meter
Total abutment base length = 13.86 meter
Components due to skew cos (angle) = 0.87
sin(angle) = 0.50

Live load from super structure

Total vertical load at bearing = 1183.91
Transverse moment due to live load = 2125.026

Live load:-braking force	70RT	70RW	A	2A	3A	70RT+A	70RW+A
Vertical force	700	920	384	768	1152	1084	1304
Braking 2 lane	20%	20%	20%	20%	20%	20%	20%
Braking 3rd-lane	5%	5%	5%	5%	5%	5%	5%
Braking force	140	184	76.8	76.8	96	159.2	203.2
Force on each Abutment	70	92	38.4	38.4	48	79.6	101.6
Force ht. from slab	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Height of slab	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Reaction on abutment	25.76	33.85	14.13	14.13	17.66	29.29	37.39

value of vertical braking force = 37.39 kN
value of horizontal braking force = 101.6 kN
longitunal component of braking force = 87.99 kN
Transverse component of braking force = 50.80 kN

Live load Moment about toe

S.no	Particular	Vertical force (KN)	Horizo.force kN	Lever arm (m)	overturning moment(kN)	Resisting moment (kN-m)	Transverse moment (kN-m)
1	Live load	1183.91		2.79	1062.515	3303.109	1840
2	Vertical Reaction due to Braking force	37.39		2.79		104.3049	
3	horizontal Braking force		87.99	5.39	474.3439		
4	Braking transverse		50.80	5.39			273.86
5	Footpath live load	0.00		2.79		0	0.00
6	Total	1221.3	138.788		1536.859	3407.414	2114.19

Load combination

Friction coefficient between concrete & soil (μ) = 0.5
Area of footing = 85.91 m^2
Section modulus of footing area about Transverse axis Zxx = 198.40 m^3
Section modulus of footing area about longitunal axis Zyy = 88.77 m^3

S.no	Loading			Vert. force (kN)	Horiz. force (kN)	Overturning Moment (kN.m)	Resisting Moment (kN.m)	Trans. moment (kN.m)	Long. moment (kN.m)
1	**(DL+LL) dry condition**								
	Dead load			9956.2	1661.06	4637.42	32878.2	0.00	
	Live load			1221.30	87.99	1536.86	3407.41	2114.19	
	Total			11177.5	1749.04	6174.28	36286	2114.19	4538.8
	Stress P/A or M/Zxx&M/Zyy (kN/m2)			130.107				10.656	51.13
				Calculated		Permissible			
	Max. base pressure (kN/m^2)			191.89	100%	216	(As per IRC:78-2000 clause 706.1.2)		
	Min. base pressure (kN/m^2)			68.32		0	(As per IRC:78-2000 clause 706.3.3)		
	FS against overturning			5.88		2	(As per IRC:78-2000 clause 706.3.4)		
	FS against sliding			3.20		1.5	(As per IRC:78-2000 clause 706.3.4)		

Pressure distribution at foundation

P/A = 130.107 kN/m^2
Mxx/Zxx = 51.13 kN/m^2
Myy/Zyy = 10.66 kN/m^2

Gross pressure intensity at foundation

Maximum pressure at toe = P/A+Myy/zyy+Mxx/Zxx 130.11+10.66+51.13 = 191.89 kN/m^2
Maximum pressure at heel = P/A+Myy/zyy-Mxx/Zxx 130.11+10.66-51.13 = 170.58 kN/m^2
Minimum pressure at toe = P/A-Myy/zyy+Mxx/Zxx 130.11-10.66+51.13 = 89.64 kN/m^2
Minimum pressure at heel = P/A-Myy/zyy-Mxx/Zxx 130.11-10.66-51.13 = 68.32 kN/m^2

Max pressure at the junction of toe & stem wall 191.89-(191.89-68.32)/2.45 = 141.46 kN/m^2
Min pressure at the junction of toe & stem wall 68.32+(191.89-68.32)/3.05 = 108.84 kN/m^2

Net pressure at toe slab

Downward wt of slab at toe 0.15*24 = 3.60 kN/m^2
at jn 1.20*24 = 28.8 kN/m^2
Downward wt of passive earth at toe 0.0 = 0.0 kN/m^2
at jn 0.0 = 0.0 kN/m^2
Net pressure at toe 191.89-3.60-0.00 = 188.29 kN/m^2
Net pressure at junction of toe & stem 141.46-28.80-0.00 = 112.7 kN/m^2

Net pressure at heel slab

Downward wt of slab at heel 0.15*24.00 = 3.60 kN/m^2
at jn 1.20*24.00 = 28.8 kN/m^2
Downward wt of backfill at heel 5.67*18.00 = 102.0 kN/m^2
at jn 4.97*18.00 = 89.4 kN/m^2
Net pressure at heel 68.32-3.60-101.99 = -37.27 kN/m^2
Net pressure at junction of heel & stem 108.84-89.39-28.80 = -9.4 kN/m^2

Design of toe slab

Design for bending moment

The forces acting acting on the toe slab are
i) downward force due to weight of slab
ii) upward soil pressure on cd

B.M at the junction, c	=				
		(188.29-112.66)*0.5*2.30^2*2/3+112.66*2.30^2*0.50		= 431.3	KN-M
Constants	c			= 6.67	
	t			= 200	
	m			= 10	
	k =1/(1+t/mc)	1/(1+200/(10*6.67))		= 0.25	
	j=1-k/3	1-0.250/3		= 0.92	
	Q=1/2ck j	1/2*6.67*0.250*0.917)		= 0.764	
Depth required		(431.35*10^6/(0.76*1000))^0.5		= 751.446	mm
Total depth provided				= 1200	mm
Effective Depth provided	cover 75mm	1,200.0-75.00-10.00		1115	mm
				> 751.4	mm
				Hence ok	
Area of steel required (M/tjd)		431.35*1,000,000/(200.0*0.92*1,115.0		= 2110.13	mm^2
Minimum area of steel as per IRC				= 0.15	%
Minimum area of steel required		0.15/100*1,115.00*1,000		= 1672.5	mm^2
Provide reinf for bending		Bar1	Bar2		
	dia	20	0		
	spacing required	149	0		
	spacing provided	140	0		
	total area provided	2244	0	= 2244	mm^2
	percentage of tensile reinf provided			= 0.20%	
				> 2110.1	mm^2
				Hence ok	
Distribution reinforcement				= 0.12	%
Average depth		(0.70+0.50)/2		= 0.60	m
Area of distribution reinforcement		0.12/100*600*1,000		= 720	mm^2
				Hence ok	
		Bar1	Bar2		
	dia	12	0		
	spacing required	157	0		
	spacing provided	150	190		
	total area provided	753.98	0	= 754	mm^2
	percentage of tensile reinf provided			= 0.13	%

There is no Tension below foundation , Hence fondation will not Have Nagative Moment at top ,
However in refrence to clause 707.2.8:of IRC 78-2000, The requrement of reinforcement at top is folows.

Minimum steel reinforcement as per above Clause		= 250	mm^2/m
Dia of reinforcement		= 12	mm
Spacing Required	(3.142/4*144)*1,000)\250.00	= 452.39	
Provide Spacing		= 150	mm
Area of reinforcement provided		= 754	mm2/m

Design for shear force

As per clause 304.7.1 of IRC: 21.2000

Angle between top & bottom face of base slab tan	β	0.30	
Base pressure intensity at distance d	112.66+(188.29-112.66)/2.30*1.20	= 140.06	kN/m^2
Shear force at distance d per m width	(188.29+140.06)\2*(2.30-1.1)	194.55	KN
Effective thickness of base slab at distance d	1.35-(1.35/2.30*1.1)*1000-0.075-0.010	1010.57	mm
Design shear stress	(194.55*1000-431.35*10^6*0.304/1,010.57)/(1,010.57*1000)	= 0.064	N/mm^2
Percentage of tensile reinf. Provided		= 0.20	%
Permissible Shear Stress		= 1.8	N/mm^2

Check for shear stress for which the Reinforcement is not Required

Stress = $K\tau_C$

K	= 1	
τ_C from table 12B of IRC 21-2000	= 0.215	N/mm2
	Hence ok	

Design of Heel slab

Design for bending moment

B.M at the junction	0.5*(-37.27--9.35)*2.9^2*2/3+-9.35*2.90^2*0.5		= 105.4	
Constants	c		= 6.7	
	t		= 200	
	m		= 10	
	k =1/(1+t/mc)	1/(1+200/(10*6.67))	= 0.250	
	j=1-k/3	1-0.250/3	= 0.917	
	Q=1/2ck j	1/2*6.67*0.250*0.917)	= 0.764	
Depth required		(105.37*10^6/(0.76*1000))^0.5	= 371.40	mm
Total depth provided			= 1200	mm
Effective Depth provided	cover 75mm	1,200.0-75.00-10.00	1115	mm
			> 371.40	mm
			Hence ok	
Area of steel required (M/tjd)	105.37*10^6/(200.0*1*1,115.0)		= 515.47	mm^2
Minimum area of steel as per IRC			= 0.15	
Minimum area of steel required		0.15/100*1,115.00*1,000	= 1672.5	mm^2

Provide reinf for bending		Bar1	Bar2		
	dia	20	0		
	spacing required	188	0		
	spacing provided	150	0		
	total area provided	2094.4	0	= 2094	mm^2
	percentage of tensile reinf provided			= 0.19%	
				> 515.47	mm^2
				Hence ok	

Provided distribution reinforcement same as in toe slab
The requirement of reinforcement in reference to clause 707.2.8 of IRC :78.2000
on opposite face (bottom) is same as for toe slab

Design for shear force

As per clause 304.7.1 of IRC: 21.2000

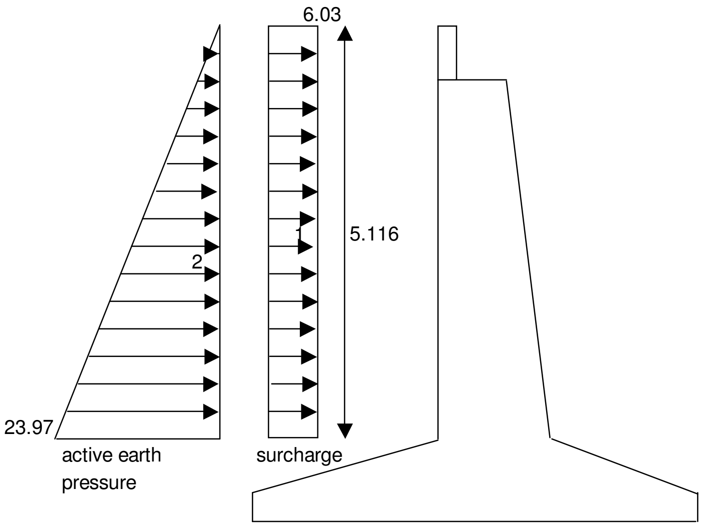
Angle between top & bottom face of base slab tan	β	0.241	
Base pressure intensity at distance d	-9.35+(-37.27--9.35)/2.90*1.12	= -17.372	kN/m^2
Shear force at distance d per m width	(-17.37+-37.27)\2*(2.90-1.1)	48.764	KN
Effective thickness of base slab at distance d	1.35-(1.35/2.90*1.1)*1000-0.075-0.010	888.545	mm
Design shear stress	(48.76*1000-1.00*10^6*0.241/888.55)/(888.55*1000)	= 0.055	N/mm^2
Percentage of tensile reinf. Provided		= 0.19	%
Permissible Shear Stress		= 1.8	N/mm^2

Check for shear stress for which the Reinforcement is not Required

Stress = $K\tau_C$

K	= 1	
τ_C from table 12B of IRC 21-2000	= 0.211	N/mm2

Design of Abutment wall



element	Ht	Earth pressure	Force	L.A	Moment
1	4.766	6.03	398.53	2.383	949.693
2	4.766	23.97	791.41	2.002	1584.184
Horizantal force			138.79	4.191	581.66
					3115.54

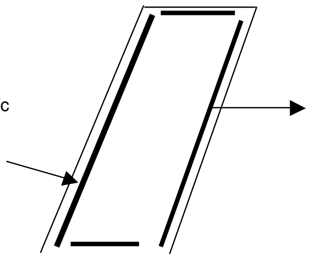
Total vertical load 1267.6 + 2988.24 = 4255.86 Kn
Longitudinal Moment = 3115.54 Kn-m
Transverse Moment = 2125.03 Kn-m

Span dislodged condition

Vertical load = 1267.62
Longitudinal moment = 2533.88
Transverse moment = 0.00

Reinforcement have been provided as follows and checked with separate programme.
The stresses in concrete and steel are well within permissible limits.

25 ϕ 200 c/c



16 ϕ 200 c/c

Horizontal reinforcement

Lateral reinforcement on each face	0.125	%
Thickness at base	1000	mm
Area of reinforcement per meter height	1250	mm2
Diameter of bars	16	mm
Spacing of bars required	161	mm
Spacing of bars provided	150	mm

Connecting links of 10mm dia tor bars @ 200 c/c spacing shall be provided.

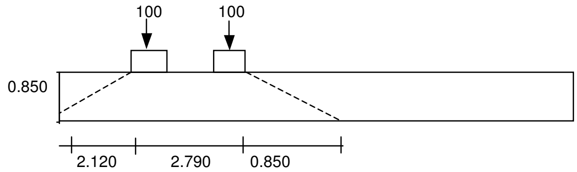
Dirt wall

Vertical reinforcement : inner base

B.M at base due to earth pressure

(a) Vertical reinforcement : inner face

BM at base due to earth pr.	6.03*0.93*0.93/2+0.5*(13.20-6.03)*0.93*0.93*0.42	3.87	KN-m
Max. live load that can come on dirt wall is due to bogie load		200.00	kN
Horizontal load @ 20%		40.00	kN



Total width of dispersion		5.760	m
Horizontal load per meter width		6.94	kN/m
Horizontal load per meter width perpendicular to dirt wall		6.94	kN/m
BM at base due to horizontal load	6.94*0.850	5.90	KN-m
Total bending moment at base	5.90+3.87	9.77	KN-m
Concrete grade for dirt wall "M"		25.00	
Constants :	c	8.33	N/mm^2
	t	200.00	N/mm^2
	m	10.00	
	k = 1/(1+t/mc)	0.294	
	j = 1-k/3	0.902	
	Q = (1/2) c k j	1.105	
Thickness required	(9.77*10^6/(1.11*1000))^0.5	94	mm
Total depth provided		300	mm
Clear cover		50	mm
Eff. cover	50.00+12.00/2	56	mm
Effective thickness provided	300.00-56.00	244	mm
Area of steel required (M/tjd)	9.77*10^6/(200.00*0.90*244.00)	222	mm^2
Min % of steel		0.15	%
Min area of steel required	0.15/100*300.00*1000	450	mm^2
Provide reinf. for bending		12	mm
dia		251	mm
spacing required	3.14/4*12.00^2*1000/450	200	mm
spacing provided		565	mm^2
area provided	3.14/4*12.00^2*1000/200.00		

Vertical reinforcement : outer face

		225	mm^2/m
Minimum vertical reinforcement as above		10	mm
dia		349	mm
spacing required	3.14/4*10.00^2*1000/225.00	200	mm
spacing provided		393	mm^2
area provided	3.14/4*10.00^2*1000/200.00		

Horizontal reinforcement :

Min % of steel		0.12%	
Thickness of section		300	mm
Horizontal reinf. on each face		360	mm^2
dia		10	mm
spacing require	3.14/4*10.00^2*1000/360.00	218	mm
spacing provided		200	mm
area provided	3.14/4*10.00^2*1000/200.00	393	mm^2

Approach Slab

Provide approach slab for a length		3.00	m
Total minimum steel		1.00%	
Thickness of approach slab		300	mm
Area of steel reinf. Per m width		3000	mm^2
Reinforcement in each layer in each direction		750	mm^2
dia		12	mm
spacing require	3.14/4*12.00^2*1000/750.00	151	mm
spacing provided		150	mm
area provided	3.14/4*12.00^2*1000/150.00	754	mm^2

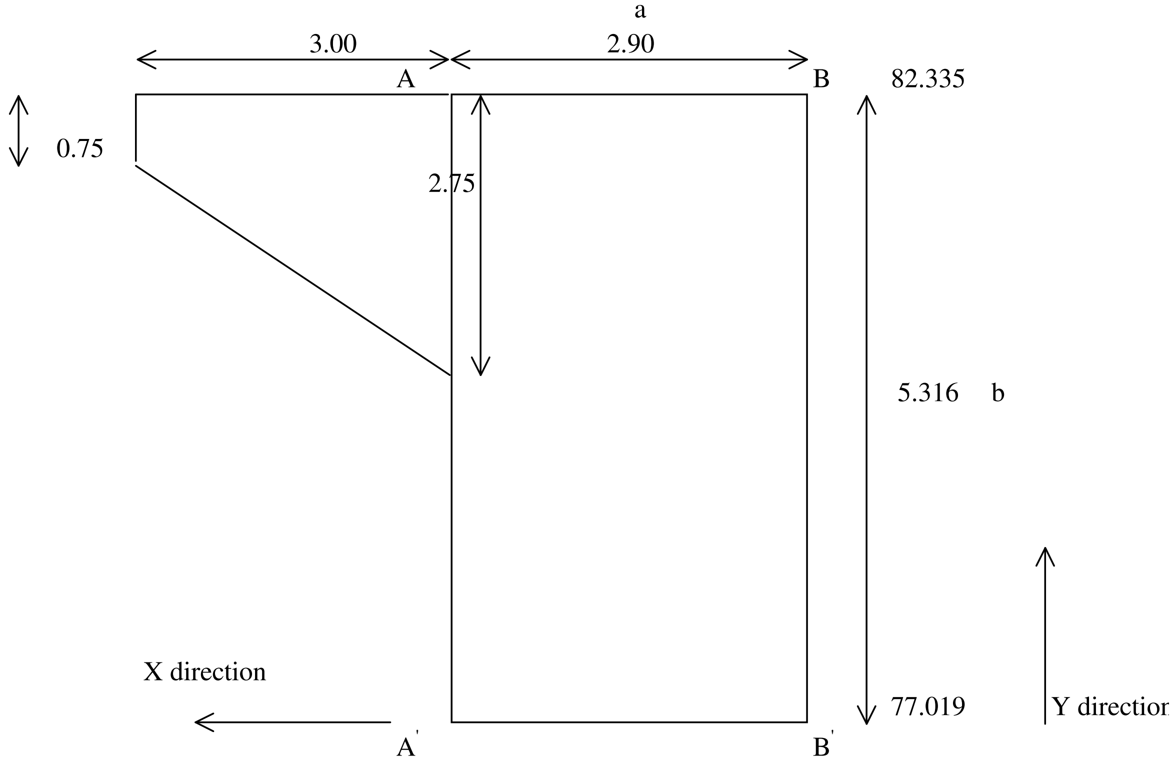
Abutment cap

The abutment cap is resting on abutment through out, provide minimum reinforcement.
As per clause 716.2.1 of IRC:78-1983

Total minimum steel		1.00%	
Thickness of abutment cap		300	mm
Area of steel reinf.		3000	mm^2
Reinforcement in each layer in each direction		750	mm^2
dia		12	mm
spacing require	3.14/4*12.00^2*1000/750.00	151	mm
spacing provided		150	mm
area provided	3.14/4*12.00^2*1000/150.00	754	mm^2

Also provide 8mm dia tor steel @ 100 mm c/c as mesh reinforcement in two layers.
One layer at 20mm and other at 100mm below top of abutment cap shall be provided.

DESIGN OF RETURNWALL



Width of Solid return wall (a)		=	2.90	
Width of Cantilever return wall		=	3.00	
Avg Height of Solid return wall (b)		=	5.316	
Height of Cantilever return at Tip		=	0.75	
Height of Cantilever return taper		=	2.00	
Height of Cantilever return at Root		=	2.75	
Thickness of Solid Return at farther end		=	0.5	
Thickness of Solid Return at Root		=	0.5	
Thickness of Solid Return at bottom		=	0.5	
Thickness of Solid Return at top		=	0.5	
Thickness of Cantilever return		=	0.5	
Unit wt of Soil		=	1.8	t/m^3
Grade of concrete		=	M 20	
	σ_{cbc}	=	680	t/m^2
	m	=	10	
	σ_{st}	=	20400	t/m^2
	k	=	0.250	
	j	=	0.917	
	R	=	77.9	t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.546	For a/b	=	0.5	β_1	=	0.631	β_2	=	0.632
			For a/b	=	0.75	β_1	=	1.246	β_2	=	1.186

a/b	=	0.546	β_1	=	0.743
			β_2	=	0.733

Live Load Surcharge:

$q = 0.2794 \times 1.8 \times 1.2 = 0.603504 \ t/m^2$

$\sigma_{bmax} = \dfrac{\beta_1 \times q \times b^2}{t^2}$

$\sigma_{amax} = \beta_2 \dfrac{\times q \times b^2}{t^2}$

$\sigma_{bmax} = \dfrac{0.742986 \times 0.603504 \times 28.26}{0.25} = 50.68635 \ t/m^2$

For 1000 mm of width

$Z = \dfrac{1000 \times 250000}{6} = 41666667 \ mm^3 = 0.041667 \ m^3$

Hence Moment /m width along Y direction

M_Y /m width $= 50.68635 \times 0.041667 = 2.111931$ t-m/m

$\sigma_{amax} = \dfrac{0.732879 \times 0.603504 \times 28.26}{0.25} = 49.99681 \ t/m^2$

For 1000 mm of width

$Z = \dfrac{1000 \times 250000}{6} = 41666667 \ mm^3 = 0.041667 \ m^3$

Hence Moment /m width along X direction

M_X /m width $= 49.99681 \times 0.041667 = 2.0832$ t-m/m

Case (2) For Triangular loading due to earth pressure

$a/b = 0.545523$ For $a/b = 0.5$ $\beta_1 = 0.328$ $\beta_2 = 0.2$

For $a/b = 0.75$ $\beta_1 = 0.537$ $\beta_2 = 0.276$

$a/b = 0.545523$ $\beta_1 = 0.366057$

$\beta_2 = 0.213839$

Earth pressure:

$q = 0.2794 \times 1.8 \times 5.316 = 2.673523 \ t/m^2$

$\sigma_{bmax} = \dfrac{\beta_1 \times q \times b^2}{t^2}$

$\sigma_{amax} = \beta_2 \dfrac{\times q \times b^2}{t^2}$

$\sigma_{bmax} = \dfrac{0.366057 \times 2.673523 \times 28.26}{0.25} = 110.6274 \ t/m^2$

For 1000 mm of width

$Z = \dfrac{1000 \times 250000}{6} = 41666667 \ mm^3 = 0.041667 \ m^3$

Hence Moment /m width along Y direction

M_Y /m width = 110.6274 x 0.041667 = **4.609475** t-m/m

$$\sigma_{amax} = \frac{0.213839 \times 2.673523 \times 28.26}{0.25} = 64.62502 \text{ t/m}^2$$

For 1000 mm of width

$$Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3$$
$$= 0.041667 \text{ m}^3$$

Hence Moment /m width along X direction

M_X /m width = 64.62502 x 0.041667 = **2.692709** t-m/m

Total Moment in Solid Return /m height = **4.776** t-m/m
Along X-direction

Total Moment in Solid Return /m width = **6.721** t-m/m
Along Y-direction

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A′

M = 0.2794 x 1.2 x 1.8 x 0.75 x 3.00 x 1.50
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 3.00 x 1.50
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (3.00 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (3.00 - X)
= 2.036826 + 0.636508 + 0.11176 x 6.75 + 0.653796 x 4.5
= **6.37** t-m

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

Effective depth available = 500 - 50 - 20 - 6 = 424 mm

$M = R \times b \times d^2$
= 77.91667 x 2.750 x 0.179776 = 38.52075 t-m

$$A_{st} = \frac{6.37 \times 10\text{^}6}{20400 \times 0.916667 \times 0.424} = 803.3745 \text{ mm}^2$$

A_{st}/m = 292.1362 mm^2/m

Provide 10 mm dia @ 200 mm c/c providing 393 mm^2 on earth face.

Provide 8 mm dia @ 200 mm c/c providing 251 mm^2 on other face.

Along Horizantal direction.

Design of Solid Return:

Moment due to Cantilever Return:

Moment due to Earth pressure at face B-B′

$$M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 3.00 \times 4.40$$
$$+ 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 3.00 \times 4.40$$
$$+ 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times dx \times (5.90 - X)$$
$$+ 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times (5.90 - X)$$

$$= 5.97469 + 1.867091 + 0.11176 \times 32.85 + 0.653796 \times 17.55$$

$$= \mathbf{22.99} \text{ t-m}$$

Moment in Solid Return /m height = $4.775909 + \frac{22.98722}{5.316}$ = 9.10 t-m/m

Moment in Solid Return /m width = = 6.72 t-m/m

Design of face B-B′

Moment in Solid Return /m height = **9.10** t-m/m

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 20 - 10 = 420 mm

$$M = R \times b \times d^2$$
$$= 77.91667 \times 1.000 \times 0.1764 = 13.74 \text{ t-m}$$

$$A_{st} = \frac{9.10 \times 10\text{^}6}{20400 \times 0.916667 \times 0.42} = 1159 \text{ mm}^2$$

$$A_{st}/m = 1159 \text{ mm}^2/m$$

Provide 16 mm dia @ 150 mm c/c providing 1340 mm^2 on earth face.

Provide 10 mm dia @ 150 mm c/c providing 524 mm^2 on other face.

Along Horizantal direction.

Design of face A′-B′

Moment in Solid Return /m width = **6.721407** t-m/m

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 0 - 10 = 440 mm

$$M = R \times b \times d^2$$
$$= 77.91667 \times 1.000 \times 0.1936 = 15.08 \text{ t-m}$$

$$A_{st} = \frac{6.721407 \times 10\text{^}6}{20400 \times 0.916667 \times 0.44} = 817 \text{ mm}^2$$

$$A_{st}/m = 817 \text{ mm}^2/m$$

Provide 16 mm dia @ 150 mm c/c providing 1340 mm^2 on earth face.

Provide 10 mm dia @ 150 mm c/c providing 524 mm^2 on other face.

Along Vertical direction.

```
       ABUT SHAFT .case 1
10-03-07

       Depth of Section             =     1.000 m
       Width of Section             =    13.860 m

  along width-compression face- no of bar:  65   tension face- no of bar:  65
  Dia (mm)                                  16                             25
  Cover (cm)                               7.50                          7.50
  along depth-compression face- no of bar:   6   tension face- no of bar:   6
  Dia (mm)                                  16                             16
  Cover  (cm)                              7.50                          7.50

       Modular Ratio : Compression  =      10.0
       Modular Ratio : Tension      =      10.0
       Allowable Concrete Stress    =     68.00 Kg/cm^2
       Allowable Steel Stress       =   2000.00 Kg/cm^2

       Axial Load                   =   425.586 T
       Mxx                          =   311.554 Tm
       Myy                          =   212.503 Tm


       Intercept of Neutral axis : X axis : = 103.867 m
                                 : y axis : =    .332 m

                 Concrete Stress Governs Design



       Stress in Concrete due to Loads =   28.65 Kg/cm^2
       Stress in Steel    due to Loads =  549.59 Kg/cm^2
       Percentage of Steel             =     .34 %


       ABUT SHAFT .case 2
10-03-07

       Depth of Section             =     1.000 m
       Width of Section             =    13.860 m

       Modular Ratio : Compression  =      10.0
       Modular Ratio : Tension      =      10.0
       Allowable Concrete Stress    =     68.00 Kg/cm^2
       Allowable Steel Stress       =   2000.00 Kg/cm^2

       Axial Load                   =   126.700 T
       Mxx                          =   253.300 Tm
       Myy                          =      .010 Tm


       Intercept of Neutral axis : X axis : = ******* m
                                 : y axis : =    .221 m

                    Steel Stress Governs Design

       Stress in Concrete due to Loads =   22.56 Kg/cm^2
       Stress in Steel    due to Loads =  719.01 Kg/cm^2
       Percentage of Steel             =     .34 %
```

Dead load Reaction From Super Structure (Stadd Output)

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	1	0	402.09	0	0	0	0
152	1	0	37.92	0	0	0	0
153	1	0	248.15	0	0	0	0
154	1	0	237.04	0	0	0	0
155	1	0	231.32	0	0	0	0
156	1	0	212.41	0	0	0	0
157	1	0	157.28	0	0	0	0
Total Dead Load Reaction in KN			**1526.21**				

Super imposed Dead load Reaction From Super Structure (Stadd Output)

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	2	0	111.1	0	0	0	0
152	2	0	29.6	0	0	0	0
153	2	0	19.53	0	0	0	0
154	2	0	20.38	0	0	0	0
155	2	0	23.75	0	0	0	0
156	2	0	28.89	0	0	0	0
157	2	0	41.08	0	0	0	0
Total Dead Load Reaction in KN			**274.33**				

Live load Reaction From Super Structure For Class 3A Lane (Stadd Output)

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	34	0	162.91	0	0	0	0
152	34	0	-1.24	0	0	0	0
153	34	0	254.41	0	0	0	0
154	34	0	222.32	0	0	0	0
155	34	0	186.67	0	0	0	0
156	34	0	165.09	0	0	0	0
157	34	0	121.87	0	0	0	0
Total Reaction with Impact in KN			**1112.030**				

Live load Reaction From Super Structure For Class 70RW withought A lane (Stadd Output

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	98	0	102.89	0	0	0	0
152	98	0	-2.64	0	0	0	0
153	98	0	331.04	0	0	0	0
154	98	0	365.44	0	0	0	0
155	98	0	54.3	0	0	0	0
156	98	0	-11.61	0	0	0	0
157	98	0	-6.44	0	0	0	0
Total Reaction with Impact in KN			**832.98**				

Live load Reaction From Super Structure For Class A withought 70RW (Stadd Output

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	174	0	10.6	0	0	0	0
152	174	0	-5.13	0	0	0	0
153	174	0	20.57	0	0	0	0
154	174	0	31.94	0	0	0	0
155	174	0	142.3	0	0	0	0
156	174	0	145.29	0	0	0	0
157	174	0	5.36	0	0	0	0
Total Reaction with Impact in KN			**350.93**				

Live load Reaction From Super Structure For Class 70RT withought A lane (Stadd Output

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	236	0	-9.66	0	0	0	0
152	236	0	1.57	0	0	0	0
153	236	0	273.82	0	0	0	0
154	236	0	392.73	0	0	0	0
155	236	0	92.05	0	0	0	0
156	236	0	-23.35	0	0	0	0
157	236	0	-0.63	0	0	0	0
Total Reaction with Impact in KN			**726.53**				

Live load Reaction From Super Structure For Class A withought 70RT (Stadd Output)

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
1	314	0	8.94	0	0	0	0
152	314	0	-5.74	0	0	0	0
153	314	0	19.83	0	0	0	0
154	314	0	29.84	0	0	0	0
155	314	0	134.44	0	0	0	0
156	314	0	150.93	0	0	0	0
157	314	0	10.27	0	0	0	0
Total Reaction with Impact in KN			**348.51**				

```
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     2. START JOB INFORMATION
     3. ENGINEER DATE 09-DEC-06
     4. END JOB INFORMATION
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     6. PAGE LENGTH 1000
     7. UNIT METER KN
     8. JOINT COORDINATES
     9. 1 1 0 0; 8 12.55 0 0; 17 5.62 0 12; 136 0.538002 0 0; 137 -6.39022 0
12
    10. 152 -5.9282 0 12; 153 -0.1547 0 2; 154 -1.3094 0 4; 155 -2.4641 0 6
    11. 156 -3.6188 0 8; 157 -4.7735 0 10; 158 11.39 0 2; 159 10.24 0 4; 160
9.08 0 6
    12. 161 7.93 0 8; 162 6.77 0 10; 163 -0.6167 0 2; 164 -1.77141 0 4
    13. 165 -2.92611 0 6; 166 -4.08081 0 8; 167 -5.23552 0 10; 168 -4.7735 0
12
    14. 169 -3.6188 0 10; 170 -3.6188 0 12; 171 -2.4641 0 8; 172 -2.4641 0 10
    15. 173 -2.4641 0 12; 174 -1.3094 0 6; 175 -1.3094 0 8; 176 -1.3094 0 10
    16. 177 -1.3094 0 12; 178 -0.1547 0 4; 179 -0.1547 0 6; 180 -0.1547 0 8
    17. 181 -0.1547 0 10; 182 -0.1547 0 12; 183 1 0 2; 184 1 0 4; 185 1 0 6;
186 1 0 8
    18. 187 1 0 10; 188 1 0 12; 189 5.6226 0 10; 190 5.6226 0 8; 191 5.6226 0
6
    19. 192 5.6226 0 4; 193 5.6226 0 2; 194 5.6226 0 0; 195 6.7773 0 8; 196
6.7773 0 6
    20. 197 6.7773 0 4; 198 6.7773 0 2; 199 6.7773 0 0; 200 7.932 0 6; 201
7.932 0 4
    21. 202 7.932 0 2; 203 7.932 0 0; 204 9.0867 0 4; 205 9.0867 0 2; 206
9.0867 0 0
    22. 207 10.2414 0 2; 208 10.2414 0 0; 209 11.3961 0 0; 210 3.3113 0 0
    23. 211 3.3113 0 2; 212 3.3113 0 4; 213 3.3113 0 6; 214 3.3113 0 8
    24. 215 3.3113 0 10; 216 3.3113 0 12; 217 13.012 0 0; 218 11.862 0 2
    25. 219 10.712 0 4; 220 9.562 0 6; 221 8.412 0 8; 222 7.262 0 10; 223
6.112 0 12
    26.
*******************************************************************************
*
    27.
*******************************************************************************
*
    28. MEMBER INCIDENCES
    29. 1 1 210; 255 136 1; 257 136 163; 285 152 168; 286 1 153; 287 137 152
    30. 288 8 158; 289 153 154; 290 154 155; 291 155 156; 292 156 157; 293 157
152
    31. 294 158 159; 295 159 160; 296 160 161; 297 161 162; 298 162 17; 299
153 183
    32. 300 154 178; 301 155 174; 302 156 171; 303 157 169; 304 163 164; 305
164 165
    33. 306 165 166; 307 166 167; 308 167 137; 309 167 157; 310 166 156; 311
165 155
    34. 312 164 154; 313 163 153; 314 168 170; 315 157 168; 316 169 172; 317
156 169
    35. 318 170 173; 319 169 170; 320 171 175; 321 155 171; 322 172 176; 323
171 172
    36. 324 173 177; 325 172 173; 326 174 179; 327 154 174; 328 175 180; 329
174 175
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    37. 330 176 181; 331 175 176; 332 177 182; 333 176 177; 334 178 184; 335
153 178
    38. 336 179 185; 337 178 179; 338 180 186; 339 179 180; 340 181 187; 341
180 181
    39. 342 182 188; 343 181 182; 344 183 211; 345 1 183; 346 184 212; 347 183
184
    40. 348 185 213; 349 184 185; 350 186 214; 351 185 186; 352 187 215; 353
186 187
    41. 354 188 216; 355 187 188; 356 189 162; 357 17 189; 358 190 195; 359
189 190
    42. 360 191 196; 361 190 191; 362 192 197; 363 191 192; 364 193 198; 365
192 193
    43. 366 194 199; 367 193 194; 368 195 161; 369 162 195; 370 196 200; 371
195 196
    44. 372 197 201; 373 196 197; 374 198 202; 375 197 198; 376 199 203; 377
198 199
    45. 378 200 160; 379 161 200; 380 201 204; 381 200 201; 382 202 205; 383
201 202
    46. 384 203 206; 385 202 203; 386 204 159; 387 160 204; 388 205 207; 389
204 205
    47. 390 206 208; 391 205 206; 392 207 158; 393 159 207; 394 208 209; 395
207 208
    48. 396 209 8; 397 158 209; 398 210 194; 399 211 193; 400 210 211; 401 212
192
    49. 402 211 212; 403 213 191; 404 212 213; 405 214 190; 406 213 214; 407
215 189
    50. 408 214 215; 409 216 17; 410 215 216; 411 217 218; 412 218 219; 413
219 220
    51. 414 220 221; 415 221 222; 416 222 223; 417 17 223; 418 8 217; 419 158
218
    52. 420 159 219; 421 160 220; 422 161 221; 423 162 222
    53. DEFINE MATERIAL START
    54. ISOTROPIC MATERIAL1
    55. E 2.5E+007
    56. POISSON 0.15
    57. DENSITY 24
    58. DAMP 7.90066E+033
    59. END DEFINE MATERIAL
    60. MEMBER PROPERTY INDIAN
    61. 299 TO 303 309 TO 313 316 320 322 326 328 330 334 336 338 340 344 346
348 -
    62. 350 352 356 358 360 362 364 368 370 372 374 378 380 382 386 388 392
399 401 -
    63. 403 405 407 419 TO 423 PRIS YD 0.85 ZD 1
    64. 1 255 285 287 314 318 324 332 342 354 366 376 384 390 394 396 398 409
417 -
    65. 418 PRIS YD 0.85 ZD 0.5
    66. 257 286 288 TO 298 304 TO 308 411 TO 416 PRIS AX 0.001 IX 0.001 IZ
0.001
    67. 315 317 319 321 323 325 327 329 331 333 335 337 339 341 343 369 371
373 375 -
    68. 377 379 381 383 385 387 389 391 393 395 397 PRIS YD 0.85 ZD 1.15
    69. 400 402 404 406 408 410 PRIS YD 0.85 ZD 2.31
    70. 345 347 349 351 353 355 357 359 361 363 365 367 PRIS YD 0.85 ZD 1.385
    71. SUPPORTS
    72. 1 152 TO 157 PINNED
    73. 8 17 158 TO 162 FIXED BUT FX FZ MX MY MZ
```

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    74. CONSTANTS
    75. MATERIAL MATERIAL1 MEMB 1 255 257 285 TO 423
    76. MEMBER RELEASE
    77. 417 TO 422 START FY MZ
    78. 255 287 309 TO 313 END MZ
    79. DEFINE MOVING LOAD
    80. TYPE 1 LOAD 13.5 13.5 57 57 34 34 34 34
    81. DIST 1.1 3.2 1.2 4.3 3 3 3 WID 1.8
    82. TYPE 2 LOAD 40 60 60 85 85 85 85
    83. DIST 3.96 1.52 2.13 1.37 3.05 1.37 WID 1.93
    84. TYPE 3 LOAD 35 35 35 35 35 35 35 35 35 35
    85. DIST 0.457 0.457 0.457 0.457 0.457 0.457 0.457 0.457 0.457 WID 2.06
    86. LOAD 1 DEAD LOAD(SLAB LOAD)
    87. FLOOR LOAD
    88. YRANGE 0 0 FLOAD -20.4 XRANGE -6.92822 13.55 ZRANGE 0 12
    89. LOAD 2 DEAD LOAD(SIDL)
    90. ************CRASH BARIEAR LOAD
    91. MEMBER LOAD
    92. 1 255 285 287 314 318 324 332 342 354 366 376 384 390 394 396 398 409
417 -
    93. 418 UNI GY -10
    94. ****************WEARING COAT*2
    95. FLOOR LOAD
    96. YRANGE 0 0 FLOAD -2 XRANGE -6.92822 13.55 ZRANGE 0 13.86
    97. LOAD 3 LIVE LOAD
    98. **************THREE LANE OF CLASS A
    99. LOAD GENERATION 75
   100. TYPE 1 -26 0 2.7 XINC 0.5
   101. TYPE 1 -26 0 6.2 XINC 0.5
   102. TYPE 1 -26 0 9.7 XINC 0.5
   103. *******************70R WHEELED WITHOUGHT CLASS A
   104. LOAD GENERATION 65
   105. TYPE 2 -20 0 4.06 XINC 0.5
   106. *****************ONE LANE OF CLASS A WITHOUGHT 70R WHEELED
   107. LOAD GENERATION 75
   108. TYPE 1 -26 0 7.74 XINC 0.5
   109. ********************70R TRACKED TRAIN WITHOUGHT CLASS A
   110. LOAD GENERATION 65
   111. TYPE 3 -15 0 4.18 XINC 0.5
   112. *****************ONE LANE OF CLASS A WITHOUGHT 70R TRACK
   113. LOAD GENERATION 75
   114. TYPE 1 -26 0 7.85 XINC 0.5
   115. PERFORM ANALYSIS
   116. LOAD LIST 1
   117. PRINT SUPPORT REACTION LIST 1 152 TO 157
        SUPPORT  REACTION LIST     1

   SUPPORT REACTIONS -UNIT KN   METE    STRUCTURE TYPE = FLOOR
   -----------------
```

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	0.00	402.09	0.00	0.00	0.00	0.00
152	1	0.00	37.92	0.00	0.00	0.00	0.00
153	1	0.00	248.15	0.00	0.00	0.00	0.00
154	1	0.00	237.04	0.00	0.00	0.00	0.00

```
   155    1       0.00    231.32       0.00       0.00       0.00       0.00
   156    1       0.00    212.41       0.00       0.00       0.00       0.00
   157    1       0.00    157.28       0.00       0.00       0.00       0.00
  118. *PRINT SUPPORT REACTION LIST 8 17 158 TO 162
  119. *********************THREE LANE OF CLASS A
  120. *LOAD LIST 2 TO 77
  121. LOAD LIST 34
  122. PRINT SUPPORT REACTION LIST 1 152 TO 157
       SUPPORT  REACTION LIST     1

  SUPPORT REACTIONS -UNIT KN   METE    STRUCTURE TYPE = FLOOR
  -----------------
JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z

     1   34      0.00    148.94       0.00       0.00       0.00       0.00
   152   34      0.00     -2.94       0.00       0.00       0.00       0.00
   153   34      0.00    177.76       0.00       0.00       0.00       0.00
   154   34      0.00    171.23       0.00       0.00       0.00       0.00
   155   34      0.00    127.75       0.00       0.00       0.00       0.00
   156   34      0.00    103.22       0.00       0.00       0.00       0.00
   157   34      0.00     63.95       0.00       0.00       0.00       0.00

  123. *****************70R WHEELED WITHOUGHT CLASS A
  124. *LOAD LIST 78 TO 142
  125. LOAD LIST 98
  126. PRINT SUPPORT REACTION LIST 1 152 TO 157
       SUPPORT  REACTION LIST     1
  SUPPORT REACTIONS -UNIT KN   METE    STRUCTURE TYPE = FLOOR
  -----------------
JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z

     1   98      0.00     91.72       0.00       0.00       0.00       0.00
   152   98      0.00     -2.47       0.00       0.00       0.00       0.00
   153   98      0.00    290.68       0.00       0.00       0.00       0.00
   154   98      0.00    320.30       0.00       0.00       0.00       0.00
   155   98      0.00     39.47       0.00       0.00       0.00       0.00
   156   98      0.00     -7.77       0.00       0.00       0.00       0.00
   157   98      0.00     -5.51       0.00       0.00       0.00       0.00
  127. *****************ONE LANE OF CLASS A WITHOUGHT 70R WHEELED
  128. *LOAD LIST 143 TO 217
  129. LOAD LIST 174
  130. PRINT SUPPORT REACTION LIST 1 152 TO 157
       SUPPORT  REACTION LIST     1

JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z
     1  174      0.00      9.83       0.00       0.00       0.00       0.00
   152  174      0.00     -3.54       0.00       0.00       0.00       0.00
   153  174      0.00     20.52       0.00       0.00       0.00       0.00
   154  174      0.00     34.18       0.00       0.00       0.00       0.00
   155  174      0.00     97.00       0.00       0.00       0.00       0.00
   156  174      0.00     82.24       0.00       0.00       0.00       0.00
   157  174      0.00      3.11       0.00       0.00       0.00       0.00

  131. ********************70R TRACKED TRAIN WITHOUGHT CLASS A
  132. *LOAD LIST 218 TO 282
  133. LOAD LIST 236
  134. PRINT SUPPORT REACTION LIST 1 152 TO 157
```

SUPPORT REACTION LIST 1

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	236	0.00	-8.64	0.00	0.00	0.00	0.00
152	236	0.00	1.49	0.00	0.00	0.00	0.00
153	236	0.00	246.27	0.00	0.00	0.00	0.00
154	236	0.00	347.84	0.00	0.00	0.00	0.00
155	236	0.00	91.82	0.00	0.00	0.00	0.00
156	236	0.00	-22.94	0.00	0.00	0.00	0.00
157	236	0.00	-0.70	0.00	0.00	0.00	0.00

```
135. ******************ONE LANE OF CLASS A WITHOUGHT 70R TRACK
136. *LOAD LIST 283 TO 357
137. LOAD LIST 314
138. PRINT SUPPORT REACTION LIST 1 152 TO 157
```

SUPPORT REACTION LIST 1

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	314	0.00	8.87	0.00	0.00	0.00	0.00
152	314	0.00	-3.84	0.00	0.00	0.00	0.00
153	314	0.00	19.96	0.00	0.00	0.00	0.00
154	314	0.00	32.51	0.00	0.00	0.00	0.00
155	314	0.00	92.98	0.00	0.00	0.00	0.00
156	314	0.00	85.61	0.00	0.00	0.00	0.00
157	314	0.00	5.74	0.00	0.00	0.00	0.00

```
139. FINISH
```

DESIGN OF SUPERSTRUCTURE

For Design of Superstructure of solid slab 10.0 m with a skew of 30^0 refer MOST STANDARD Drawing titled "STANDARD DRAWING FOR ROAD BRIDGES (R.C.C Solid slab superstructure (150 and 300 Skew) span 4.0 m to 10.0 m 9With and without footpath)" Drg. No. SD/172.

MINOR BRIDGE AT CH:29/500

DESIGN OF SUBSTRUCTURE

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete	Abutment	= M - 30
	Pile cap & Pile	= M - 35
Grade of reinforcement steel		= Fe - 415
Centre to Centre distance of A / Expansion joints		= 21.600 m
Centre to Centre distance of Bearing		= 21.000 m
Depth of superstructure		= 2100 mm
Thickness of wearing coat		= 56.00 mm
Formation level along C of carriage way		= 98.300 m
Height of bearing and Pedestal		= 0.350 m
L.W.L.		= 90.145 m
H.F.L		= 94.609 m
M.S.L		= 92.569
Abutment cap top level		= 95.794 m
Pile cap top level		= 89.145 m
Depth of pile cap		= 1800 mm
Pile cap bottom level		= 87.345 m
Diameter of bored cast-in-situ piles		= 1200 mm
Length of piles above rock level		= 0.000 m
Length of piles below rock level		= 0.000 m
Total length of pile		= 13.500 m
Load carrying capacity of each pile		= **207** Tons

Live Load **(a)** Class A three Lane

(b) Class 70R wheeled + Class A single lane

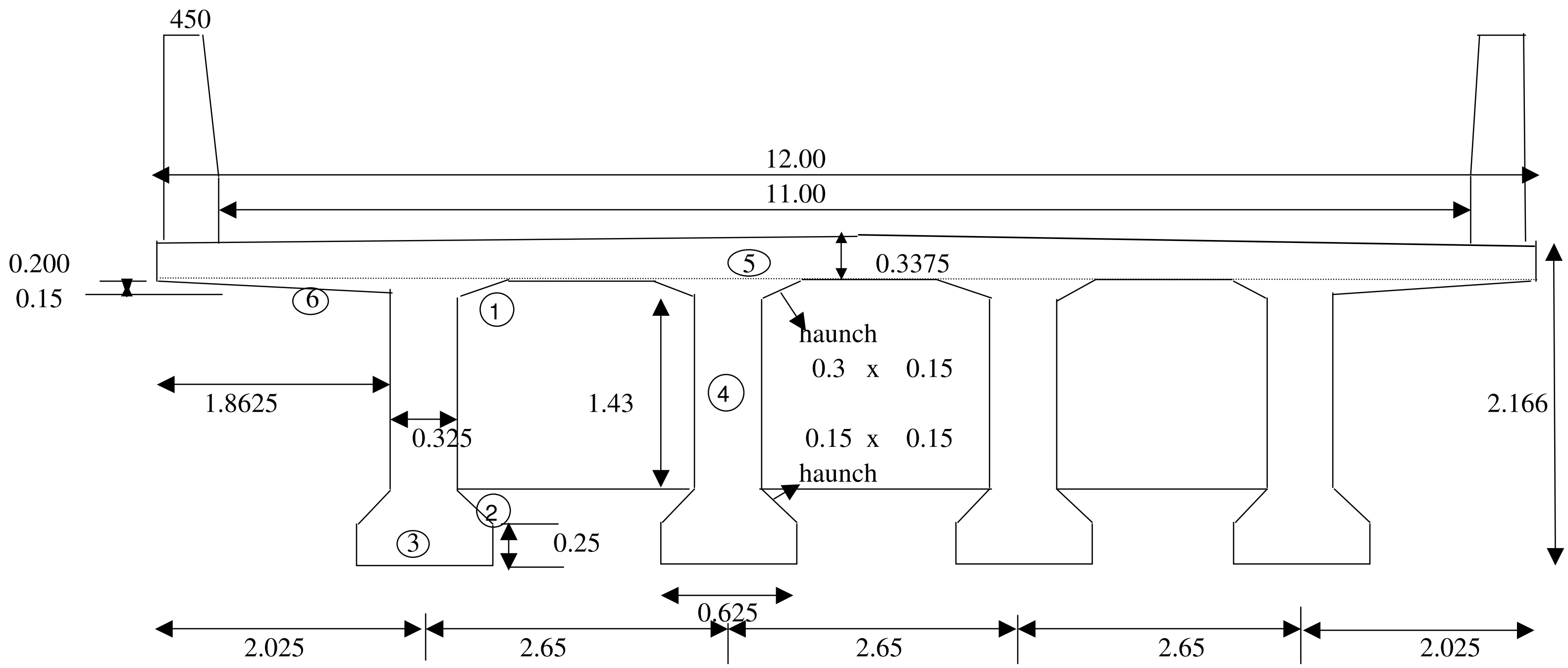
Bearing Elastometric

Seismic zone = II

1 IRC : 6 - 2000

2 IRC : 21 - 2000

3 IRC : 78 - 2000



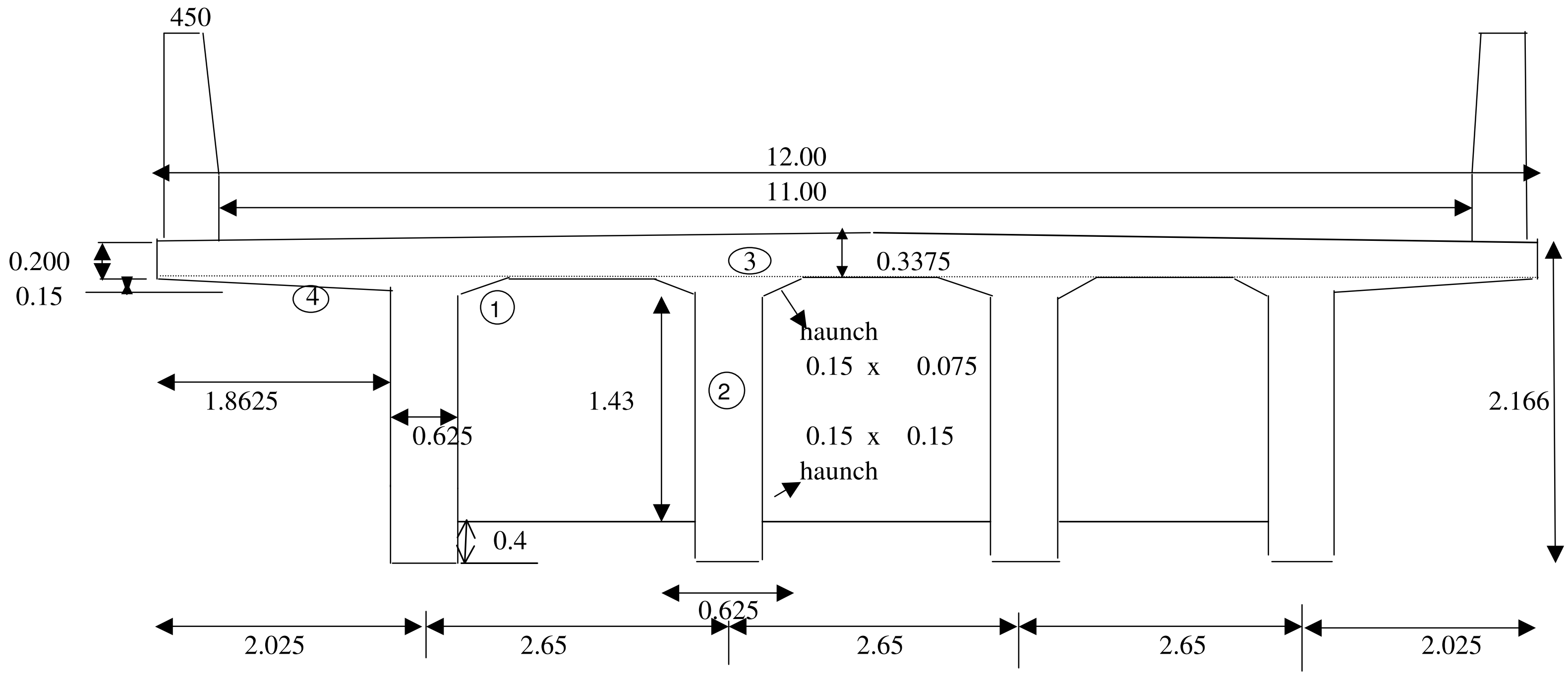
I ***cross sectional area at mid span***

1	top haunch	0.3	x	0.15	x	6	=	0.27	m^2
2	bottom haunch	0.15	x	0.15	x	8	=	0.18	m^2
3	bottom rect	0.25	x	0.625	x	4	=	0.625	m^2
4	web	0.325	x	1.4285	x	4	=	1.8571	m^2
5	slab	0.2688	x	12.00	x	1	=	3.23	m^2
6		0.15	x	1.8625	x	2	=	0.2794	m^2
							=	**6.44**	m^2

intermediate cross gitder

1.43 x 2.325 x 0.3 x 3 x 1 = 2.99 m^3

II ***cross sectional area at end section***

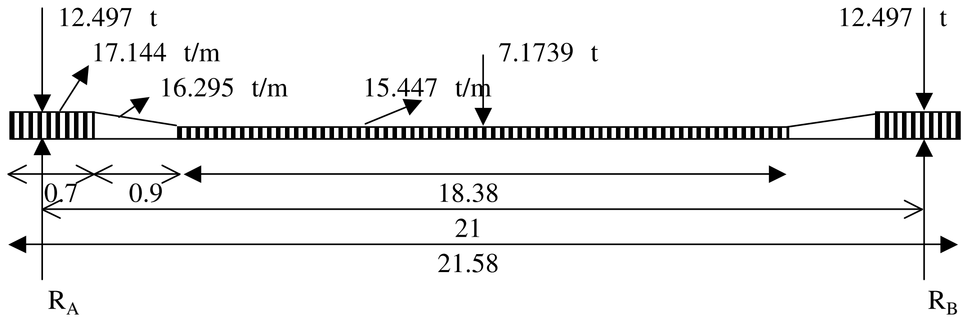


I ***cross sectional area at end span***

1	top haunch	0.15	x	0.075	x	6	=	0.0675 m^2
2	web	0.625	x	1.4285	x	4	=	3.5713 m^2
3		0.2688	x	12.00	x	1	=	3.23 m^2
4		0.15	x	1.8625	x	2	=	0.2794 m^2
							=	7.14 m^2

End cross girder

1.43 x 2.025 x 3 x 0.3 x 2 = 5.2069 m^3



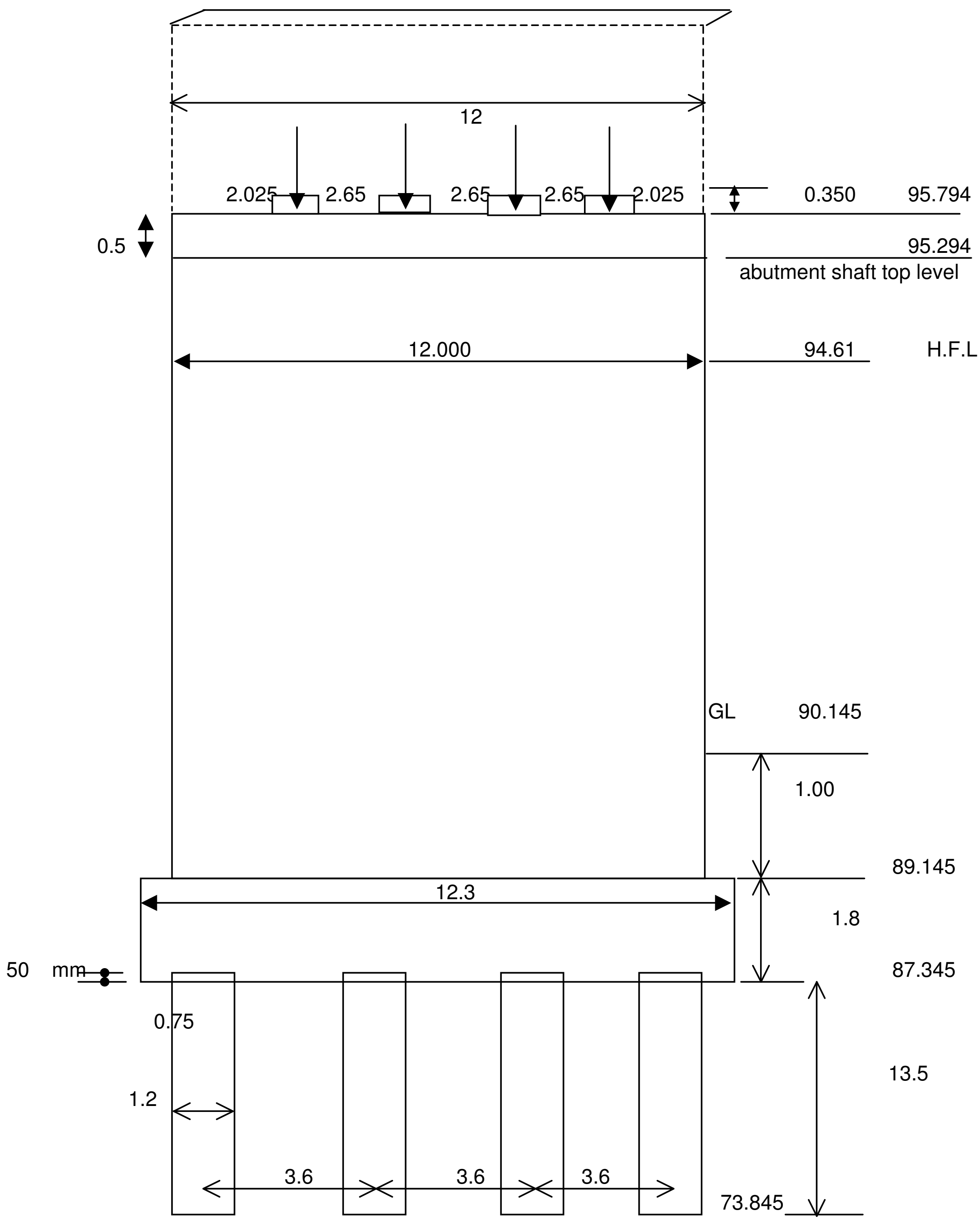
R_A = 184.71 t

S.I.D.L

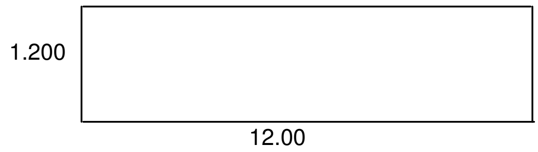
1 Wearing Coat	@ 1.3552 t/m		=	1.3552	t/m
2 Crash barrier	@ 1.000 t/m				
(provide one side enter 1,bothsides enter 2 , otherwise enter 0)		2	=	2.00	t/m
				3.36	t/m

R_A = 36.236 t

Total DL + SIDL reaction = 220.95 t


12
2.025
2.65
2.65
2.65
2.025
0.350
95.794
0.5
95.294
abutment shaft top level
12.000
94.61
H.F.L
GL
90.145
1.00
89.145
12.3
1.8
50 mm
87.345
0.75
13.5
1.2
3.6
3.6
3.6
73.845

Loads from substructure



1 DRY Condition

1 **Wt of abutment cap**

Rectangular

	12.00	x	0.50	x	1.20	x	2.4		=	**17.28**	t
Dirtwall	12.00	x	2.206	x	0.30	x	2.4		=	19.0598	t
a *Approach slab*	12.00	x	0.3	x	1.75	x	2.4		=	15.12	t
									=	**34.1798**	t

2 *Wt of Abutment Shaft*

12	x	1.2	x	6.15	x	2.4	=	212.509 t
0.7854	x	0	x	6.15	x	2.4	=	0 t
							=	**212.509** t

3 *Wt of Pile cap*

12.3 x 8.7 x 1.8 x 2.4 = **462.283** t

4 *Weight due to Earth*

$\left(12.3 \times 8.7 - \left(12 \times 1.2\right)\right) \times 1.8 \times 1 =$ 166.698 t

H.F.L Condition

Rectangular

	12	x	0.5	x	1.20	x	2.4		=	**17.28**	t
Dirtwall	12	x	2.206	x	0.30	x	2.4		=	19.0598	t
a *Approach slab*	12	x	0.3	x	1.75	x	2.4		=	15.12	t
									=	**34.1798**	t

2 *Wt of Abutment Shaft*

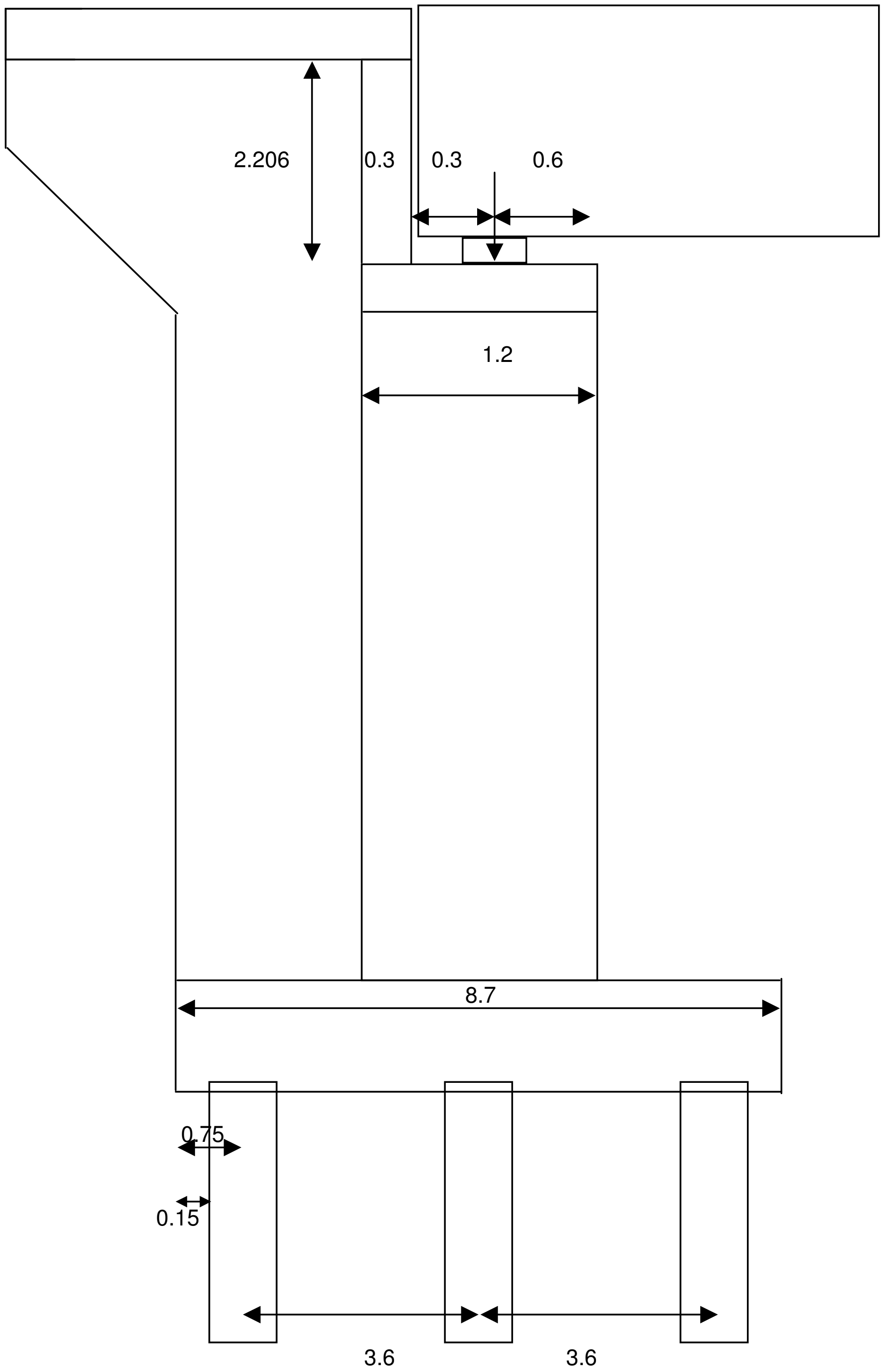
12	x	1.2	x	0.69	x	2.4	=	23.6736 t
12	x	1.2	x	5.46	x	1.4	=	110.154 t
0.7854	x	0	x	0.69	x	2.4	=	0 t
0.7854	x	0	x	5.46	x	1.4	=	0 t
							=	**133.828** t

3 *Wt of Pile cap*

12.3 x 8.7 x 1.8 x 1.4 = **269.665** t

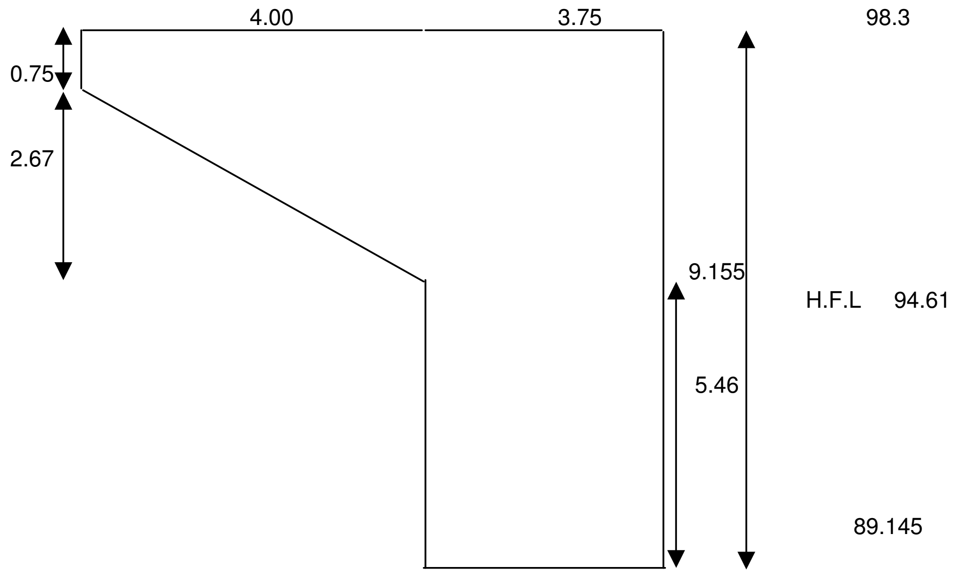
5 *Weight due to Earth*

12.3 x 8.7 - 12 x 1.2 x 1 x 1 = 92.61 t



2.206
0.3
0.3
0.6
1.2
8.7
0.75
0.15
3.6
3.6

Wt of Returnwall on pile cap



Height of returnwall	=	9.16	m
width of returnwall	=	0.45	m
wt of returnwall (dry condition)	=	**46.08**	t
wt of returnwall (H.F.L condition)	=	12.91	t
	=	23.95	t
	=	**36.86**	t

Longitudinal Force :

For span of 21.60 m.

Longitudinal translation due to creep, shrinkage & temperature = 0.0005

Horizontal movement = $0.0005 \times \frac{21.60}{2} \times 1000$ = 5.400 mm

Size of bearing = 500 x 250 x 50 mm

Strain in bearing = $\frac{5.400}{50}$ = 0.108

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.108 x 1.0 x 500 x 250 = 13500 N = 1.376 t

Total shear force for 4 bearings (with 5 % increase)

= 1.376 x 4 x 1.05 = 5.780 t

Refer IRC : 6 clause 214.5.1.5;

10 % increase for variation in movement of span

Total shear force = 1.1 x 5.780 = 6.358 t

As per clause 214.2 of IRC:6, horizontal braking force F_h, for each span is:

For Class A Single lane : F_h = 0.2 x 55.4 = 11.080 t

For class 70R wheeled : F_h = 0.2 x 100 = 20.000 t

For class A 3 lane F_h = 0.2 x 55.4 + 0.05 x 55.4 = 13.85 t

For class 70R wheeled +class A 1 lane

F_h = 0.2 x 100 + 0.05 x 55.4 = 22.77 t

For class A 3 lane

Longitudinal horizontal force = $\frac{13.850}{2}$ + 6.358 = 13.28 t Say, 14.0 t

For class 70R wheeled +class A 1 lane

Longitudinal horizontal force = $\frac{22.770}{2}$ + 6.358 = 17.7 t Say, 18.0 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 3 lane	**14.00**
70R+class A 1lane	**18.00**
spandislodged	**0.00**

Moment in Transverse Direction

Eccentricity of vertical load in transverse direction,

(a) Class A 3 lane = = 0.7

(b) Class 70R + Class A 1 lane = 1.721

Live load	Vertical load	Longitudinal moment	Transverse moment
70R+class A 1lane	112.38	0.00	193.39
Class A 3 lane	107.19	0.00	75.03

SUMMARY OF LOAD ON ABUTMENT FROM SUPERSTRUCTURE

Load	Vertical load	Horizontal force	Longitudinal moment	Transverse moment
Dead load	184.71	-	-	-
SIDL	36.24	-	-	-
Live Load				
class A 3lane	107.19	14.00	0.00	75.03
70RW + classA1lane	112.38	18.00	0.00	193.39
Span dislodged condition	0.00	0.00	0.00	0.00

Case-I(1) :DRY. condition with L.L:

L.W.L. = 90.145 m
Pile cap top level = 89.145 m
Pile cap bottom level = 87.345 m

Vertical load on Pile foundation;
Load from superstructure

S.I.D.L. + D.L. = 36.24 + 184.71 = 220.95 t
\+ **Live load** = 112.38 t
333.33 t Say, 334.00 t

Load from substructure & pile cap =
51.46 + 212.51 462.28 = 726.25 t Say, 728.00 t
Abut cap Abut shaft pile cap

Horizontal longitudinal force at bearing level for pi(= 18.000 t
Moment = 18.000 x 8.799 = 158.4
Total = 158.38 Say, 159 t-m

Transverse moment due to eccentricity of live load class 70R wheeled;
= 193.39 Say, 194.0 t-m

Case-II(1) :H.F.L. case (Live load condition) :

Pile cap top level = 89.145 m
Pile cap bottom level = 87.345 m

Load from superstructure:

S.I.D.L. + D.L. = 36.24 + 184.71 = 220.95 t
Live load = 112.38 t
333.33 t Say, 334.00 t

Load from substructure & pile cap = 454.953 t Say, 454.95 t Total load = 788.95 t

Moment in Transverse direction;
Due to live load = 193.39 t-m
Say, 194.00 t-m

Moment in Longitudinal direction;
Due to live load = 159.00 t-m
Say, 159.00 t-m

Case-I(2) :DRY. case (Span dislodged condition) :

Vertical load on Pile foundation;
Load from superstructure

S.I.D.L. + D.L.	=	0.00	+	0.00	=	0.00	t			
Live load					=	0.00	t			
						0.00	t	Say,	0.00	t

Load from substructure & pile cap = 726.252 t Say, 727.00 t

Horizontal longitudinal force at bearing level for pi(= 0.00 t

Moment	=	0.00	x	8.799	=	0.000	t-m			
			Total		=	0.000	Say,	0	t-m	

Transverse moment = 0.00 Say, 0.0 t-m

Case-II(2) :H.F.L. case (Span dislodged condition) :

Vertical load on Pile foundation;
Load from superstructure

S.I.D.L. + D.L.	=	0.00	+	0.00	=	0.00	t			
Live load					=	0.00	t			
						0.00	t	Say,	0.00	t

Load from substructure & pile cap = 454.953 t Say, 455.00 t

Horizontal longitudinal force at bearing level for pi(= 0.00 t

Moment	=	0.00	x	8.799	=	0.000	t-m			
			Total		=	0.000	Say,	0	t-m	

Summary :

	P (t)	H (t)	M_L (t-m)	M_T (t-m)
1 DL	184.71	0.00	0.00	0.00
2 SIDL	36.24	0.00	0.00	0.00
3 LL 70RW +Class A	112.38	18.00	159.00	193.39
4 Span dislodged	0.00	0.00	0.00	0.00

Substructure weight

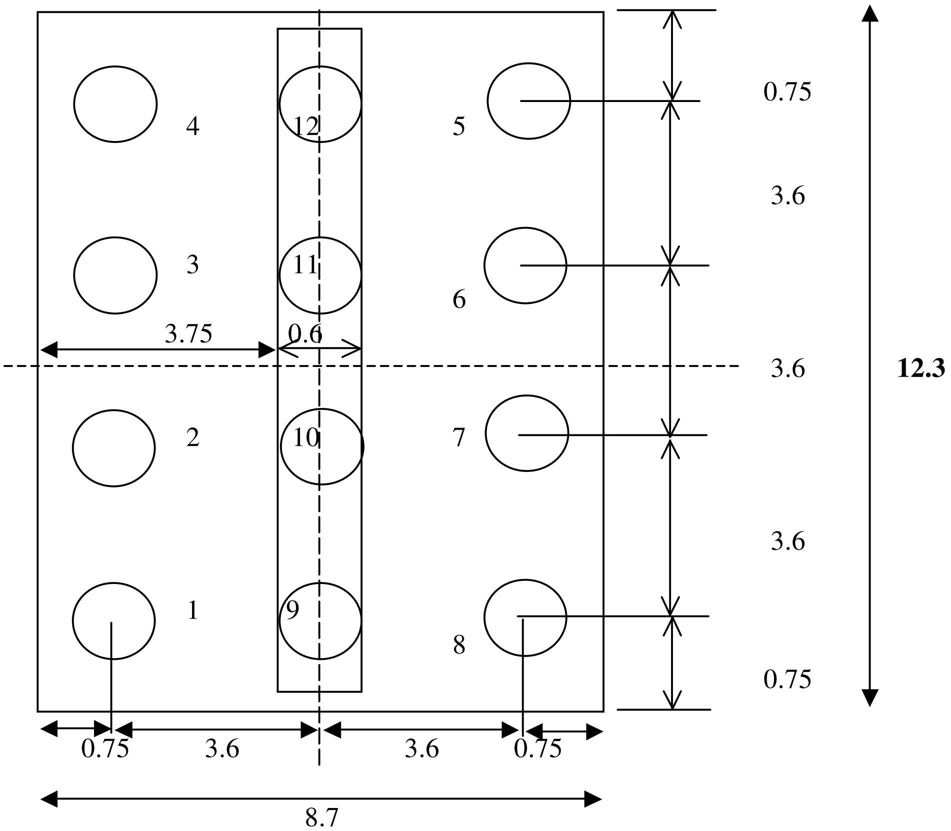
DRY					
1	Abutment cap +	51.46 t			
	Dirtwall				
2	Abutment shaft	212.51 t			
3	Pile cap	462.28 t			
		726.25 t			
H.F.L					
1	Abutment cap	51.46 t			
2	Abutment shaft	133.83 t			
3	Pile cap	269.67 t			
4	Earth on pile cap	92.61 t			
		454.9529 t			

Earthpressure				
1 Dry condition		442.33 t	2076.89 t-m	
2 H.F.L		347.31 t	1511.64 t-m	

Wt and Moments due to earth & Returnwall at Pile Cap bottom				
1 drycondition	760.094 t		1881.23 t-m	
Returnwall	46.078 t		114.04 t-m	
	806.172 t		- **1995.27** t-m	
2 H.F.L	558.472 t		1382.22 t-m	
Returnwall	36.857 t		91.22 t-m	
	595.330 t		- **1473.44** t-m	

Load Case	**Total Load**	**Longitudinal Moment**	**Transverse Moment**
	P	$\mathbf{M_L}$	$\mathbf{M_T}$
I DRY Condition			
1 With L.L	1865.75	240.62	193.39
2 Span dislodged	1532.42	81.62	0.00
II **H.F.L Condition**			
1 With LL	1383.61	197.20	193.39
2 Span dislodged	1050.28	38.20	0.00

No of Piles = 12



Pile No		x	Z_L	y	Z_T
1		3.6	28.8	5.4	36
2		3.6	28.8	1.8	108
3		3.6	28.8	1.8	108
4		3.6	28.8	5.4	36
5		3.6	28.8	5.4	36
6		3.6	28.8	1.8	108
7		3.6	28.8	1.8	108
8		3.6	28.8	5.4	36
9		0	0	5.4	36
10		0	0	1.8	108
11		0	0	1.8	108
12		0	0	5.4	36
		103.68		194.4	

Case I **DRY** with L.L

1

1 $$P_{max,min} = \frac{1865.75}{12} - \frac{240.62}{28.80} + \frac{193.39}{36.00} \qquad P = 152.50 \text{ t}$$

2 $$P_{max,min} = \frac{1865.75}{12} - \frac{240.62}{28.80} + \frac{193.39}{108.00} \qquad P = 148.92 \text{ t}$$

3 $$P_{max,min} = \frac{1865.75}{12} - \frac{240.62}{28.80} - \frac{193.39}{108.00} \qquad P = 145.33 \text{ t}$$

4 $$P_{max,min} = \frac{1865.75}{12} - \frac{240.62}{28.80} - \frac{193.39}{36.00} \qquad P = 141.75 \text{ t}$$

5 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{28.80} - \frac{193.39}{36.00} \qquad P = 158.46 \text{ t}$$

6 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{28.80} - \frac{193.39}{108.00} \qquad P = 162.04 \text{ t}$$

7 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{28.80} + \frac{193.39}{108.00} \qquad P = 165.62 \text{ t}$$

8 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{28.80} + \frac{193.39}{36.00} \qquad P = 169.21 \text{ t}$$

9 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{0.00} + \frac{193.39}{36.00} \qquad P = 160.85 \text{ t}$$

10 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{0.00} + \frac{193.39}{108.00} \qquad P = 157.27 \text{ t}$$

11 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{0.00} - \frac{193.39}{108.00} \qquad P = 153.69 \text{ t}$$

12 $$P_{max,min} = \frac{1865.75}{12} + \frac{240.62}{0.00} - \frac{193.39}{36.00} \qquad P = 150.11 \text{ t}$$

2 **spandislodged** DRY

1 $$P_{max,min} = \frac{1532.42}{12} - \frac{81.62}{28.80} + \frac{0.00}{36.00} \qquad P = 124.87 \text{ t}$$

2 $$P_{max,min} = \frac{1532.42}{12} - \frac{81.62}{28.80} + \frac{0.00}{108.00} \qquad P = 124.87 \text{ t}$$

3 $$P_{max,min} = \frac{1532.42}{12} - \frac{81.62}{28.80} - \frac{0.00}{108.00} \qquad P = 124.87 \text{ t}$$

4 $$P_{max,min} = \frac{1532.42}{12} - \frac{81.62}{28.80} - \frac{0.00}{36.00} \qquad P = 124.87 \text{ t}$$

5 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{28.80} - \frac{0.00}{36.00} \qquad P = 130.54 \text{ t}$$

6 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{28.80} - \frac{0.00}{108.00} \qquad P = 130.54 \text{ t}$$

7 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{28.80} + \frac{0.00}{108.00} \qquad P = 130.54 \text{ t}$$

8 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{28.80} + \frac{0.00}{36.00} \qquad P = 130.54 \text{ t}$$

9 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{0.00} + \frac{0.00}{36.00} \qquad P = 127.70 \text{ t}$$

10 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{0.00} + \frac{0.00}{108.00} \qquad P = 127.70 \text{ t}$$

11 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{0.00} - \frac{0.00}{108.00} \qquad P = 127.70 \text{ t}$$

12 $$P_{max,min} = \frac{1532.42}{12} + \frac{81.62}{0.00} - \frac{0.00}{36.00} \qquad P = 127.70 \text{ t}$$

H.F.L with L.L

II

1 1 $P_{max,min} = \frac{1383.61}{12} - \frac{197.20}{28.80} + \frac{193.39}{36.00}$ P = 113.83 t

2 $P_{max,min} = \frac{1383.61}{12} - \frac{197.20}{28.80} + \frac{193.39}{108.00}$ P = 110.24 t

3 $P_{max,min} = \frac{1383.61}{12} - \frac{197.20}{28.80} - \frac{193.39}{108.00}$ P = 106.66 t

4 $P_{max,min} = \frac{1383.61}{12} - \frac{197.20}{28.80} - \frac{193.39}{36.00}$ P = 103.08 t

5 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{28.80} - \frac{193.39}{36.00}$ P = 116.78 t

6 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{28.80} - \frac{193.39}{108.00}$ P = 120.36 t

7 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{28.80} + \frac{193.39}{108.00}$ P = 123.94 t

8 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{28.80} + \frac{193.39}{36.00}$ P = 127.52 t

9 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{0.00} + \frac{193.39}{36.00}$ P = 120.67 t

10 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{0.00} + \frac{193.39}{108.00}$ P = 117.09 t

11 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{0.00} - \frac{193.39}{108.00}$ P = 113.51 t

12 $P_{max,min} = \frac{1383.61}{12} + \frac{197.20}{0.00} - \frac{193.39}{36.00}$ P = 109.93 t

2 spandislodged condition H.F.L

1 $P_{max,min} = \frac{1050.28}{12} - \frac{38.20}{28.80} + \frac{0.00}{36.00}$ P = 86.20 t

2 $P_{max,min} = \frac{1050.28}{12} - \frac{38.20}{28.80} + \frac{0.00}{108.00}$ P = 86.20 t

3 $P_{max,min} = \frac{1050.28}{12} - \frac{38.20}{28.80} - \frac{0.00}{108.00}$ P = 86.20 t

4 $P_{max,min} = \frac{1050.28}{12} - \frac{38.20}{28.80} - \frac{0.00}{36.00}$ P = 86.20 t

5 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{28.80} - \frac{0.00}{36.00}$ P = 88.85 t

6 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{28.80} - \frac{0.00}{108.00}$ P = 88.85 t

7 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{28.80} + \frac{0.00}{108.00}$ P = 88.85 t

8 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{28.80} + \frac{0.00}{36.00}$ P = 88.85 t

9 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{0.00} + \frac{0.00}{36.00}$ P = 87.52 t

10 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{0.00} + \frac{0.00}{108.00}$ P = 87.52 t

11 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{0.00} - \frac{0.00}{108.00}$ P = 87.52 t

12 $P_{max,min} = \frac{1050.28}{12} + \frac{38.20}{0.00} - \frac{0.00}{36.00}$ P = 87.52 t

DRY	**P_{max}**		=	**169.21**	t
	P_{min}		=	**124.87**	t
H.F.L	**P_{max}**		=	**127.52**	t
	P_{min}		=	**86.20**	t

Horizantal load per pile

Dry Condition

Total horizantal force

with L.L = 18.00 + 442.33 = 460.33 t

= 38.36104 t per pile

span dislodged condition = 0.00 + 442.33 = 442.33 t

= 36.86104 t per pile

H.F.L Condition

Total horizantal force

with L.L = 18.00 + 347.31 = 365.31 t

= 30.44263 t per pile

span dislodged condition = 0.00 + 347.31 = 347.31 t

= 28.94263 t per pile

Determination of Lteral Deflection of pile :

$T = \sqrt[5]{\dfrac{E \times I}{K_1}}$ E = Youngs modulus kg/m^2

K_1 = Constant kg/cm^3

$R = \sqrt[4]{\dfrac{E \times I}{K_2}}$ K_2 kg/cm^2

I = moment of inertia of pile cross section cm^4

Dia of pile = 1.20

$E = 5000 \times \sqrt{35} = 301533\ kg/cm^2$

610.73

$I = \dfrac{3.14}{64} \times 120 = 10178760\ cm^4$

$K_1 = 0.775$

$K_2 = 0$

$T = 330.78$

$R = 0$

L_e = Embeded Length of pile

= 87.35 - 73.845 = 1350 cm

L_1 = Exposed Length = 87.35 - 87.35 = 0 cm

$\dfrac{L_1}{T} = 0$

$\dfrac{L_f}{T} = 2.19$

L_f = 724.42 cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12 \; E \; I} = \frac{38.361 \times 724.42}{12 \times 301533 \times 10178760}$$

$$= 0.396 \text{ cm}$$

$$= 3.96 \text{ mm}$$

Maximum Moment

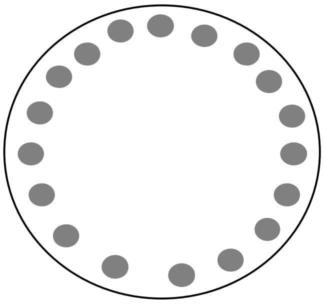
$$M_F = Q \times \frac{\left(L_1 + L_f\right)}{2} = 138.95 \text{ t-m}$$

$$M = m \times M_F$$

$$= 0.83 \times 138.95 = 115.33 \text{ t-m}$$

Reinforcement in pile:

Provide 25 mm dia 28 nos 137.44 cm^2



Provide 8 mm Rings at 180 mm c/c

DESIGN OF PILE SECTION		
INPUT DATA	NORMAL CASE	
	MAX. LOAD	MIN. LOAD
D=DIA OF PILE (in m)	1.200	1.200
C=COVER FROM CENTRE OF BAR (in m)	0.075	0.075
M=MODULAR RATIO (as per IRC:6)	10.000	10.000
BD=BETA ANGLE FOR DEFINING N.A. (in degree)	98.97	107.93
AS=AREA OF STEEL (in sq. Cm)	137.445	137.445
P=VERTICAL LOAD (in tonnes)	169.21	86.20
BM=BENDING MOMENT (in t-m)	115.33	115.33
SOLUTION		
RO=OUTER RADIUS=D/2	0.600	0.600
RI=RADIUS OF REINFORCEMENT RING=D/2-C	0.525	0.525
T= THICK.REINFORCEMENT RING = AS/2*3.142*RI	0.004	0.004
B=BD*PI/180	1.727	1.884
B1=COS(B)	-0.156	-0.308
A=ALPHA ANGLE=ACOS (RO*B1/RI)	1.750	1.930
RECOSB=RICOSA		
B2=SIN(B)^3	0.964	0.861
B3=SIN(4*B)	0.586	0.950
B4=SIN(2*B)	-0.308	-0.586
A=COS(A)	-0.178	-0.352
A2=SIN(2*A)	-0.351	-0.659
A3=SIN(A)	0.984	0.936
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.145	0.099
NUM2=2*(RO^3)/(1+B1)	0.512	0.624
NUM3=(RI^3) *T/(RO+RI*A1)	0.001	0.001
NUM4=(M-1) *PI+A-A2/2	30.20	30.53
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.110	0.106
DENM1=2*(RO^2)/(1+B1)	0.853	1.040
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	0.223	0.139
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.005	0.006
DENM4=(M-1) *PI*A1-A3+A*A1	-6.33	-11.56
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.162	0.080
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.682**	**1.320**
ACTUAL ECCENTRICITY EC = M/P	**0.682**	**1.338**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	-0.094	-0.185
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.506	0.415
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**1048**	**1075**
TS=TENS.STRESS IN STEEL (T/SQM)	**12793**	**18376**
PERMISSIBLE COMP.STRESS IN Concrete (in t/sq.m)	1190	1190
PERMISSIBLE TENSILE STRESS IN Steel (in t/sq.m)	20400	20400

Design of Pile cap :

I

L.W.L with L.L

Load on individual Pile

PILE 1	PILE 2	PILE 3	PILE 4	PL1 + PL2+PL3+PL4
152.50	148.92	145.33	141.75	588.50
PILE 5	**PILE 6**	**PILE 7**	**PILE 8**	**PL5+PL6+PL7+PL8**
158.46	162.04	165.62	169.21	655.34

Therefore,

Critical load for bending in transverse direction = 655.34 t

L.W.L **spandislodged**

Load on individual Pile

PILE 1	PILE 2	PILE 3	PILE 4	PL1 + PL2+PL3+PL4
124.87	124.87	124.87	124.87	499.47
PILE 5	**PILE 6**	**PILE 7**	**PILE 8**	**PL5+PL6+PL7+PL8**
130.54	130.54	130.54	130.54	522.14

Therefore,

Critical load for bending in transverse direction = 522.14 t

II

H.F.L with L.L

Load on individual Pile

PILE 1	PILE 2	PILE 3	PILE 4	PL1 + PL2+PL3+PL4
113.83	110.24	106.66	103.08	433.81
PILE 5	**PILE 6**	**PILE 7**	**PILE 8**	**PL5+PL6+PL7+PL8**
116.78	120.36	123.94	127.52	488.59

Therefore,

Critical load for bending in transverse direction = 488.59 t

H.F.L spandislodged condition

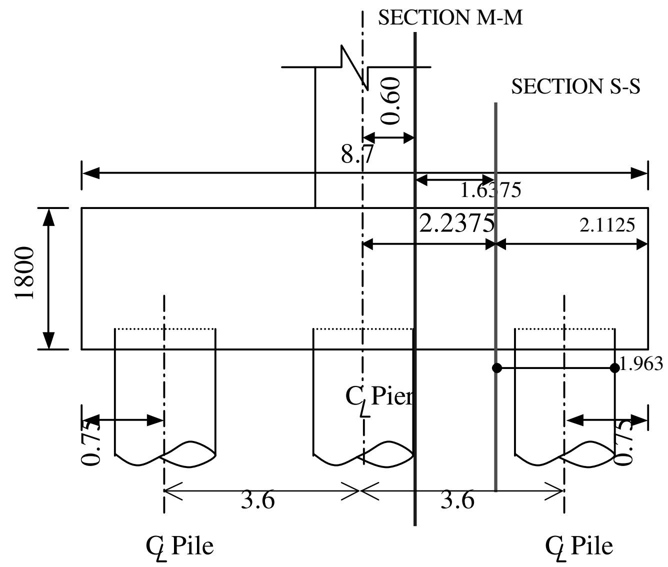
Load on individual Pile

PILE 1	PILE 2	PILE 3	PILE 4	PL1 + PL2+PL3+PL4
86.20	86.20	86.20	86.20	344.79
PILE 5	**PILE 6**	**PILE 7**	**PILE 8**	**PL5+PL6+PL7+PL8**
88.85	88.85	88.85	88.85	355.40

Therefore,

Critical load for bending in transverse direction = 355.40 t

Design of Pile cap in Longitudinal direction :



Design for bending;

Maximum load on piles in a row = 655.34 t Say, 656.00 t

The critical section for bending moment is taken at the faceof Abutment whereas for S.F.it istak at a distance of effective depth from the face of the abutment.

The critical sections of bending moment & shear forces are shown in the figure above.

Lever arm from centre of pile to the face of abut 3.60 - 0.600

= 3.000 m

Bending moment due to pile loads at the face of pier

= 656.00 x 3.000

= 1968.00 t-m

Self weight of pile cap upto critical section M-M

= 0.75 + 3.000 x 12.3 x 1.80 x 2.40 = 199.26 t

Bending moment due to self weight of pile cap

$= 199.26 \times \frac{3.750}{2}$

= 373.61 t-m

Therefore,

Net Bending moment = 1968.00 - 373.61

= 1594.4 t-m Say, 1595.00 t-m

For, M - 35 & Fe - 415

σcbc = 11.67 N/mm^2

m = 10

σst = 200 N/mm^2

K = 0.368

j = 0.877

R = 1.886

Depth of pile cap = 1800 mm

Piles shall be embedded by 50 mm into pile cap & reinforcement shall be placed by giving a 75 mm cover above piles.

$d_{eff.}$ (available) $= 1800 - 50 - 75 - \frac{75}{2} = 1637.5$ mm

$d_{eff.}$ (required) $= \frac{1595.00 \times 10^{7}}{1.886 \times 8700} = 985.93$ mm < 1637.5 mm **Safe**

Area of steel required $= \frac{1595.00 \times 10^{7}}{200 \times 0.877 \times 1637.5}$

= 55522.0 mm^2

= 4514.0 mm^2/m width

Minimum area of steel required $= \frac{0.12 \times 1800 \times 1000}{100}$ (As per cl. 305.19 of IRC :21-2000)

= 2160.0 mm^2/m width

Provide 32 O, 160 mm c/c in bottom face. 5026.5 mm^2 > 4514.0 mm^2

Minimum area of steel required on top face $= \frac{0.06 \times 1800 \times 1000}{100}$

= 1080.0 mm^2

Provide 16 O, 160 mm c/c in top face. 1256.6 mm^2 > 1080.0 mm^2

Check for shear:

Distance of the section S-S from face of Abutment (i.e. at effective depth)	=	1.6375 m
Protion of Pile cap effective for shear	=	2.1125 m
Protion of Pile effective for shear	=	1.2000 m
Overall depth of Pile cap at section S-S	=	1.800 m
Effective depth at section S-S	=	1.6375 m
Shear force due to Pile loads	=	656.000 t
Bending moment due to Pile loads	=	393.600 t-m
Weight of Pile cap upto section S-S	=	112.250 t
Lever arm for bending moment due to weight of Pile cap	=	1.056 m
Bending moment due to weight of pile cap	=	118.564 t-m
Net shear force 'V' at section S-S	=	543.750 t
Net bending moment at section S-S	=	275.036 t-m

Therefore, Net upward shear

= 543.750 t

Shear stress $= \dfrac{543.750}{1.6375 \times 12.30} = 27 \; t/m^2$

Percentage of reinforcement provided

$= \dfrac{50.265}{100 \times 1.6375} = 0.31 \; \%$

Permissible shear stress = 25.50 t/m^2

Shear reinforcement required

Minimum Shear Reinforcement :

$$\frac{A_{sw}}{b \times s} = \frac{0.4}{0.87 \times f_y}$$

$$\frac{A_{sw}}{s} = \frac{0.4 \times b}{0.87 \times f_y} = 13.627 \; mm^2/m$$

Provide 2 Legged 10 @ 175 = 27.489

<u>Design of Pile cap in Transverse direction :</u>

Minimum area of steel required $= \dfrac{0.12}{100} \times 1800 \times 1000$

$= 2160.0 \; mm^2/m$ width

Provide	20 ϕ	130	mm c/c in bottom face.	2416.6 mm^2	>	2160.0 mm^2
Provide	16 ϕ	130	mm c/c in top face.	1546.6 mm^2	>	1080.0 mm^2

Design of Abutment shaft :

Calculation of Forces at Abutment base

(At Top of Pile cap)

Checking the section at R.L. = **89.145** m

Lever arm for Longitudinal forces = 6.999 i.e(96.144 - 89.145)

DRY ***With L.L***

1 Vertical load -

S.I.D.L. = 36.24 t
Dead load = 184.71 t
Live load = 112.38 t
Abutment cap = 51.46 t
Abutment shaft = 212.509 t
Total load = 597.30 t Say, 597.30 t

2 Longitudinal moment -

Due to Longitudinal force = 18.000 x 6.999 = 125.98 t-m
Due to Earth = 1256.00 t-m
= 1381.99 t-m 1382.00 t-m

3 Transverse moment -

Due to eccentricity of live load
= 193.39 t-m Say, 194.00 t-m

VERTICAL LOAD	LONG. MOMENT	TRANS. MOMENT
597.30	1382.00	194.00

H.F.L ***With L.L***

1 Vertical load -

S.I.D.L. = 36.24 t
Dead load = 184.71 t
Live load = 112.38 t
Abutment cap = 51.46 t
Abutment shaft = 133.828 t
Total load = 518.62 t Say, 518.62 t

2 Longitudinal moment -

Due to earth = 966.507 t-m
Due to Longitudinal force = 125.98 t-m
Total moment = 1092.49 t-m Say, 1093.00 t-m

3 Transverse moment -

Due to Live load = 193.39 t-m
Total moment = 193.39 t-m Say, 194.00 t-m

VERTICAL LOAD	LONG. MOMENT	RANS. MOMEN
518.62	1093.00	194.00

DRY ***spandislodgedcondition***

1 Vertical load -

S.I.D.L. = 0.00 t
Dead load = 0.00 t
Abutment cap = 51.46 t
Abutment shaft = 212.51 t
Total load = 263.97 t Say, 263.97 t

2 Longitudinal moment -

Due to Longitudinal force = 0.000 x 6.999 = 0 t-m
Due to earth = 1256.00 t-m
= 1256.00 t-m 1257.00 t-m

3 Transverse moment -
Due to eccentricity of live load
= 0.00 t-m Say, 0.00 t-m

VERTICAL LOAD	LONG. MOMENT	TRANS. MOMENT
263.97	1257.00	0.00

H.F.L ***spandislodged condition***

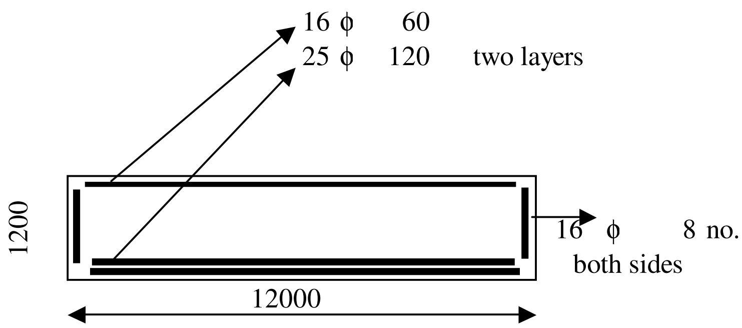
1 Vertical load -

S.I.D.L. = 0.00 t
Dead load = 0.00 t
Abutment cap = 51.46 t
Abutment shaft = 133.828 t
Total load = 185.29 t Say, 185.29 t

2 Longitudinal moment -

Due to Longitudinal force = 0.000 x 6.999 = 0 t-m
Due to earth = 966.51 t-m
= 966.51 t-m 967.00 t-m

VERTICAL LOAD	LONG. MOMENT	TRANS. MOMENT
185.29	967.00	0.00



PLAN : ABUTMENT SHAFT

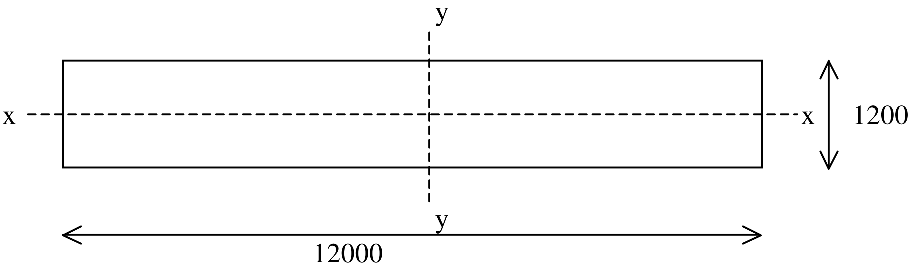
Reinforcement provided = 74185.57 mm^2

Summary of Loads at Abutment Shaft bottom :

1	**DRY condition**	**P**		**M_L**		**M_T**	
	With L.L	597.30		1382.00		194.00	
	Span dislodged	263.97		1257.00		0.00	
2	**H.F.L**						
	With L.L	518.62		1093.00		194.00	
	Span dislodged	185.29		967.00		0.00	

Check for Cracked/Uncracked Section

Length of section = 12000 mm
Width of section = 1200 mm
Gross Area of section A_g = 14400000 mm^2
Gross M.O.I of section I_{gxx} = 1.73E+12 mm^4
Gross M.O.I of section I_{gyy} = 1.73E+14 mm^4

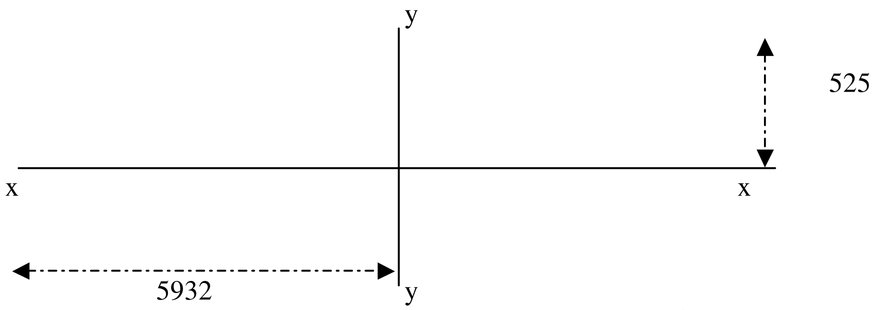


Abutment section

Transformed sectional properties of section:

Adopting

Modular ratio	m	=	10	
Cover			72.5	68
Dia of Bars		=	25	16
No of bars in tension face (longer)		=	120	
No of bars in compression face		=		60
No of bars in shorter direction		=		8
Total bars in section		=	196	
Steel Area	A_s	=	74186 mm^2	
% of Steel		=	0.5152 %	



A_{sx} = 58905 mm^2

A_{sy} = 1608.5 mm^2

Area of concrete $A_c = A_g - A_s$ = 14325814 mm^2

C.G of Steel placed on longer face = 527.5 mm

C.G of Steel placed on shorter face = 5932 mm

Transformed Area of Section A_{tfm} = 15067670 mm^2

Transformed $M.I_{txx} = I_{gxx} + 2\,(m - 1)\,A_s\,ax^2$

= 2.02303E+12 mm^4

$$Z_{xx} = \frac{M.Itxx}{d/2} = 3.37E+09\ mm^3$$

Transformed $M.I_{tyy} = I_{gyy} + 2\,(m - 1)\,A_s\,ay^2$

= 1.73819E+14 mm^4

$$Z_{yy} = \frac{M.I_{tyy}}{d/2} = 2.9E+10\ mm^3$$

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.73E+12 mm^4

Area of section = 14400000 mm^2

r = 346.4102 mm

Effective length of Abutment shaft (IRC:21-2000 cl: 306.2.1)

Abutment shaft height	L	=	6.149	m
Effective length	L_{eff}	=	10.761	m
Slenderness ratio		=	31.064	< **50**
Type of member		=		Shotr Column

Stress reduction coefficient (IRC:21-2000 cl: 306.4.2,3)

	β	=	1	
Permissible stresses **:**	*concrete*			
	σ_{cbc}	=	10	N/mm^2
	σ_{co}	=	7.5	N/mm^2
	Tensile stress	=	0.67	N/mm^2
Permissible stresses :	*Steel*			
	σ_{st}	=	200	N/mm^2

S.No Item	DRY Case	
Loads and Moments	L.L	Span dislodg
1 P	597.30 t	263.97 t
2 M_L	1382.00 t-m	1257.00 t-m
3 M_T	-194.00 t-m	0.00 t-m
Actual(calculated) Stresses		
4 $\sigma_{co,cal}$ P/A_{tfm}	0.396409711	0.175189
5 $\sigma_{cbc,cal}$ M_L/Z_{xx}	4.09879899	3.728068
6 $\sigma_{cbc,cal}$ M_T/Z_{yy}	-0.06696628	0
7 $\sigma_{cbc,cal}$ = 5 + 6	4.031832706	3.728068
Permissible Stresses		
8 σ_{cbc}	10	10
9 σ_{co}	7.5	7.5

Check for Minimum steel are mm^2		
10 Conc.Area Required for directstress (1)/(9)	796396.1	351959
11 0.8% of area required	6371.169	2815.7
12 0.3% of A_g	43200	43200
13 Governing steel mm^2	43200	43200
14 Provided Steel area mm^2	74185.57	74186
Check for safety of section		
$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.456038 < 1 O.K	0.3962 < 1 O.K

Check for Cracked /Uncracked section		
$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-3.635423	-3.553
Permissible Basic tensile stress in concrete	-0.67	-0.67
Section to be designed as	**Cracked**	**Cracked**

S.No Item	DRY Case	
Loads and Moments	L.L	Span dislodg
1 P	518.62 t	185.29 t
2 M_L	1093.00 t-m	967.00 t-m
3 M_T	-194.00 t-m	0.00 t-m
Actual(calculated) Stresses		
4 $\sigma_{co,cal}$ P/A_{tfm}	0.344190888	0.12297
5 $\sigma_{cbc,cal}$ M_L/Z_{xx}	3.241669534	2.867973
6 $\sigma_{cbc,cal}$ M_T/Z_{yy}	-0.06696628	0
7 $\sigma_{cbc,cal}$ = 5 + 6	3.17470325	2.867973
Permissible Stresses		
8 σ_{cbc}	10	10
9 σ_{co}	7.5	7.5
Check for Minimum steel are mm^2		
10 Conc.Area Required for directstress (1)/(9)	691487.3	247050
11 0.8% of area required	5531.898	1976.4
12 0.3% of A_g	43200	43200
13 Governing steel mm^2	43200	43200
14 Provided Steel area mm^2	74185.57	74186
Check for safety of section		
$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.363362 < 1 O.K	0.3032 < 1 O.K
Check for Cracked /Uncracked section		
$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-2.830512	-2.745
Permissible Basic tensile stress in concrete	-0.67	-0.67
Section to be designed as	**Cracked**	**Cracked**

Note:
The design for cracked section has been carried out as per computer programme ,
which is based on "Behaviour of columns and walls"in the book entitled "Reinforced Concr
Structural Elements" by P.Purushothaman.

EARTH PRESSURE CALCULATION

Formation level	=	98.3 m
Pilecap bottom level	=	87.345 m
Low Water Level	=	90.145 m
Highest flood level	=	94.609 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb's theory of active earth pressure

$$Ka = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha.\sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta).\sin(\phi-i)}{\sin(\alpha-\delta).\sin(\alpha+i)}}\right]^2}$$

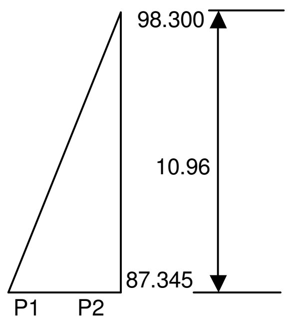
Here				
	Angle of internal friction,	ϕ	=	30 °
	Angle of friction between soil and concrete,	δ	=	20 °
	Surcharge angle	i	=	0 °
	Angle of wall face with horizontal	α	=	90 °
	Bulk density of earth	γ	=	1.8 t/m^3
	Submerged density of earth	γ_{sub}	=	1.0 t/m^3
	Width of abutment		=	12 m

$$Ka^* = \frac{\sin^2 2.09}{\sin^2 1.57 \times \sin 1.22 \times \left[\sqrt{1 + \dfrac{\sin 0.87 \times \sin 0.52}{\sin 1.22 \times \sin 1.57}}\right]^2}$$

* (value of angles in radian)

$$= \frac{0.750}{2.523} = \mathbf{0.2973}$$

CALCULATION OF EARTH PRESSURE IN DRY CONDITION



P1 = 0.2973 x 1.8 x 10.96 = 5.86 t/m^2

P2 = 0.2973 x 1.0 x 0.00 = 0 t/m^2

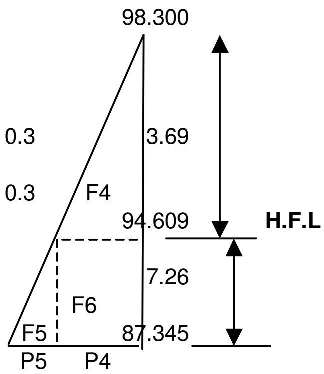
F1 = 0.5 x 5.863 x 10.96 x 12 x cos 0.3

= 362.12 t say 363 t

M = 362.12 x 0.42 x 10.96

= 1666 t-m

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION



P4 = 0.2973 x 1.8 x 3.69 = 1.98 t/m^2

P5 = 0.2973 x 1.0 x 7.26 = 2.16 t/m^2

F4 = 0.5 x 1.975 x 3.69 x 12 x cos 0.3

= 41.107 t

F5 = 0.5 x 2.16 x 7.26 x 12 x cos 0.3

= 88.45 t

F6 = 1.98 x 7.26 x 12 x cos 0.3

= 161.80 t

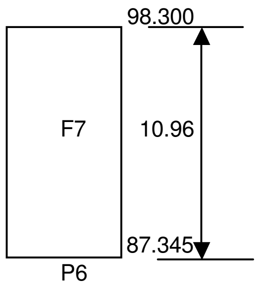
Total force, F = 41.107 + 88.45 + 161.80

= 291 t

M = 41.107 x (0.42 x 3.69 + 7.26)

$+ 88.45 \times 0.33 \times 7.26 + 161.8 \times \frac{7.26}{2}$

= 1162 t-m



CALCULATION OF LIVE LOAD SURCHARGE

Dry

P6	=	0.2973	x	1.2	x	1.8	=	0.64 t/m^2			
F7	=	0.64	x	10.96	x	12	x	cos 0.3			
	=	79 t									
M	=	79	x	$\frac{10.36}{2}$	=	411 t-m					

H.F.L

P6	=	0.2973	x	1.2	x	1.0	=	0.36 t/m^2			
F7	=	0.6422	x	3.69	x	12	x	cos 0.3	=	26.72901	t
	=	0.3568	x	7.26	x	12	x	cos 0.3	=	29.22417	t
									=	55.95318	
M	=	26.729	x	9.11	+	29.22	x	3.6	=	349.63008	tm

EARTH PRESSURE CALCULATION

Formation level	=	98.3	m
Pilecap top level	=	89.145	m
Low Water Level	=	90.145	m
Highest flood level	=	94.609	m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb's theory of active earth pressure

$$Ka = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha.\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-i)}{\sin(\alpha-\delta).\sin(\alpha+i)}}\right]^2}$$

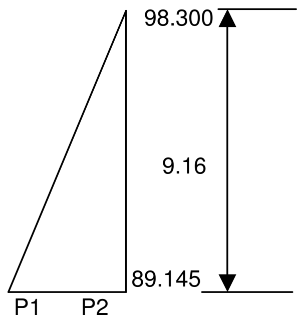
Here				
	Angle of internal friction,	ϕ	=	30 ^{o}
	Angle of friction between soil and concrete,	δ	=	20 ^{o}
	Surcharge angle	i	=	0 ^{o}
	Angle of wall face with horizontal	α	=	90 ^{o}
	Bulk density of earth	γ	=	1.8 t/m^3
	Submerged density of earth	γ_{sub}	=	1.0 t/m^3
	Width of abutment		=	12 m

$$Ka^* = \frac{\sin^2 2.09}{\sin^2 1.57 \times \sin 1.22 \times \left(\sqrt{1 + \frac{\sin 0.87 \times \sin 0.52}{\sin 1.22 \times \sin 1.57}}\right)^2}$$

* (value of angles in radian)

$$= \frac{0.750}{2.523} = \mathbf{0.2973}$$

CALCULATION OF EARTH PRESSURE IN DRY CONDITION



P1 = 0.2973 x 1.8 x 9.16 = 4.9 t/m^2

P2 = 0.2973 x 1.0 x 0.00 = 0 t/m^2

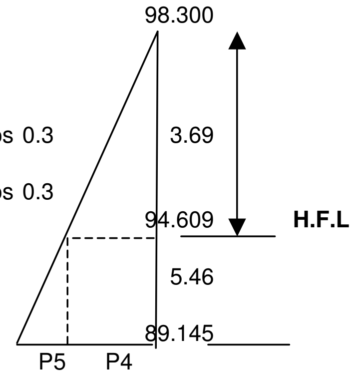
F1 = 0.5 x 4.899 x 9.16 x 12 x cos 0.3

= 252.9 t say 253 t

M = 252.9 x 0.42 x 9.16

= 972 t-m

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION



P4 = 0.2973 x 1.8 x 3.69 = 1.98 t/m^2

P5 = 0.2973 x 1.0 x 5.46 = 1.62 t/m^2

F4 = 0.5 x 1.975 x 3.69 x 12 x cos 0.3

= 41.107 t

F5 = 0.5 x 1.625 x 5.46 x 12 x cos 0.3

= 50.05 t

F6 = 1.98 x 5.46 x 12 x cos 0.3

= 121.71 t

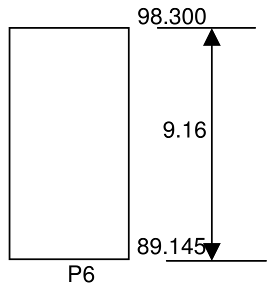
Total force, F = 41.107 + 50.05 + 121.71

= 213 t

M = 41.107 x (0.42 x 3.69 + 5.46)

+ 50.05 x 0.33 x 5.46 + 121.7 x $\frac{5.46}{2}$

= 711 t-m



CALCULATION OF LIVE LOAD SURCHARGE

Dry

P6 = 0.2973 x 1.2 x 1.8 = 0.64 t/m^2

F7 = 0.64 x 9.16 x 12 x cos 0.3

= 66 t

M = 66 x $\frac{8.56}{2}$ = 284 t-m

H.F.L

P6 = 0.2973 x 1.2 x 1.0 = 0.36 t/m^2

F7 = 0.6422 x 3.69 x 12 x cos 0.3 = 26.72901 t

= 0.3568 x 5.46 x 12 x cos 0.3 = 21.9825 t

= 48.7115

M = 26.729 x 7.31 + 21.98 x 2.7 = 255.43187 tm

Design of Abutment Cap:

As the cap is fully supported on the abutment.Minimum thickness of the cap required as per cl: 710.8.2 of IRC:78-2000 is 200 mm.

However the thickness of abutment cap is	=	500	mm			
Assuming a cap thickness of	=	500	mm			
Volume of Abutment cap	=	0.5	x	1.20	x	12
	=	7.2	m^3			
Quantity of steel	=	1 %	of volume			
	=	$\frac{1}{100}$	x	7.2		
	=	0.072	m^3			
Quantity of steel to be provided at top	=	0.036	m^3			
Quantity of steel to be provided at bottom	=	0.036	m^3			

Top & bottom face:

Quantity of steel to be provided in Longitudinal di = 0.018 m^3
Assuming a clear cover of = 50 mm
Length of bar 12.00 - 0.100 = 11.9 m
Area of steel required in Longitudinal direction

$\frac{0.018}{11.9}$ = 1512.61 mm^2

Provide **10** nos of bars **16** mm dia at top & bottom face.
= 2010.62 mm^2

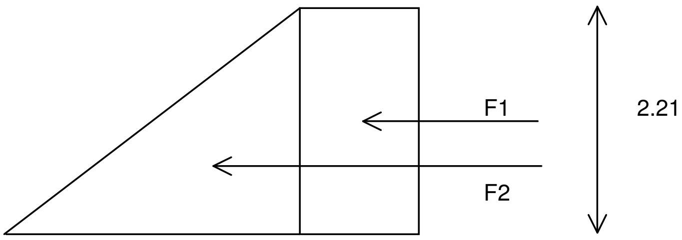
Transverse steel:

Quantity of steel to be provided in Longitudinal di = 0.018 m^3
Assuming a clear cover of = 50 mm
Assuming a dia of bar = 16 mm
Length of bar 1.20 - 0.100 = 1.1 m

Volume of each stirrup = 0.00022 m^3

no of stirrups required for m/length = 7 nos
Required Spacing = $\frac{1000}{7}$
= 142.857 mm
Provide 16 mm dia bar 140 mm c/c stirrups throught in length of abutment cap.
1436.16 mm^2

Design of Dirt wall:
Dirt wall designed as a vertical cantilever.



Intensity for rectangular portion	=	0.2794	x	2.00	x	1.2	=	0.67056	t/m^2
F1	=	0.67056	x	12.00	x	2.21	=	17.75106	t
Intensity for triangular portion	=	0.2794	x	2.00	x	2.21	=	1.232713	t/m^2
F2	=	1.232713	x	12.00	x	2.21	=	16.31619	t
M1	=	17.75106	x	1.10			=	19.57942	t-m
M2	=	16.31619	x	0.92652			=	15.11727	t-m
M1+ M2							=	**34.6967**	t-m
Total moment at base of dirt wall /m length							=	2.891391	t-m/m
Thickness of dirtwall							=	0.3	m
Assuming a clear cover on either face							=	50	mm

Vertical steel on earth face:

dia of steel bar							=	12	mm
Available effective depth	=	300	-	50	-	6	=	244	mm
effective depth req	=	2.891391					=	138.3773	mm
		1.51	x	1000					
Ast req	=	2.891391					=	666.4772	mm^2/m
		200	x	0.889	x	244			
Minimum steel	=	0.12	x	300	x	1000	=	360	mm^2/m
		100							

Provide 12 mm dia bar 200 mm c/c as vertical steel at earth face.
565.4867 mm^2/m

Disrtibution steel on earth face:

dia of steel bar							=	12	mm
Available effective depth	=	300	-	50	-	12	=	238	mm
0.3M	=			0.3	x	2.891391	=	0.867417	t-m/m
Ast req	=	0.867417					=	204.9837	mm^2/m
		200	x	0.889	x	238			
Minimum steel as per IRC:21-200 cl:305.10							=	250	mm^2/m
Governing steel at earth face							=	250	mm^2/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.
392.6991 mm^2/m

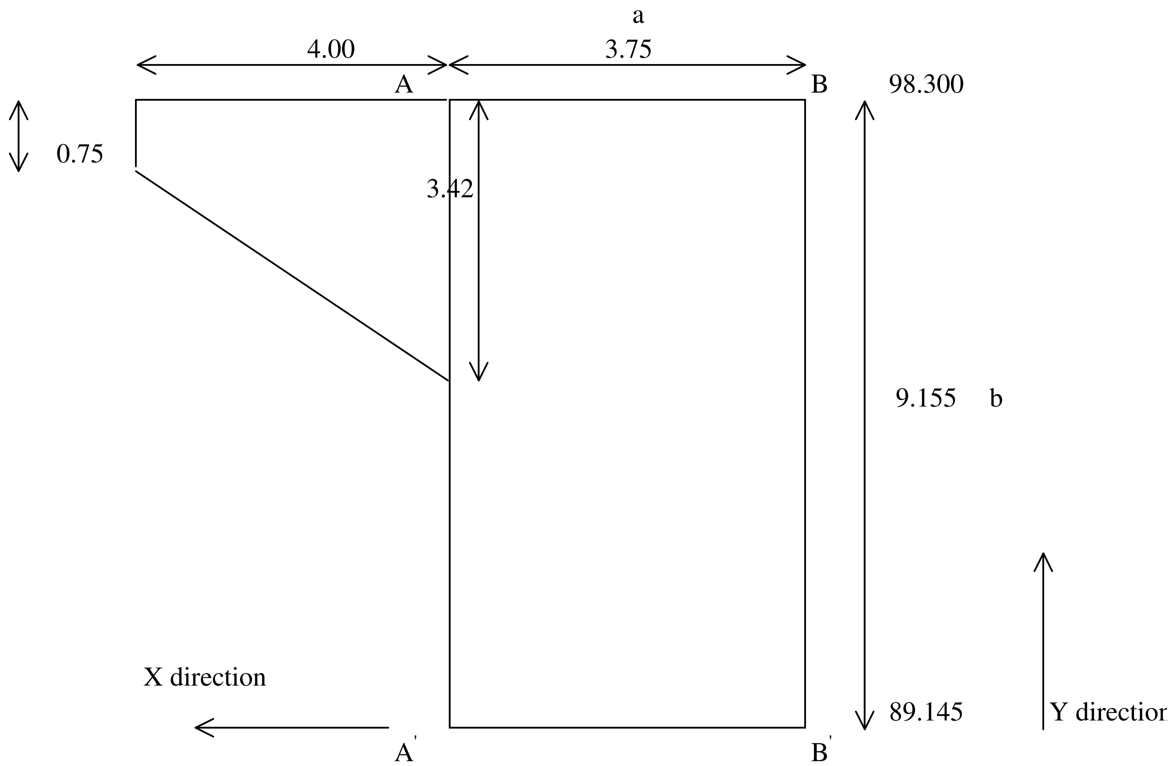
Vertical steel on earth face

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm^2/m
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.
392.6991 mm^2/m

Distribution steel:

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm^2/m
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.
392.6991 mm^2/m

DESIGN OF RETURNWALL



Width of Solid return wall (a)		=	3.75	
Width of Cantilever return wall		=	4.00	
Avg Height of Solid return wall (b)		=	9.155	
Height of Cantilever return at Tip		=	0.75	
Height of Cantilever return taper		=	2.67	
Height of Cantilever return at Root		=	3.42	
Thickness of Solid Return at farther end		=	0.5	
Thickness of Solid Return at Root		=	0.5	
Thickness of Solid Return at bottom		=	0.5	
Thickness of Solid Return at top		=	0.5	
Thickness of Cantilever return		=	0.5	
Unit wt of Soil		=	1.8	t/m^3
Grade of concrete		=	M 30	
	σ_{cbc}	=	1020	t/m^2
	m	=	10	
	σ_{st}	=	20400	t/m^2
	k	=	0.333	
	j	=	0.889	
	R	=	151.1	t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.4096122	For a/b	=	0.375	β_1	=	0.353	β_2	=	0.398
			For a/b	=	0.5	β_1	=	0.631	β_2	=	0.632

a/b = 0.4096122 β_1 = 0.429978

β_2 = 0.462794

Live Load Surcharge:

$q = 0.2794 \times 1.8 \times 1.2 = 0.603504$ t/m^2

$$\sigma_{bmax} = \frac{\beta_1 \times q \times b^2}{t^2}$$

$$\sigma_{amax} = \beta_2 \frac{\times q \times b^2}{t^2}$$

$$\sigma_{bmax} = \frac{0.4299776 \times 0.603504 \times 83.81}{0.25} = 86.99668 \text{ t/m}^2$$

For 1000 mm of width

$$Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 = 0.041667 \text{ m}^3$$

Hence Moment /m width along Y direction

M_Y /m width $= 86.99668 \times 0.041667 = 3.624862$ t-m/m

$$\sigma_{amax} = \frac{0.4627941 \times 0.603504 \times 83.81}{0.25} = 93.63639 \text{ t/m}^2$$

For 1000 mm of width

$$Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 = 0.041667 \text{ m}^3$$

Hence Moment /m width along X direction

M_X /m width $= 93.636389 \times 0.041667 = 3.901516$ t-m/m

Case (2) For Triangular loading due to earth pressure

a/b = 0.4096122 For a/b = 0.375 β_1 = 0.212 β_2 = 0.148
For a/b = 0.5 β_1 = 0.328 β_2 = 0.200

a/b = 0.4096122 β_1 = 0.24412
β_2 = 0.162399

Earth pressure:

$q = 0.2794 \times 1.8 \times 9.155 = 4.604233$ t/m^2

$$\sigma_{bmax} = \frac{\beta_1 \times q \times b^2}{t^2}$$

$$\sigma_{amax} = \beta_2 \frac{\times q \times b^2}{t^2}$$

$$\sigma_{bmax} = \frac{0.2441202 \times 4.604233 \times 83.81}{0.25} = 376.8232 \text{ t/m}^2$$

For 1000 mm of width

$$Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 = 0.041667 \text{ m}^3$$

Hence Moment /m width along Y direction

M_Y /m width = 376.82315 x 0.041667 = **15.70096** t-m/m

$$\sigma_{amax} = \frac{0.1623987 \times 4.604233 \times 83.81}{0.25} = 250.6781 \text{ t/m}^2$$

For 1000 mm of width

$$Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 = 0.041667 \text{ m}^3$$

Hence Moment /m width along X direction

M_X /m width = 250.67814 x 0.041667 = **10.44492** t-m/m

Total Moment in Solid Return /m height = **14.346** t-m/m Along X-direction

Total Moment in Solid Return /m width = **19.326** t-m/m Along Y-direction

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A′

M = 0.2794 x 1.2 x 1.8 x 0.75 x 4.00 x 2.00
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 2.00
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (4.00 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (4.00 - X)
= 3.621024 + 1.13157 + 0.11176 x 21.33333 + 0.653796 x 10.66667
= **14.11063** t-m

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

Effective depth available = 500 - 50 - 20 - 6 = 424 mm

$M = R \times b \times d^2$
= 151.1111 x 3.417 x 0.179776 = 92.81768 t-m

$$A_{st} = \frac{14.11063 \times 10^6}{20400 \times 0.888889 \times 0.424} = 1835.283 \text{ mm}^2$$

A_{st}/m = 537.1559 mm^2/m

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm^2 on earth face.

Provide 8 mm dia @ 150 mm c/c providing 335.1032 mm^2 on other face.

Along Horizantal direction.

Design of Solid Return:

Moment due to Cantilever Return:

Moment due to Earth pressure at face B-B'

M = 0.2794 x 1.2 x 1.8 x 0.75 x 4.00 x 5.75
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 5.75
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (7.75 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (7.75 - X)

= 10.41044 + 3.2532638 + 0.11176 x 101.3333 + 0.653796 x 40.66667

= **51.57643** t-m

Moment in Solid Return /m height = 14.34644 + $\frac{51.57643}{9.155}$ = 19.98013 t-m/m

Moment in Solid Return /m width = = 19.32583 t-m/m

Design of face B-B'

Moment in Solid Return /m height = **19.98013** t-m/m

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 20 - 10 = 420 mm

M = R x b x d^2
= 151.1111 x 1.000 x 0.1764 = 26.656 t-m

A_{st} = $\frac{19.98013 \times 10^6}{20400 \times 0.888889 \times 0.42}$ = 2623.441 mm^2

A_{st}/m = 2623.441 mm^2/m

Provide 25 mm dia @ 180 mm c/c providing 2727.077 mm^2 on earth face.

Provide 12 mm dia @ 180 mm c/c providing 628.3185 mm^2 on other face.

Along Horizantal direction.

Design of face A'-B'

Moment in Solid Return /m width = **19.32583** t-m/m

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 0 - 10 = 440 mm

M = R x b x d^2
= 151.1111 x 1.000 x 0.1936 = 29.25511 t-m

A_{st} = $\frac{19.32583 \times 10^6}{20400 \times 0.888889 \times 0.44}$ = 2422.187 mm^2

A_{st}/m = 2422.187 mm^2/m

Provide 25 mm dia @ 190 mm c/c providing 2583.547 mm^2 on earth face.

Provide 16 mm dia @ 190 mm c/c providing 1058.221 mm^2 on other face.

Along Vertical direction.

DISTRIBUTION FACTOR		=	0.500
Active earth pressure	K_a	=	0.279
FOR NORMAL CASE, FA		=	1.0
unit wt of soil	γ	=	1.8
	δ	=	20
clear span between return wall		=	11.000 m
width of return wall		=	0.45
Avg. cover to reinforcement. (FOR 2-3 LAYERS)		=	0.150 m
H (From formation level) =	4.500 m		

DESIGN FOR BENDING MOMENT

S.NO.	UNITS	1
H (FROM TOP)	m	4.500
WIDTH OF RETURN AT THIS LEVEL	m	3.750
FORCE DUE TO EARTH PRESSURE	t	28.669
MOMENT DUE TO EARTH PRESSURE	tm	54.184
FORCE DUE TO L.L. SURCHARGE	t	15.290
MOMENT DUE TO L.L. SURCHARGE	tm	34.403
TOTAL HORI. FORCE FROM SUPERSTR.	t	10.000
TOTAL MOMENT	tm	88.586
DESIGN MOMENT	tm	88.586
REQUIRED EFFECTIVE DEPTH	m	1.211
EFF. DEPTH AVAILABLE	m	3.600
AREA OF STEEL REQUIRED	cm^2	13.569
DIAMETER OF BAR PROVIDED	mm	32
TOTAL NO. OF BARS	no.	4
AREA OF STEEL PROVIDED	cm^2	32.17
		O.K.
CHECK FOR SHEAR STRESS		
SHEAR FORCE	t	48.959
SHEAR STRESS	t/m^2	27.199
Ast/bd x 100	%	0.075
PERMISSIBLE SHEAR STRESS	MPa	0.20
PERMISSIBLE SHEAR STRESS	t/m^2	20.40
Asv/sv	cm^2/m	0.926

```
152. DEFINE MOVING LOAD
153. ****WITH IMPACT FACTOR
154. TYPE 1 LOAD 1.57 1.57 6.63 6.63 3.9542 3.9542 3.9542 3.9542
155. DIST 1.1 3.2 1.2 4.3 3 3 3 WID 1.8
156. TYPE 2 LOAD 4.4 6.6 6.6 9.35 9.35 9.35 9.35
157. DIST 3.96 1.52 2.13 1.37 3.05 1.37 WID 1.93
158. ***** OUTER GIRDER************
159. *******CLASS 70R 1 LANE
160. LOAD GENERATION 175
161. TYPE 2 -13.4 0 4.06 XINC 0.2
162. PERFORM ANALYSIS

163. LOAD LIST 109
164. PRINT SUPPORT REACTIONS
```

SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
32	109	0.00	15.41	0.00	0.00	0.00	0.00
62	109	0.00	7.77	0.00	0.00	0.00	0.00
92	109	0.00	3.41	0.00	0.00	0.00	0.00
122	109	0.00	-1.46	0.00	0.00	0.00	0.00
44	109	0.00	48.90	0.00	0.00	0.00	0.00
74	109	0.00	36.61	0.00	0.00	0.00	0.00
104	109	0.00	0.51	0.00	0.00	0.00	0.00
134	109	0.00	-1.14	0.00	0.00	0.00	0.00

************** END OF LATEST ANALYSIS RESULT **************

165. FINISH

```
   153. DEFINE MOVING LOAD
   154. ****WITH IMPACT FACTOR
   155. TYPE 1 LOAD 1.57 1.57 6.63 6.63 3.9542 3.9542 3.9542 3.9542
   156. DIST 1.1 3.2 1.2 4.3 3 3 3 WID 1.8
   157. *******CLASS A 3 LANE
   158. LOAD GENERATION 202
   159. TYPE 1 -18.8 0 2.7 XINC 0.2
   160. TYPE 1 -18.8 0 6.2 XINC 0.2
   161. TYPE 1 -18.8 0 9.7 XINC 0.2
   162. PERFORM ANALYSIS

   163. LOAD LIST 74
   164. PRINT SUPPORT REACTIONS
 SUPPORT  REACTION

   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
```

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
32	74	0.00	47.28	0.00	0.00	0.00	0.00
62	74	0.00	24.88	0.00	0.00	0.00	0.00
92	74	0.00	35.69	0.00	0.00	0.00	0.00
122	74	0.00	21.01	0.00	0.00	0.00	0.00
44	74	0.00	16.41	0.00	0.00	0.00	0.00
74	74	0.00	10.19	0.00	0.00	0.00	0.00
104	74	0.00	10.88	0.00	0.00	0.00	0.00
134	74	0.00	8.13	0.00	0.00	0.00	0.00

```
   ************** END OF LATEST ANALYSIS RESULT **************

   165. FINISH
```

ABUTMENT SHAFT .case 1 dry 17-12-06
Depth of Section = 1.200 m
Width of Section = 12.000 m
along width-compression face- no of bar: 60 tension face- no of bar: 120
Dia (mm) 16 25
Cover (cm) 7.50 7.50
along depth-compression face- no of bar: 8 tension face- no of bar: 8
Dia (mm) 16 16
Cover (cm) 7.50 7.50
Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 597.300 T
Mxx = 1382.000 Tm
Myy = 194.000 Tm
Intercept of Neutral axis : X axis : = 243.790 m
: y axis : = .350 m
Steel Stress Governs Design
Stress in Concrete due to Loads = 80.57 Kg/cm^2
Stress in Steel due to Loads = 1821.51 Kg/cm^2
Percentage of Steel = .52 %

ABUTMENT SHAFT .case 1 dry span dislodged 17-12-06
Depth of Section = 1.200 m
Width of Section = 12.000 m
Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 263.970 T
Mxx = 1257.000 Tm
Myy = .100 Tm

Intercept of Neutral axis : X axis : = ******* m
: y axis : = .311 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 70.37 Kg/cm^2
Stress in Steel due to Loads = 1842.73 Kg/cm^2
Percentage of Steel = .52 %

ABUTMENT SHAFT .case 1 h.f.l 17-12-06
Depth of Section = 1.200 m
Width of Section = 12.000 m
along width-compression face- no of bar: 60 tension face- no of bar: 120

Dia (mm)	16	25
Cover (cm)	7.50	7.50

along depth-compression face- no of bar: 8 tension face- no of bar: 8

Dia (mm)	16	16
Cover (cm)	7.50	7.50

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 518.620 T
Mxx = 1061.000 Tm
Myy = 194.000 Tm

Intercept of Neutral axis : X axis : = 192.802 m
: y axis : = .361 m
Steel Stress Governs Design
Stress in Concrete due to Loads = 62.48 Kg/cm^2
Stress in Steel due to Loads = 1360.47 Kg/cm^2
Percentage of Steel = .52 %

ABUTMENT SHAFT .case 2 span dislodged 17-12-06
Depth of Section = 1.200 m
Width of Section = 12.000 m
Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 185.290 T
Mxx = 935.000 Tm
Myy = .100 Tm
Intercept of Neutral axis : X axis : = ******* m
: y axis : = .309 m
Steel Stress Governs Design

Stress in Concrete due to Loads = 52.29 Kg/cm^2
Stress in Steel due to Loads = 1378.83 Kg/cm^2
Percentage of Steel = .52 %

```
INPUT FILE: orissa21superDL.STD
     1. STAAD SPACE
     2. START JOB INFORMATION
     3. ENGINEER DATE 15-SEP-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0 0 0; 2 0.35 0 0; 3 0.7 0 0; 4 1.6 0 0; 5 2.9625 0 0; 6 5.575 0 0
     9. 7 8.1875 0 0; 8 10.8 0 0; 9 13.4125 0 0; 10 16.025 0 0; 11 18.6375 0 0
    10. 12 20 0 0; 13 20.9 0 0; 14 21.25 0 0; 15 21.6 0 0; 16 0 0 1.0125
    11. 17 0.35 0 1.0125; 18 0.7 0 1.0125; 19 1.6 0 1.0125; 20 2.9625 0 1.0125
12. 21 5.575 0 1.0125; 22 8.1875 0 1.0125; 23 10.8 0 1.0125; 24 13.4125 0
1.0125
    13. 25 16.025 0 1.0125; 26 18.6375 0 1.0125; 27 20 0 1.0125; 28 20.9 0
1.0125
    14. 29 21.25 0 1.0125; 30 21.6 0 1.0125; 31 0 0 2.025; 32 0.35 0 2.025
    15. 33 0.7 0 2.025; 34 1.6 0 2.025; 35 2.9625 0 2.025; 36 5.575 0 2.025
    16. 37 8.1875 0 2.025; 38 10.8 0 2.025; 39 13.4125 0 2.025; 40 16.025 0
2.025
    17. 41 18.6375 0 2.025; 42 20 0 2.025; 43 20.9 0 2.025; 44 21.25 0 2.025
    18. 45 21.6 0 2.025; 46 0 0 3.35; 47 0.35 0 3.35; 48 0.7 0 3.35; 49 1.6 0
3.35
    19. 50 2.9625 0 3.35; 51 5.575 0 3.35; 52 8.1875 0 3.35; 53 10.8 0 3.35
    20. 54 13.4125 0 3.35; 55 16.025 0 3.35; 56 18.6375 0 3.35; 57 20 0 3.35
21. 58 20.9 0 3.35; 59 21.25 0 3.35; 60 21.6 0 3.35; 61 0 0 4.675; 62 0.35 0
4.675
    22. 63 0.7 0 4.675; 64 1.6 0 4.675; 65 2.9625 0 4.675; 66 5.575 0 4.675
    23. 67 8.1875 0 4.675; 68 10.8 0 4.675; 69 13.4125 0 4.675; 70 16.025 0
4.675
    24. 71 18.6375 0 4.675; 72 20 0 4.675; 73 20.9 0 4.675; 74 21.25 0 4.675
    25. 75 21.6 0 4.675; 76 0 0 6;77 0.35 0 6; 78 0.7 0 6; 79 1.6 0 6;80
2.9625 0 6
    26. 81 5.575 0 6; 82 8.1875 0 6; 83 10.8 0 6; 84 13.4125 0 6; 85 16.025 0
6
    27. 86 18.6375 0 6; 87 20 0 6; 88 20.9 0 6; 89 21.25 0 6; 90 21.6 0 6
    28. 91 0 0 7.325; 92 0.35 0 7.325; 93 0.7 0 7.325; 94 1.6 0 7.325
    29. 95 2.9625 0 7.325; 96 5.575 0 7.325; 97 8.1875 0 7.325; 98 10.8 0
7.325
    30. 99 13.4125 0 7.325; 100 16.025 0 7.325; 101 18.6375 0 7.325; 102 20 0
7.325
    31. 103 20.9 0 7.325; 104 21.25 0 7.325; 105 21.6 0 7.325; 106 0 0 8.65
    32. 107 0.35 0 8.65; 108 0.7 0 8.65; 109 1.6 0 8.65; 110 2.9625 0 8.65
    33. 111 5.575 0 8.65; 112 8.1875 0 8.65; 113 10.8 0 8.65; 114 13.4125 0
8.65
    34. 115 16.025 0 8.65; 116 18.6375 0 8.65; 117 20 0 8.65; 118 20.9 0 8.65
    35. 119 21.25 0 8.65; 120 21.6 0 8.65; 121 0 0 9.975; 122 0.35 0 9.975
    36. 123 0.7 0 9.975; 124 1.6 0 9.975; 125 2.9625 0 9.975; 126 5.575 0
9.975
    37. 127 8.1875 0 9.975; 128 10.8 0 9.975; 129 13.4125 0 9.975;13016.025 0
9.975
    38. 131 18.6375 0 9.975; 132 20 0 9.975; 133 20.9 0 9.975; 134 21.25 0
9.975
    39. 135 21.6 0 9.975; 136 0 0 10.9875; 137 0.35 0 10.9875; 138 0.7 0
10.9875
    40. 139 1.6 0 10.9875; 140 2.9625 0 10.9875; 141 5.575 0 10.9875
    41. 142 8.1875 0 10.9875; 143 10.8 0 10.9875; 144 13.4125 0 10.9875
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    42. 145 16.025 0 10.9875; 146 18.6375 0 10.9875; 147 20 0 10.9875
    43. 148 20.9 0 10.9875; 149 21.25 0 10.9875; 150 21.6 0 10.9875; 151 0 0
12
    44. 152 0.35 0 12; 153 0.7 0 12; 154 1.6 0 12; 155 2.9625 0 12; 156 5.575
0 12
    45. 157 8.1875 0 12; 158 10.8 0 12; 159 13.4125 0 12; 160 16.025 0 12
    46. 161 18.6375 0 12; 162 20 0 12; 163 20.9 0 12; 164 21.25 0 12; 165 21.6
0 12
    48. **********************************************
    49. *          STAAD.PRO GENERATED COMMENT          *
    50. **********************************************
    51. *1 1 2 14
    52. *15 16 17 28
    53. *29 31 32 42
    54. *43 46 47 56
    55. *57 61 62 70
    56. *71 76 77 84
    57. *85 91 92 98
    58. *99 106 107 112
59. *113 121 122 126
    60. *127 136 137 140
    61. *141 151 152 154
    62. ***********************************************
    63. MEMBER INCIDENCES
    64. 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
65. 11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 16 17; 16 17 18; 17 18 19;18 19
20
66. 19 20 21; 20 21 22; 21 22 23; 22 23 24; 23 24 25; 24 25 26; 25 26 27; 26
27 2 8
67. 27 28 29; 28 29 30; 29 31 32; 30 32 33; 31 33 34; 32 34 35; 33 35 36; 34
36    37
68. 35 37 38; 36 38 39; 37 39 40; 38 40 41; 39 41 42; 40 42 43; 41 43 44; 42
44 45
69. 43 46 47; 44 47 48; 45 48 49; 46 49 50; 47 50 51; 48 51 52; 49 52 53; 50
53 54
70. 51 54 55; 52 55 56; 53 56 57; 54 57 58; 55 58 59; 56 59 60; 57 61 62; 58
62 63
71. 59 63 64; 60 64 65; 61 65 66; 62 66 67; 63 67 68; 64 68 69; 65 69 70; 66
70  71
    72. 67 71 72; 68 72 73; 69 73 74; 70 74 75; 71 76 77; 72 77 78; 73 78 79;
74 79 80
73. 75 80 81; 76 81 82; 77 82 83; 78 83 84; 79 84 85; 80 85 86; 81 86 87; 82
87 88
74. 83 88 89; 84 89 90; 85 91 92; 86 92 93; 87 93 94; 88 94 95; 89 95 96; 90
96 97
75. 91 97 98; 92 98 99; 93 99 100; 94 100 101; 95 101 102; 96 102 103; 97 103
104
    76. 98 104 105; 99 106 107; 100 107 108; 101 108 109; 102 109 110; 103 110
111
77. 104 111 112; 105 112 113; 106 113 114; 107 114 115; 108 115 116; 109 116
117
78. 110 117 118; 111 118 119; 112 119 120; 113 121 122; 114 122 123; 115 123
124
79. 116 124 125; 117 125 126; 118 126 127; 119 127 128; 120 128 129; 121 129
130
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80. 122 130 131; 123 131 132; 124 132 133; 125 133 134; 126 134 135; 127 136
137
81. 128 137 138; 129 138 139; 130 139 140; 131 140 141; 132 141 142; 133 142
143
82. 134 143 144; 135 144 145; 136 145 146; 137 146 147; 138 147 148; 139 148
149
83. 140 149 150; 141 151 152; 142 152 153; 143 153 154; 144 154 155; 145 155
156
84. 146 156 157; 147 157 158; 148 158 159; 149 159 160; 150 160 161; 151 161
162
    85. 152 162 163; 153 163 164; 154 164 165; 155 1 16; 156 16 31; 157 31 46
    86. 158 46 61; 159 61 76; 160 76 91; 161 91 106; 162 106 121; 163 121 136
87. 164 136 151; 165 2 17; 166 17 32; 167 32 47; 168 47 62; 169 62 77; 170 77
92
    88. 171 92 107; 172 107 122; 173 122 137; 174 137 152; 175 3 18; 176 18 33
    89. 177 33 48; 178 48 63; 179 63 78; 180 78 93; 181 93 108; 182 108 123
    90. 183 123 138; 184 138 153; 185 4 19; 186 19 34; 187 34 49; 188 49 64;
189 64 79
    91. 190 79 94; 191 94 109; 192 109 124; 193 124 139; 194 139 154; 195 5 20
92. 196 20 35; 197 35 50; 198 50 65; 199 65 80; 200 80 95; 201 95 110; 202 110
125
93. 203 125 140; 204 140 155; 205 6 21; 206 21 36; 207 36 51; 208 51 66; 209
66 81
    94. 210 81 96; 211 96 111; 212 111 126; 213 126 141; 214 141 156; 215 7 22
95. 216 22 37; 217 37 52; 218 52 67; 219 67 82; 220 82 97; 221 97 112; 222 112
127
  96. 223 127 142; 224 142 157; 225 8 23; 226 23 38; 227 38 53; 228 53 68; 229
68 83
    97. 230 83 98; 231 98 113; 232 113 128; 233 128 143; 234 143 158; 235 9 24
 98. 236 24 39; 237 39 54; 238 54 69; 239 69 84; 240 84 99; 241 99 114; 242
114 129
    99. 243 129 144; 244 144 159; 245 10 25; 246 25 40; 247 40 55; 248 55 70
    100. 249 70 85; 250 85 100; 251 100 115; 252 115 130; 253 130 145; 254 145
160
   101. 255 11 26; 256 26 41; 257 41 56; 258 56 71; 259 71 86; 260 86 101; 261
101 116
   102. 262 116 131; 263 131 146; 264 146 161; 265 12 27; 266 27 42; 267 42 57
   103. 268 57 72; 269 72 87; 270 87 102; 271 102 117; 272 117 132; 273 132
147
   104. 274 147 162; 275 13 28; 276 28 43; 277 43 58; 278 58 73; 279 73 88;
280 88 103
   105. 281 103 118; 282 118 133; 283 133 148; 284 148 163; 285 14 29; 286 29
44
   106. 287 44 59; 288 59 74; 289 74 89; 290 89 104; 291 104 119; 292 119 134
   107. 293 134 149; 294 149 164; 295 15 30; 296 30 45; 297 45 60; 298 60 75
   108. 299 75 90; 300 90 105; 301 105 120; 302 120 135; 303 135 150; 304 150
165
   109. DEFINE MATERIAL START
   110. ISOTROPIC MATERIAL1
   111. E 2.7386E+006
   112. POISSON 0.156297
   113. ISOTROPIC MATERIAL2
   114. E 2.7386E+006
   115. POISSON 0.15
   116. DENSITY 2.4
   117. END DEFINE MATERIAL
   118. MEMBER PROPERTY INDIAN
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   119. **** OUTER GIRDER UNIFORM
   120. 29 30 41 42 113 114 125 126 PRIS AX 2 IX 0.119 IY 0.8439 IZ 0.865
   121. **** INNER GIRDER UNIFORM
   122. 57 58 69 70 85 86 97 98 PRIS AX 1.826 IX 0.117 IY 0.4236 IZ 0.792
   123. **** OUTER GIRDER FLARING
   124. 31 40 115 124 PRIS AX 1.77 IX 0.072 IY 0.8189 IZ 0.7995
   125. **** INNER GIRDER FLARING
   126. 59 68 87 96 PRIS AX 1.5975 IX 0.06985 IY 0.411 IZ 0.735
   127. **** OUTER GIRDER MIDSPAN
   128. 32 TO 39 116 TO 123 PRIS AX 1.5437 IX 0.0245 IY 0.794 IZ 0.734
   129. **** INNER GIRDER MIDSPAN
   130. 60 TO 67 88 TO 95 PRIS AX 1.369 IX 0.0227 IY 0.3984 IZ 0.678
   131. ****** END DIAPHRAGM
   132. 167 TO 172 287 TO 292 PRIS AX 0.955 IX 0.01877 IY 0.1366 IZ 0.4167
   133. ****** CROSS GIRDER
   134. 227 TO 232 PRIS AX 0.653125 IX 0.006803 IY 0.3715 IZ 0.0034
   135. ****** NOMINAL LONGITUDINAL MEMBERS
   136. 1 TO 28 43 TO 56 71 TO 84 PRIS AX 0.0001 IX 0.0001 IY 0.0001 IZ 0.0001
   137. 99 TO 112 127 TO 154 PRIS AX 0.0001 IX 0.0001 IY 0.0001 IZ 0.0001
   138. ****** TRANSVERSE MEMBERS
   139. 185 TO 194 265 TO 274 PRIS AX 0.283 IX 0.003 IY 0.03 IZ 0.001473
   140. 195 TO 204 255 TO 264 PRIS AX 0.4969 IX 0.005176 IY 0.1636 IZ 0.002588
   141. 205 TO 224 235 TO 254 PRIS AX 0.654 IX 0.0068 IY 0.3715 IZ 0.003402
   142. **** NOMINAL TRANSVERSEMEMBERS
143. 155 TO 166 173 TO 184 225 226 PRIS AX 0.0001 IX 0.0001 IY 0.0001 IZ
0.0001
   144. 233 234 275 TO 286 293 TO 304 PRIS AX 0.0001 IX 0.0001 IY 0.0001 IZ
0.0001
   145. SUPPORTS
   146. 32 62 92 122 PINNED
   147. 44 74 104 134 PINNED
   148. CONSTANTS
   149. MATERIAL MATERIAL1 MEMB 1 TO 28 43 TO 56 71 TO 84 99 TO 112 127 TO 304
   150. MATERIAL MATERIAL2 MEMB 29 TO 42 57 TO 70 85 TO 98 113 TO 126
   151. LOAD 1 SELFWEIGHT
   152. SELF Y -1
   153. LOAD 2
   154. JOINT LOAD
   155. 32 62  FY -1.5264
   156. 62 92  FY -3.0258
   157. 44  134 FY -1.5264
   158. 74 104 FY -3.0258
   159. 38  128 FY -1.5264
   160. 68 98 FY -3.0258
   161. PERFORM ANALYSIS
   162. PRINT SUPPORT REACTIONS
 SUPPORT  REACTION

   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------

 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


     32    1      0.00     41.13      0.00      0.00      0.00      0.00
           2      0.00      2.41      0.00      0.00      0.00      0.00
     62    1      0.00     36.88      0.00      0.00      0.00      0.00
```

	2	0.00	5.94	0.00	0.00	0.00	0.00
92	1	0.00	36.88	0.00	0.00	0.00	0.00
	2	0.00	4.41	0.00	0.00	0.00	0.00
122	1	0.00	41.13	0.00	0.00	0.00	0.00
	2	0.00	0.89	0.00	0.00	0.00	0.00
44	1	0.00	41.13	0.00	0.00	0.00	0.00
	2	0.00	2.41	0.00	0.00	0.00	0.00
74	1	0.00	36.88	0.00	0.00	0.00	0.00
	2	0.00	4.41	0.00	0.00	0.00	0.00
104	1	0.00	36.88	0.00	0.00	0.00	0.00
	2	0.00	4.41	0.00	0.00	0.00	0.00
134	1	0.00	41.13	0.00	0.00	0.00	0.00
	2	0.00	2.41	0.00	0.00	0.00	0.00

************** END OF LATEST ANALYSIS RESULT **************

163. FINISH

```
150. *** CRASH BARRIER
151. LOAD  1 CRASH
152. MEMBER LOAD
153. 29 TO 42 113 TO 126 UNI GY -1.0
154. 29 TO 42 UMOM GX -2.025
155. 113 TO 126 UMOM GX 2.025
156. LOAD 2 WEARING COAT
157. MEMBER LOAD
158. 29 TO 42 113 TO 126 UNI GY -0.570
159. 57 TO 70 85 TO 98 UNI GY -0.53
160. 1 TO 14 141 TO 154 UNI GY -0.1878
161. PERFORM ANALYSIS


162. PRINT SUPPORT REACTIONS
SUPPORT  REACTION

SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
-----------------
```

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
32	1	0.00	18.26	0.00	0.00	0.00	0.00
	2	0.00	9.59	0.00	0.00	0.00	0.00
62	1	0.00	-7.46	0.00	0.00	0.00	0.00
	2	0.00	4.32	0.00	0.00	0.00	0.00
92	1	0.00	-7.46	0.00	0.00	0.00	0.00
	2	0.00	4.32	0.00	0.00	0.00	0.00
122	1	0.00	18.26	0.00	0.00	0.00	0.00
	2	0.00	9.59	0.00	0.00	0.00	0.00
44	1	0.00	18.26	0.00	0.00	0.00	0.00
	2	0.00	9.59	0.00	0.00	0.00	0.00
74	1	0.00	-7.46	0.00	0.00	0.00	0.00
	2	0.00	4.32	0.00	0.00	0.00	0.00
104	1	0.00	-7.46	0.00	0.00	0.00	0.00
	2	0.00	4.32	0.00	0.00	0.00	0.00
134	1	0.00	18.26	0.00	0.00	0.00	0.00
	2	0.00	9.59	0.00	0.00	0.00	0.00

```
************** END OF LATEST ANALYSIS RESULT **************

163. FINISH
```

Seismic case

Calculation of seismic coefficient

Transverse direction

As per modified clause for interim measures for seismic provisions of IRC:6-2000

Horizantal seismic coefficient A_h = $\dfrac{Z/2 \times S_a/g}{R/I}$

Where Zone factor	Z	=	0.10
Importance factor	I	=	1.50
Response reduction factor	R	=	2.50

S_a/g is average response acceleration coefficient for 5% damping depending upon fundamental period of vibration T

$$T = 2.00\sqrt{\frac{D}{1000 \times F}}$$

Where

D is appropriate dead load of superstructure and Live load

DL	184.71	t
SIDL	36.24	t
LL	112.38	t
DL+SIDL+50% LL =	2771.38	kN

F is horizantal force required to be applied at centre of mass of superstructure for 1mm Horizantal deflection at top of abutment along considered direction of motion

Relation between deflection at x from free end of cantilever , subjected to force 'F' at free end is given by

$$\mathbf{F} = \frac{E\ I\ y}{(L^2.x/2)-(x^3/6)-(L^3/3)}$$

Modulus of elasticity of concrete	E	27386.13		
		2793385.0	t/m^2	
	I	172.80	m^4	
Deflection at distance x from free end of cantilever				
	y	=	0.001	m
Distance from fixed end of cantilever to point of application of load				
	L	=	6.65	m
Distance of CG of super structure from deck slab top			0.67	m
Distance of CG of super structure from free end of cantilever				
	x	=	1.43	m
	F	=	72161.98	kN
Corresponding time period	T	=	0.01	sec
	S_a/g	=	2.50	
There fore	A_h	=	0.075	

Longitudinal direction

As per modified clause for interim measures for seismic provisions of IRC:6-2000

Horizantal seismic coefficient $A_h = \dfrac{Z/2 \times S_a/g}{R/I}$

Where Zone factor Z = 0.10

Importance factor I = 1.50

Response reduction factor R = 2.50

S_a/g is average response acceleration coefficient for 5% damping depending upon fundamental period of vibration T

$$T = 2.00 \sqrt{\frac{D}{1000xF}}$$

Where

D is appropriate dead load of superstructure and Live load

For longitudinal direction, live load is not to be considered.

DL+SILL = 2209.48 kN

F is horizantal force required to be applied at centre of mass of superstructure for 1mm Horizantal deflection at top of abutment along considered direction of motion

Relation between deflection at x from free end of cantilever , subjected to force 'F' at free end is given by

$$\mathbf{F} = \frac{E\ I\ y}{(L^2.x/2)-(x^3/6)-(L^3/3)}$$

Modulus of elasticity of concrete E 27386.13

2793385.0 t/m^2

I 1.73 m^4

Deflection at distance x from free end of cantilever

y = 0.001 m

Distance from fixed end of cantilever to point of application of load

L = 6.65 m

Top of abutment from free end of cantilever x = 1.43 m

F = 721.62 kn

Corresponding time period T = 0.11 sec

S_a/g = 2.50

There fore A_h = 0.075

Ground level 90.145

Weight due to Concrete components

	component	Area factor	Length	Width	Height	Density	Seismic Force	L.A	Moment
1	Dirt wall	1	12.0	0.3	2.206	2.4	1.429	9.552	13.654
2	Abut cap	1	12.0	1.2	0.5	2.4	1.296	8.199	10.626
3	Abut shaft	1	12.0	1.2	5.149	2.4	13.346	4.875	65.056
							16.072		**89.336**

Lever arm for the loads from super structure

Formation level	98.300	m
Wearing coat	56	mm
Top of deck slab	98.244	m
Distance of CG of super structure below deck slab top	0.671	m
Pile cap bottom	87.345	m
Lever arm for DL	10.228	m

Distance of CG of SIDL above deck slab top	0.274	m
Level arm for SIDL	11.173	m
Distance of CG of live load above formation level	1.2	
Lever arm for live load	12.155	

Load from Superstruture

Longitudinal direction

	Weight	Force	L.A	moment
DL	184.71	13.85	10.228	141.69
SIDL	36.24	2.72	11.173	30.36
Total		**16.57**		**172.05**

Transverse direction

	Weight	Force	L.A	moment
DL	184.71	13.853	10.23	141.69
SIDL	36.24	2.718	11.17	30.36
LL 50%	56.19	4.214	12.16	51.22
Total		**20.785**		**223.27**

Summary

seismic longitudinal case

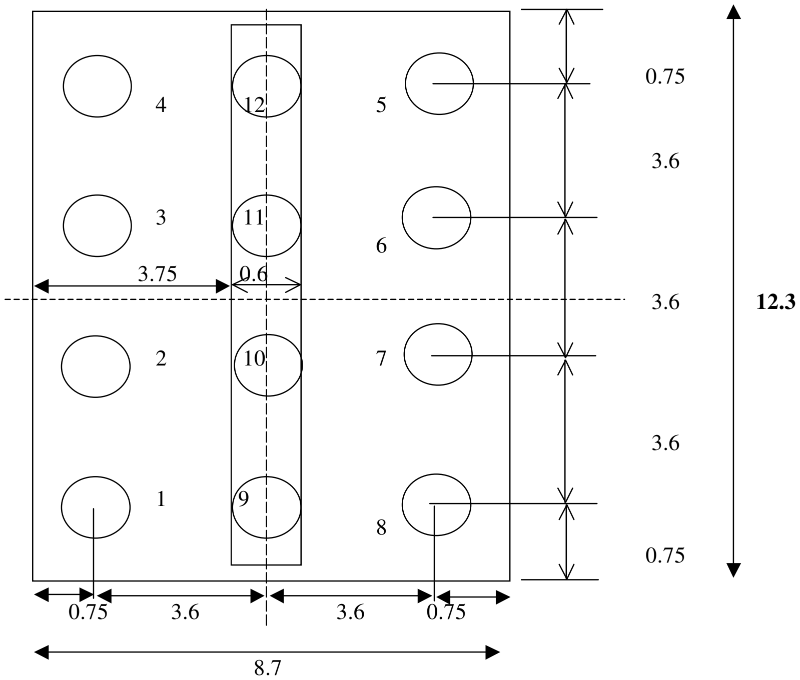
	P	M_L	M_T
Total load on piles for normal case	1865.75	240.62	193.39
Additional horizantal force and moment due to seismic	-	261.386	-
(Live load not to be considered for longitudinal moment)			
Reduction in longitudinal moment due to braking force	-	-102.44	-
Reduction in transverse moment due to live load			-193.39
	1865.75	**399.566**	**0.00**

seismic Transverse case

		P		M_L	M_T
Total load on piles for normal case		1865.75		240.62	193.39
Additional horizantal force and moment due to seismic		-		-	312.61
(Only 50%% live load is to be considered for transverse moment)					
Reduction in longitudinal moment due to braking force		-		-79.5 -	
Reduction in transverse moment due to live load	-		-		-96.70
		1865.75		**161.12**	**409.31**

Load Case	**Total Load**	**Longitudinal Moment**	**Transverse Moment**
	P	**M_L**	**M_T**
I Seismic Longitudinal direction			
1 With L.L	1865.75	399.57	0.00
II **Seismic Transverse direction**			
1 With LL	1865.75	161.12	409.31

No of Piles = 12



Pile No		x	Z_L	y	Z_T
1		3.6	28.8	5.4	36
2		3.6	28.8	1.8	108
3		3.6	28.8	1.8	108
4		3.6	28.8	5.4	36
5		3.6	28.8	5.4	36
6		3.6	28.8	1.8	108
7		3.6	28.8	1.8	108
8		3.6	28.8	5.4	36
9		0	0	5.4	36
10		0	0	1.8	108
11		0	0	1.8	108
12		0	0	5.4	36
		103.68		194.4	

Case I **Seismic Longitudinal direction**

1

1 $P_{max,min} = \frac{1865.75}{12} - \frac{399.57}{28.80} + \frac{0.00}{36.00}$ $P = 141.61$ t

2 $P_{max,min} = \frac{1865.75}{12} - \frac{399.57}{28.80} + \frac{0.00}{108.00}$ $P = 141.61$ t

3 $P_{max,min} = \frac{1865.75}{12} - \frac{399.57}{28.80} - \frac{0.00}{108.00}$ $P = 141.61$ t

4 $P_{max,min} = \frac{1865.75}{12} - \frac{399.57}{28.80} - \frac{0.00}{36.00}$ $P = 141.61$ t

5 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{28.80} - \frac{0.00}{36.00}$ $P = 169.35$ t

6 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{28.80} - \frac{0.00}{108.00}$ $P = 169.35$ t

7 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{28.80} + \frac{0.00}{108.00}$ $P = 169.35$ t

8 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{28.80} + \frac{0.00}{36.00}$ $P = 169.35$ t

9 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{0.00} + \frac{0.00}{36.00}$ $P = 155.48$ t

10 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{0.00} + \frac{0.00}{108.00}$ $P = 155.48$ t

11 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{0.00} - \frac{0.00}{108.00}$ $P = 155.48$ t

12 $P_{max,min} = \frac{1865.75}{12} + \frac{399.57}{0.00} - \frac{0.00}{36.00}$ $P = 155.48$ t

II **Seismic Transverse direction**

1 1 $P_{max,min} = \frac{1865.75}{12} - \frac{161.12}{28.80} + \frac{409.31}{36.00}$ $P = 161.25$ t

2 $P_{max,min} = \frac{1865.75}{12} - \frac{161.12}{28.80} + \frac{409.31}{108.00}$ $P = 153.67$ t

3 $P_{max,min} = \frac{1865.75}{12} - \frac{161.12}{28.80} - \frac{409.31}{108.00}$ $P = 146.10$ t

4 $P_{max,min} = \frac{1865.75}{12} - \frac{161.12}{28.80} - \frac{409.31}{36.00}$ $P = 138.52$ t

5 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{28.80} - \frac{409.31}{36.00}$ $P = 149.70$ t

6 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{28.80} - \frac{409.31}{108.00}$ $P = 157.28$ t

7 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{28.80} + \frac{409.31}{108.00}$ $P = 164.86$ t

8 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{28.80} + \frac{409.31}{36.00}$ $P = 172.44$ t

9 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{0.00} + \frac{409.31}{36.00}$ $P = 166.85$ t

10 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{0.00} + \frac{409.31}{108.00}$ $P = 159.27$ t

11 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{0.00} - \frac{409.31}{108.00}$ $P = 151.69$ t

12 $P_{max,min} = \frac{1865.75}{12} + \frac{161.12}{0.00} - \frac{409.31}{36.00}$ $P = 144.11$ t

Seismic Longitudina	P_{max}	=		**169.35**	t
	P_{min}	=		**141.61**	t
Seismic Transverse	P_{max}	=		**172.44**	t
	P_{min}	=		**138.52**	t

Permissible capacity of pile in normal case = 207 t

Permissible capacity of pile in seismic case = 1.33 x 207 = 255.3 t > 172.44 t

Hence OK

Horizantal load per pile due to normal & seismic
Seismic Longitudinal direction:

Total horizantal force due to deck movement	=	6.36	t	
Horizantal seismic force due superstructure (DL+SIDL)	=	16.57	t	
(Live load not to be considered for longitudinal direction)				
Horizantal seismic force due to abutment avove G.L	=	16.07	t	
	=	**39.00**	t	
Number of Piles	=	12.00		
	=	3.25	t	per pile
Horizantal force in Normal condition due to earth	=	442.33	t	
	=	36.86	t	per pile
Therefore Total Horizantal force per pile	=	**40.11**	t	per pile
Maximum Load on the Pile	=	**169.35**		
Minimum Load on the Pile	=	**141.61**		

Determination of Lteral Deflection of pile :

$$T = \sqrt[5]{\frac{E \times I}{K_1}}$$

E = Youngs modulus kg/m^2

K_1 = Constant kg/cm^3

$$R = \sqrt[4]{\frac{E \times I}{K_2}}$$

K_2 kg/cm^2

I = moment of inertia of pile cross section cm^4

Dia of pile = 1.20 m

$E = 5000 \times \sqrt{35} = 301533\ kg/cm^2$

$I = \frac{3.14}{64} \times 120\ \hat{}\ 4 = 10178760\ cm^4$

$K_1 = 0.775$

$K_2 = 0$

$T = 330.78$

$R = 0$

L_e = Embeded Length of pile

= 87.35 - 73.845 = 1350 cm

L_1 = Exposed Length = 87.35 - 87.35 = 0 cm

$\frac{L_1}{T} = 0$

$\frac{L_f}{T} = 2.19$

$L_f = 724.42$ cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12\ E\ I} = \frac{40.111 \times 724.42}{12 \times 301533 \times 10178760}$$

$$= 0.414 \text{ cm}$$
$$= 4.14 \text{ mm}$$

In seismic case as permissble increase in deflection is 33%, hence

Permissible deflection = 5 x 1.33 = 6.65 mm > 4.1402 mm

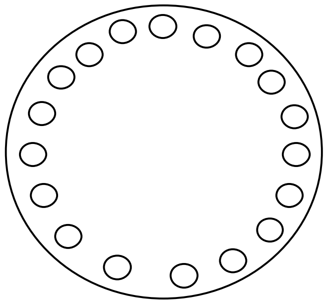
Maximum Moment

$$M_F = Q \times \frac{\left(L_1 + L_f\right)}{2} = 145.29 \text{ t-m}$$

$$M = m \times M_F$$
$$= 0.83 \times 145.29 = 120.59 \text{ t-m}$$

Reinforcement in pile:

Provide 25 mm dia 28 nos 137.44 cm^2



Provide 8 mm Rings at 180 mm c/c

Horizantal load per pile due to normal & seismic
Seismic Transverse direction:

Total horizantal force due to deck movement	=	12.050	t	
Horizantal seismic force due superstructure (DL+SIDL)	=	16.571	t	
Horizantal seismic force due superstructure (LL)	=	4.214	t	
Horizantal force due to abutment avove G.L	=	16.072	t	
	=	**36.86**	t	
Number of Piles	=	12.00		
	=	3.071	t	per pile
Horizantal force in Normal condition due to earth	=	442.33	t	
	=	37.865	t	per pile
Therefore Total Horizantal force per pile	=	**37.99**	t	per pile
Maximum Load on the Pile	=	**172.44**		
Minimum Load on the Pile	=	**138.52**		

Determination of Lteral Deflection of pile :

$$T = \sqrt[5]{\frac{E \times I}{K_1}}$$

E = Youngs modulus kg/m^2

K_1 = Constant kg/cm^3

$$R = \sqrt[4]{\frac{E \times I}{K_2}}$$

K_2 kg/cm^2

I = moment of inertia of pile cross section cm^4

Dia of pile = 1.20

$E = 5000 \times \sqrt{35} = 301533$ kg/cm^2

610.73

$I = \frac{3.14}{64} \times 120 = 10178760$ cm^4

K_1 = 0.775

K_2 = 0

T = 330.78

R = 0

L_e = Embeded Length of pile

= 87.35 - 73.845 = 1350 cm

L_1 = Exposed Length = 87.35 - 87.35 = 0 cm

$$\frac{L_1}{T} = 0$$

$$\frac{L_f}{T} = 2.19$$

L_f = 724.42 cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12\ E\ I} = \frac{37.99 \times 724.42}{12 \times 301533 \times 10178760}$$

= 0.392 cm

= 3.92 mm

in seismic transverse case permissible increase in deflection = 1.33 x 5 6.65 mm > 3.92 mm

Maximum Moment

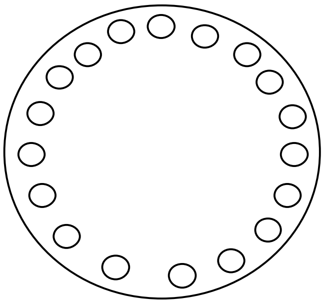
$$M_F = Q \times \frac{\left(L_1 + L_f\right)}{2} = 137.6 \text{ t-m}$$

$$M = m \times M_F$$

$$= 0.83 \times 137.6 = 114.21 \text{ t-m}$$

Reinforcement in pile:

Provide 25 mm dia 28 nos 137.44 cm^2



Provide 8 mm Rings at 180 mm c/c

DESIGN OF PILE SECTION		
INPUT DATA	**Seismic long. Case**	
	MAX. LOAD	**MIN. LOAD**
D=DIA OF PILE (in m)	1.200	1.200
C=COVER FROM CENTRE OF BAR (in m)	0.075	0.075
M=MODULAR RATIO (as per IRC:6)	10.000	10.000
BD=BETA ANGLE FOR DEFINING N.A. (in degree)	98.46	104.13
AS=AREA OF STEEL (in sq. Cm)	137.445	137.445
P=VERTICAL LOAD (in tonnes)	169.35	141.61
BM=BENDING MOMENT (in t-m)	120.59	120.59
SOLUTION		
RO=OUTER RADIUS=D/2	0.600	0.600
RI=RADIUS OF REINFORCEMENT RING=D/2-C	0.525	0.525
T= THICK.REINFORCEMENT RING = AS/2*3.142*RI	0.004	0.004
B=BD*PI/180	1.718	1.817
B1=COS(B)	-0.147	-0.244
A=ALPHA ANGLE=ACOS (RO*B1/RI)	1.740	1.853
RECOSB=RICOSA		
B2=SIN(B)^3	0.968	0.912
B3=SIN(4*B)	0.557	0.834
B4=SIN(2*B)	-0.291	-0.473
A=COS(A)	-0.168	-0.279
A2=SIN(2*A)	-0.331	-0.536
A3=SIN(A)	0.986	0.960
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.148	0.117
NUM2=2*(RO^3)/(1+B1)	0.507	0.571
NUM3=(RI^3) *T/(RO+RI*A1)	0.001	0.001
NUM4=(M-1) *PI+A-A2/2	30.18	30.40
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.110	0.107
DENM1=2*(RO^2)/(1+B1)	0.844	0.952
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	0.229	0.171
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.004	0.005
DENM4=(M-1) *PI*A1-A3+A*A1	-6.03	-9.36
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.166	0.116
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.666**	**0.929**
ACTUAL ECCENTRICITY EC = M/P	**0.712**	**0.852**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	-0.088	-0.146
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.512	0.454
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**1021**	**1224**
TS=TENS.STRESS IN STEEL (T/SQM)	**12233**	**18115**
PERMISSIBLE COMP.STRESS IN Concrete 50% increase (in t/sq.m)	1785	1785
PERMISSIBLE TENSILE STRESS IN Steel 50% increase(in t/sq.m)	30600	30600

Hence stresses in conc and steel are within permissible limits, hence safe.

DESIGN OF PILE SECTION		
INPUT DATA	**Seismic trans. Case**	
	MAX. LOAD	**MIN. LOAD**
D=DIA OF PILE (in m)	1.200	1.200
C=COVER FROM CENTRE OF BAR (in m)	0.075	0.075
M=MODULAR RATIO (as per IRC:6)	10.000	10.000
BD=BETA ANGLE FOR DEFINING N.A. (in degree)	97.16	103.44
AS=AREA OF STEEL (in sq. Cm)	137.445	137.445
P=VERTICAL LOAD (in tonnes)	172.44	138.52
BM=BENDING MOMENT (in t-m)	114.21	114.21
SOLUTION		
RO=OUTER RADIUS=D/2	0.600	0.600
RI=RADIUS OF REINFORCEMENT RING=D/2-C	0.525	0.525
T= THICK.REINFORCEMENT RING = AS/2*3.142*RI	0.004	0.004
B=BD*PI/180	1.696	1.805
B1=COS(B)	-0.125	-0.232
A=ALPHA ANGLE=ACOS (RO*B1/RI)	1.714	1.840
RECOSB=RICOSA		
B2=SIN(B)^3	0.977	0.920
B3=SIN(4*B)	0.479	0.807
B4=SIN(2*B)	-0.247	-0.452
A=COS(A)	-0.142	-0.266
A2=SIN(2*A)	-0.282	-0.512
A3=SIN(A)	0.990	0.964
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.155	0.121
NUM2=2*(RO^3)/(1+B1)	0.493	0.563
NUM3=(RI^3) *T/(RO+RI*A1)	0.001	0.001
NUM4=(M-1) *PI+A-A2/2	30.13	30.37
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.111	0.108
DENM1=2*(RO^2)/(1+B1)	0.822	0.938
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	0.243	0.178
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.004	0.005
DENM4=(M-1) *PI*A1-A3+A*A1	-5.26	-8.96
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.177	0.122
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.628**	**0.884**
ACTUAL ECCENTRICITY EC = M/P	**0.662**	**0.825**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	-0.075	-0.139
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.525	0.461
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**974**	**1136**
TS=TENS.STRESS IN STEEL (T/SQM)	**11120**	**16389**
PERMISSIBLE COMP.STRESS IN Concrete 50% increase (in t/sq.m	1785	1785
PERMISSIBLE TENSILE STRESS IN Steel 50% increase(in t/sq.m)	30600	30600

Hence stresses in conc and steel are within permissible limits, hence safe.

Seismic check for Abutment wall

Seismic coefficient for longitudinal direction	0.075
Seismic coefficient for transverse direction	0.075

Weight due to Concrete components above pile cap

	component	Area factor	Length	Width	Height	Density	Seismic Force	L.A	Moment
1	Dirt wall	1	12.0	0.3	2.206	2.4	1.429	7.752	11.081
2	Abut cap	1	12.0	1.2	0.50	2.4	1.296	6.399	8.293
3	Abut shaft	1	12.0	1.2	5.149	2.4	13.346	3.075	41.033
							16.072		**60.407**

Lever arm for the loads from super structure for moment at base of abutment wall

Formation level	98.300	m
Wearing coat	56	mm
Top of deck slab	98.244	m
Distance of CG of super structure below deck slab top	0.671	m
Pile cap Top	89.145	
Pile cap bottom	87.345	m
Lever arm for DL	8.428	m
Distance of CG of SIDL above deck slab top	0.274	m
Level arm for SIDL	9.373	m
Distance of CG of live load above formation level	1.2	
Lever arm for live load	10.355	

Load from Superstruture

Longitudinal direction

	Weight	Force	L.A	moment
DL	184.71	13.85	8.428	116.75
SIDL	36.24	2.72	9.373	25.47
Total		**16.57**		**142.22**

Transverse direction

	Weight	Force	L.A	moment
DL	184.71	13.853	8.43	116.75
SIDL	36.24	2.718	9.37	25.47
LL 50%	56.19	4.214	10.36	43.64
Total		**20.785**		**185.86**

Summary

seismic longitudinal case

	P	M_L	M_T
Total load on abutment shaft for normal case	597.30	1382.00	194.00
Additional horizantal force and moment due to seismic	-	202.629	-
(Live load not to be considered for longitudinal moment)			
Reduction in longitudinal moment due to braking force	-	-102.44	-
Reduction in transverse moment due to live load			-194.00
	597.30	**1482.190**	**0.00**

Stress in concrete due to loads	=	87.29	**kg/cm^2**	**<**	**153.00**	**kg/cm^2**
Stress in steel due to loads	=	2015.78		**<**	**3058.80**	**kg/cm^2**

The stresses are well within permissible limits.

seismic Transverse case

	P	M_L	M_T
Total load on abutment shaft for normal case	597.30	1382.00	194.00
Additional horizantal force and moment due to seismic	-	-	246.27
(Only 50%% live load is to be considered for transverse moment)			
Reduction in longitudinal moment due to braking force	-	-84.340	-
Reduction in transverse moment due to live load	- -		-96.70
	597.30	**1297.66**	**343.57**

Stress in concrete due to loads	=	80.16	**kg/cm^2**	**<**	**153.00**	**kg/cm^2**
Stress in steel due to loads	=	1745.81		**<**	**3058.80**	**kg/cm^2**

The stresses are within permissible limits.

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     CHECK FOR seismic longitudinal

     Depth of Section             =     1.200 m
     Width of Section             =    12.000 m

along width-compression face- no of bar:  60   tension face- no of bar: 120
Dia (mm)                                  16                              25
Cover (cm)                               7.50                          10.50
along depth-compression face- no of bar:   8   tension face- no of bar:   8
Dia (mm)                                  16                              16
Cover  (cm)                              7.50                           7.50

     Modular Ratio : Compression  =      10.0
     Modular Ratio : Tension      =      10.0
     Allowable Concrete Stress    =    153.00 Kg/cm^2
     Allowable Steel Stress       =   3058.50 Kg/cm^2

     Axial Load                   =   597.300 T
     Mxx                          =  1482.190 Tm
     Myy                          =      .010 Tm


     Intercept of Neutral axis : X axis : = ******* m
                               : y axis : =    .331 m

                  Steel Stress Governs Design

     Stress in Concrete due to Loads =   87.29 Kg/cm^2
     Stress in Steel    due to Loads = 2015.78 Kg/cm^2
     Percentage of Steel             =     .52 %

     CHECK FOR seismic transverse

     Depth of Section             =     1.200 m
     Width of Section             =    12.000 m

     Modular Ratio : Compression  =      10.0
     Modular Ratio : Tension      =      10.0
     Allowable Concrete Stress    =    153.00 Kg/cm^2
     Allowable Steel Stress       =   3058.50 Kg/cm^2

     Axial Load                   =   597.300 T
     Mxx                          =  1297.660 Tm
     Myy                          =   343.570 Tm


     Intercept of Neutral axis : X axis : = 135.971 m
                               : y axis : =    .354 m

                  Steel Stress Governs Design

     Stress in Concrete due to Loads =   80.16 Kg/cm^2
     Stress in Steel    due to Loads = 1745.81 Kg/cm^2
     Percentage of Steel             =     .52 %
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DESIGN OF SUPERSTRUCTURE

For Design of Superstructure of RCC girder 21 m span refer MOST STANDARD Drawing titled “STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and slab superstructure)” Drg. No. SD/220 to SD/226.