GOVERNMENT OF ORISSA

WORKS DEPARTMENT

ORISSA STATE ROAD PROJECT

FINAL DETAILED ENGINEERING REPORT
FOR PHASE-I ROADS

DESIGN REPORT OF BRIDGES
(BHAWANIPATNA TO KHARIAR)

CONSULTING
Engineers Group Ltd. Jaipur
E-12, Moji Colony, Malviya Nagar , Jaipur, Raj.
☎+ 91-141-2520899, Telefax +91-141-2521348
E-mail : ceg@cegindia.com URL : www.cegindia.com

INDEX

INTRODUCTION

INTRODUCTION

This report presents the design of bridges for Bhawanipatna to Khariar on SH-16. The total width of carriageway is 12.0 m with clear carriageway width of 11.0 m and crash barrier of 0.5 m on each side in minor bridges does not lie in village vicinity. Footpath of 1.5 m shall be provided on both sides for major bridges and minor bridges lies in the village vicinity. The bridge having 7.5 m width in good condition has been retained without any widening as per the discussion and decision with PIU.

There are total 21 Nos of existing bridges in this stretch. The existing and new proposal has been summarized on the next page.

From Ch. 27/600 to 29/400, there are total 6 nos. of bridges on Tel River. 3 Nos. of bridges are towards Bhawanipatna side w.r.t. Main Tel River bridge and 2 bridges are towards Khariar side.

As per the drawing received from Executive Engineer Bhawanipatna,HFL observed on 3rd July 2006 was around 1.5 m above bridge deck w.r.t. bridge at Ch. 27/850. The level difference between main bridge on Tel and the above bridge is about 6 m within a distance of around 125 m. As per highway alignment, it is not possible to keep the bridge at the same level. Due to submergence, difficulty in highway alignment , Detailed hydrology report, Review committee meeting and Inspection of Bhawanipatna-Khariar Road followed by Inspection note vide letter no. PIU(WB) 25/2006, it is decided to dismantle the existing bridge at Ch. 27/800 and at Ch. 27/850 and construction of new bridge of around 175 m clear vent way. It is proposed to construct 8 spans of 32.2 m PSC Girder and substructure resting on Pile foundations as per geotechnical and hydrology report. As per the discussion with PIU, raising of other minor bridges at Ch. 27/600, 28/900 and 29/400 has been suggested provided the parameter like SBC and reinforcement mentioned in working drawings based on detailed calculations are met.

Main Tel bridge has been kept as it is as per detailed inspection and performance of the bridge. For bridge at Ch. 29/400 besides raising, 2 additional spans of 9.2 m on both sides has been added. Initially 2 spans of 32.2 m was proposed for the above bridge.

Detailed analysis of existing bridges to be raised has been made on the basis of modern day loading and dimension detail drawing No. D-III/25/1-78-79 provided by PIU.

In final DPR, rehabilitation measures of bridge at Ch. 69/300 has been proposed as per the review committee inspection note vide letter no. PIU(WB) 25/2006, however as the NDT results are not satisfactory, it is suggested to do core cutting of slab during execution to know the residual strength of concrete.

Slab type superstructure of 10.0 m and RCC girder of 12.0 m, 14.0 m and 21.0 m has been taken from the MOST standard drawings.

Box type structure has also been taken from the MOST – standard drawings.

Substructure of the bridges has been designed as per the program based on latest IRC codes on standard excel sheet-moving live load using standard software STAAD-PRO.

Following live loads has been considered for 12.0 m overall width.

1. Single lane of Class 70-R wheeled+ Class A
2. Single lane of class 70-R Tracked + class A
3. Class-A, 3 lane

The structural drawing has been prepared in AUTOCAD.

Following changes has been done in Final DPR.

- Tel bridge and its approaches designs has been revised as per the actual HFL as per detailed hydrology report and waterway as per the Inspection note vide letter no. PIU(WB) 25/2006 and letter no. PIU (WB)/36/05/part/7227/22/2/07.
- Bridge at Ch. 69/300 reconstruction case in draft DPR has been revised to Rehabilitation case as per the Inspection note vide letter no. PIU(WB) 25/2006.

- Discrepancy in Soil report has been taken care as per the comments received on 30.12.06 and compliance on dated 17-01-07.
- Bed protection and length of apron has been incorporated as per discussion.
- Seismic analysis as per letter no. PIU(WB)43/2006/11973/dated 20th march 2007, has been done in our sample calculation for bridge at Ch. 29/500 in Berhampur-Rayagada road and are not critical for seismic zone II and as per our experience in this regard, hence in other designs it is ignored.
- Coherence in list of culverts in the drawing, report and scheduled of culverts has been maintained with proper design chainages as per letter no. PIU (WB)/36/05/part/7227/22/2/07.

Code of Reference

1. **IRC : 21 - 2000**
2. **IRC : 6 - 2004**
3. **IRC : 78 - 2000**
4. **IRC : 89 - 1997**
5. **IRC : 89 - 1997 (Part II)**
6. **SP : 13 – 2004**
7. **Specifications for Road and Bridge works (MOST Book)**
8. **MOST standard plans Culverts with or without Cushion**

Sl.No.	Location/ Chainage	Existing Span Arrangement	Existing Carriage way width	Type of Bridge	Recommended for	Proposed Spand Arrangement	Type of foundation	Type of Substructure	Type of Superstructure	Remarks
1	3/050	3 x 8.8	7.7	High Level	Repair/Rahabilitation	-	Open foundation	PCC wall type	RCC Solid Slab	Good Repair/ Rehabilitation required
2	4/450	4 x 9.9	6.7	Submersible	Repair/Rahabilitation	-	Open foundation	PCC wall type	RCC Solid Slab	Good Repair/ Rehabilitation required
3	8/600	1 x 7.3	7.2	Submersible	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge, existing submersible
4	10/500	1 x 7.4	7.2	Submersible	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge, existing submersible
5	13/750	2 x 7.2	7.3	Submersible	Reconsturction	1 x 14.6 RCC T - Beam Girder	Open foundation	RCC wall type	RCC T - Beam Girder	New bridge, existing in poor condition
6	17/120	1 x 7.55	3.1	Submersible	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge, existing in poor condition
7	21/000	(3 x 0.6) + (7 x 1.2)	6.7	Hume pipe Vented cause way	Reconsturction	1 x 12.6 RCC T - Beam Girder	Open foundation	RCC wall type	RCC T - Beam Girder	New bridge, existing causeway
8	27/600	7 x 9.2	6.85	Submersible	Repair/Rahabilitation	-	Open foundation	RCC Pier, RCC Pier Type Abutment	RCC Solid Slab	Good Repair/ Rehabilitation required
9	27/800	10 x 1.2	6.6	Hume pipe Vented cause way	Reconsturction	8 x 32.2 PSC Girder	Pile foundation	RCC wall type	PSC Girder	New bridge, existing submersible
10	27/850	9 x 9.2	7.2	Submersible	Reconsturction					
11	28/400	(2 x 9.9) + (1 x 24.37) + (1 x 34.9) + (10 x 40.85)	6.85	High Level	Nothing to do	-	Well Foundation	RCC Pier, RCC Pier Type Abutment	Balanced Cantilever Box Girder	Good Nothing to do
12	28/900	4 x 9.2	7	Submersible	Repair/Rahabilitation	-	Open foundation	RCC pier, Rcc pier type abutment	RCC Solid Slab	Good Repair/ Rehabilitation required
13	29/400	2 x 9.2	7.1	Submersible	Repair/Rahabilitation	(2 x 9.2) + (2 x 9.2) Additional Spans on each sides	Pile foundation	RCC wall type	PSC Girder	Good Repair/ Rehabilitation required
14	45/700	1 x 6.2	10.4	High Level	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge existing in poor condition
15	54/600	3 x 8.5	7.5	Submersible	Realigned	3 x 10.8 m RCC Solid Slab	Open foundation	RCC wall type	RCC Solid Slab	New bridge, existing in realignment

Sl.No.	Location/ Chainage	Existing Span Arrangement	Existing Carriage way width	Type of Bridge	Recommended for	Proposed Spand Arrangement	Type of foundation	Type of Substructure	Type of Superstructure	Remarks
16	58/900	3 x 6.8	8	Submersible	Realigned	1 x 21.6 RCC T - Beam Girder	Open foundation	RCC wall type	RCC T - Beam Girder	New bridge, existing in realignment
17	59/100	(7 x 32.7) + (1 x 7.6)	7.2	High Level	Repair/Rahabilitation	-	Well Foundation	RCC wall type	PSC Girder	Good Repair/ Rehabilitation required
18	59/400	5 x 4.0	7.8	High Level	Repair/Rahabilitation	-	Raft Foundation	RCC cell box		Good Repair/ Rehabilitation required
19	63/650	1 x 6.6	9	High Level	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge existing in poor condition
20	66/500	1 x 6.4	8.8	High Level	Reconsturction	1 x 8.0 RCC Box	Raft Foundation	RCC cell box		New bridge existing in poor condition
21	69/300	1 x 7.2	8.5	High Level	Repair/Rahabilitation	-	Open foundation	RR Stone Masonry wall type	RCC Solid Slab	Repair/ Rehabilitation required

BRIDGE AT CH:13+750

DESIGN OF SUBSTRUCTURE

DESIGN DATA

Formation Level	=	235.100 m
Ground Level	=	229.452 m
Lowest Water Level	=	225.824 m
Highest Flood Level	=	232.670 m
Founding Level	=	225.824 m
Thickness of bearing & pedestal	=	0.300 m
Width of abutment	=	12.000 m
Bouyancy factor	=	1.0
Safe Bearing Capacity	=	86.530 t/sqm
Dry density of earth	=	1.800 t/cum
Submerged density of earth	=	1.0 t/cum
Saturated density of earth	=	2.000 t/cum
Coefficient of base friction	=	0.7
Span (c/c of exp. joint)	=	14.600 m
Overall Width of deck slab	=	12.000 m
Width of carriageway	=	11.000 m
Width of crash barrier	=	0.500 m
Depth of Superstructure	=	1.400 m
Tkickness of wearing coat	=	0.056 m
Unit wt of concrete	=	2.400 t/m^3
no. of elastomeric bearing	=	4
size of elastomer brgs.(500 x 250	x	50 mm)
Grade of Concrete	-	M 25 25
Grade of Reinforcement	-	415 (HYSD)
Live Load	-	One Lane of 70R Wheeled + Class A
	-	3 lanes of Class A
Permissible Compressive stress in Concrete	-	850 t/m^2
Permissible Tensile stress in Steel	-	20400 t/m^2
Modular ratio, m	-	10
factor, k	-	0.294
Lever arm factor, j	-	0.902
Moment of Resistance	-	113 t/m^2
Thickness of returnwall		0.5 m

COEFFICIENT OF ACTIVE EARTH PRESSURE

AS PER COULOMB'S THEORY, COEFFICIENT OF ACTIVE EARTH PRESSURE IS

$$K_a = \frac{SIN^2(\alpha+\phi)}{SIN^2\alpha \, . \, SIN(\alpha-\delta)\left[1+\sqrt{\dfrac{SIN(\alpha+\varphi).SIN(\varphi-\iota)}{SIN(\alpha-\delta).SIN(\varphi+\iota)}}\right]^2}$$

WHERE

ϕ = ANGLE OF INTERNAL FRICTION OF EARTH

α = ANGLE OF INCLINATION OF BACK OF WALL

δ = ANGLE OF INTERNAL FRICTION BETWEEN WALL & EARTH

ι = ANGLE OF INCLINATION OF BACKFILL

HERE	ϕ =	30^o	=	0.524 Radian
	α =	90^o	=	1.571 Radian
	δ =	20^o	=	0.349 Radian
	ι =	0^o	=	0 Radian
			K_a =	0.2973

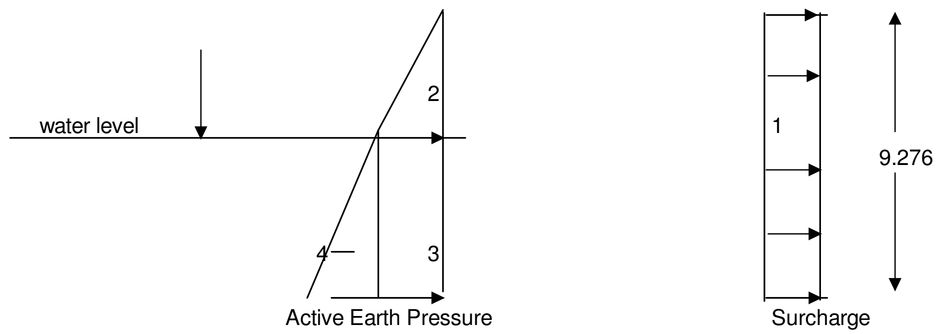
Therefore, Horizontal coefficient of Active earth pressure = **$K_aCOS\phi$ = K_{ha} = 0.2794**

HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 9.276 m

For DESIGN purpose, the height of abutment is considered as, say, = **9.280 m**

CALCULATION OF ACTIVE EARTH PRESSURE



DRY. condition

a) Service Condition

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	74.64	4.638	346.17
2	Dry Earth	0.5	9.276	4.66	259.62	3.896	1011.48
3		1.0	0.000	4.66	0.00	0.000	0.00
4	SubmgEarth	0.5	0.000	0.00	0.00	0.000	0.00
TOTAL					**334.26**		**1357.64**

b) Span Dislodge Condition

Net force = 334.26 - 74.64 = **259.62** t

Net moment = 1357.64 - 346.17 = **1011.48** tm

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	74.64	4.638	346.17
2	Dry Earth	0.5	2.430	1.36	19.80	7.867	155.73
3		1.0	6.846	1.36	111.55	3.423	381.82
4	SubmgEarth	0.5	6.846	1.91	78.56	2.282	179.28
TOTAL					**284.54**		**1063.01**

b) Span Dislodge Condition

Net force = 284.54 - 74.64 = **209.91** t

Net moment= 1063.01 - 346.17 = **716.84** tm

Forces & moments due to Abutment (Concrete) components

DRY Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment about toe
1	Toe Slab	1.0	2.000	12.00	0.500	2.40	28.80	1.000	28.80
2		0.5	2.000	12.00	0.700	2.40	20.16	1.333	26.88
3	Heel Slab	1.0	3.400	12.00	0.500	2.40	48.96	4.900	239.90
4		0.5	3.400	12.00	0.700	2.40	34.27	4.333	148.51
5	Stem Wall	1.0	1.200	12.00	7.220	2.40	249.52	2.600	648.76
6		0.5	0.000	12.00	7.220	2.40	0.00	2.000	0.00
7	stem rect	1.0	1.200	12.00	1.200	2.40	41.47	2.600	107.83
10	Cap	1.0	1.200	12.00	0.300	2.40	10.37	2.600	26.96
11	Dirt Wall	1.0	0.300	12.00	1.406	2.40	12.15	3.050	37.05
8	Retutnwall	0.5	3.400	1.00	0.700	2.40	2.86	5.467	15.61
8a		1.0	3.400	1.00	8.776	2.40	71.61	4.900	350.90
TOTAL							**520.17**		**1631.20**

H.F.L. Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment ,@ Toe
1	Toe Slab	1.0	2.000	12.00	0.500	1.40	16.80	1.000	16.80
2		0.5	2.000	12.00	0.700	1.40	11.76	1.333	15.68
3	Heel Slab	1.0	3.400	12.00	0.500	1.40	28.56	4.900	139.94
4		0.5	3.400	12.00	0.700	1.40	19.99	4.333	86.63
5	Stem Wall	1.0	1.200	12.00	5.646	1.40	113.82	2.600	295.94
6		0.5	0.000	12.00	5.646	1.40	0.00	2.000	0.00
5	Stem Wall	1.0	1.200	12.00	0.374	2.40	12.93	2.600	33.61
6		0.5	0.000	12.00	0.374	2.40	0.00	2.000	0.00
7	stem rect	1.0	1.200	12.00	1.200	1.40	24.19	2.600	62.90
10	Cap	1.0	1.200	12.00	0.30	2.40	10.37	2.600	26.96
11	Dirt Wall	1.0	0.300	12.00	1.41	2.40	12.15	3.050	37.05
8	Returnwall	0.5	3.400	1.00	0.70	1.40	1.67	5.467	9.11
8c		1.0	3.400	1.00	5.65	1.40	26.87	4.900	131.69
8d		1.0	3.400	1.00	2.43	2.40	19.83	4.900	97.16
TOTAL							**298.94**		**953.47**

Forces & moments due to Earth and LL surcharge

DRY Case : Self weight of Earth

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@ Toe
8	DRY EARTH	0.5	3.400	11.00	0.700	1.8	23.56	5.467	128.81
8a		1.0	3.400	11.00	8.076	1.800	543.68	4.900	2664.01
TOTAL							**567.24**		**2792.82**

H.F.L Case :

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@Toe
8	SATURATED SOIL	0.5	3.400	11.000	0.7	1.000	13.09	5.467	71.56
8c		1.0	3.400	11.000	5.646	1.000	211.16	4.9	1034.69
8d	DRY	1.0	3.400	11.000	2.43	1.800	163.59	4.900	801.58
TOTAL							**387.84**		**1907.82**

L.L.SURCHARGE

Element No	Component	Area Factor	Length	Width	Height	Density	Weight	C.G.from toe	Moment `@ Toe
12	DRY EARTH	0.00	3.400	11.00	1.20	1.80	**0.00**	4.900	**0.00**

SUMMARY OF FORCES AND MOMENTS:

LOAD CASE	Case. L.W.L.		Case. H.F.L.		
	Service Cond.	Span dislodged	Service Cond.	Span dislodged	
Vertical load from superstructure including LL	**211.04**	0.00	**211.04**	0.00	
Vertical load from substructure (b)	1087.41	1087.41	686.78	686.78	
Total Vertical Load V = (a) + (b)	1298.45	1087.41	897.81	686.78	
Total Horizontal Force H =	**350.26**	259.62	**300.54**	209.91	
Moment @ toe due to (a)	**548.69**	0.00	**548.69**	0.00	
Moment @ toe due to (b)	4424.02	4424.02	2861.29	2861.29	
Total Moment @ toe (M)	4972.72	4424.02	3409.98	2861.29	
Dist. of C.G. of V from toe Z = M/V	3.830	4.07	3.80	4.17	
eccentricity (e = Z -b/2)	0.530	0.768	0.498	0.866	
Relieving Moment @ c/l base (M1)	**687.85**	**835.57**	**447.20**	**594.93**	
overturning moment due to					
Horz. braking force	**130.72**	0.00	**130.72**	0.00	
Earth Pressure	1357.64	1011.48	1063.01	716.84	
Total overturning Moment (M2)	**1488.36**	**1011.48**	**1193.73**	**716.84**	
Net moment (M2-M1) = M_L	**800.52**	**175.91**	**746.53**	**121.91**	
Factor of Safety					
Against overturning (M / M2)	**3.34**	**4.37**	**2.86**	**3.99**	Safe against overturning
Against sliding (µ x V / H)	**2.595**	**2.932**	**2.091**	**2.290**	Safe against sliding

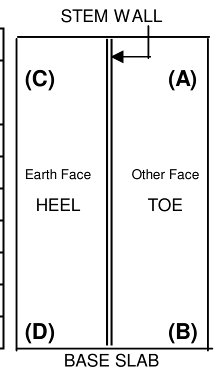
I.R.C 78-2000:cl 706.3.4

Area of base (A) = 6.600 x 12.00 = 79.20 m^2

Z_L = 87.12 m^3
Z_T = 158.40 m^3

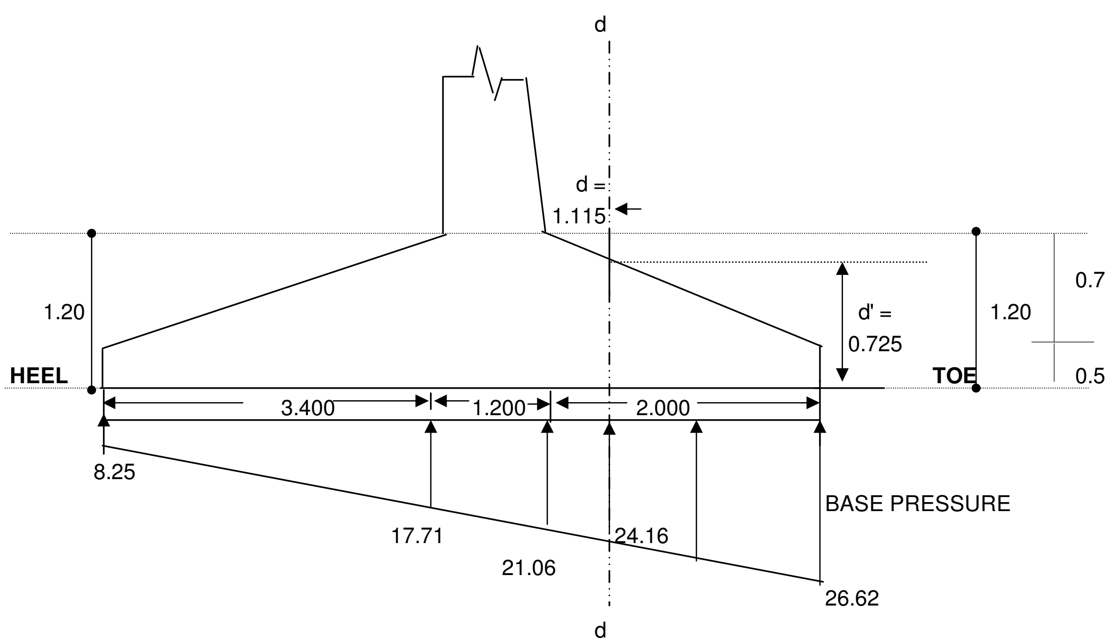
CHECK FOR BASE PRESSURE:

Base Pressure		LWL CASE		HFL CASE	
		Service Cond.	Span dislodged	Service Cond.	Span dislodged
	P/A	16.39	13.73	11.34	8.67
	M_L/Z_L	9.19	2.02	8.57	1.40
	M_T/Z_T	1.04	0.00	1.04	0.00
(A)	$(P/A + M_L/Z_L + M_T/Z_T)$	**26.62**	**15.75**	**20.95**	**10.07**
(B)	$(P/A + M_L/Z_L - M_T/Z_T)$	24.54	15.75	18.86	10.07
(C)	$(P/A - M_L/Z_L + M_T/Z_T)$	8.25	11.71	3.81	7.27
(D)	$(P/A - M_L/Z_L - M_T/Z_T)$	**6.165**	**11.71**	**1.73**	**7.27**



Max.Base Pressure = **26.62** **t/m^2** **<** **86.53** **Hence O.K.**
Min. Base Pressure = **1.73** **t/m^2** **>** **0** **Hence O.K.**

DESIGN OF TOE SLAB



BENDING MOMENT AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward Loads	1	1.0	2.400	1.000	2.400
	2	0.5	1.680	0.667	1.120
Upward Base pressure	Rect.	1.0	-42.111	1.000	-42.111
	Trian.	0.5	-5.569	1.333	-7.425
TOTAL			-43.600		-46.016

Bending Moment at face of stem 46.016 tm/m

Effective depth required 0.639 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 2243 mm^2

Minimum steel required 1673 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 669 mm^2/m
mainsteel 2243 mm^2 12 φ , @ 150 C/C
753.9822

Hence provide , 25 φ , @ 175 C/C
0 φ , @ 175 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 250 mm^2/m
provide 12 φ , @ 150 C/C
753.9822

Check for Shear

SHEAR FORCE AT "d" FROM FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward loads	1	1.0	1.062	0.443	0.470
	2	0.5	0.743	0.295	0.219
Upward base pressure	Rect.	1.0	-21.382	0.443	-9.461
	Trian.	0.5	-1.090	0.590	-0.643
TOTAL			-20.667		-9.416

Effective depth (d') at distence d 0.725 m

Shear force at critical section 20.7 t

Bending Moment at critical section 9.42 tm

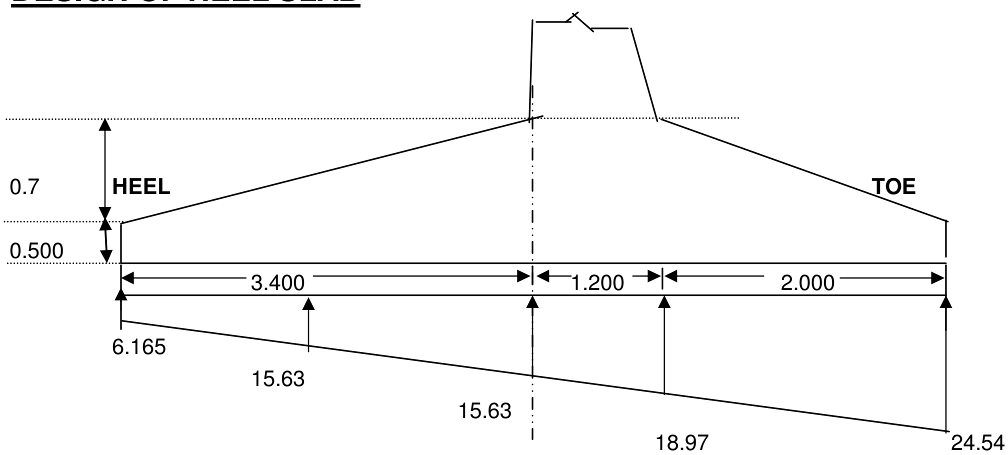
tan β = 0.36

Net shear force S-M*tanβ/d' **15.96** t

Hence, Shear stress 22.02 t/m^2

% of reinforcement 0.39
Permissible shear stress 27.92 t/m^2 **Hence O.K.**

DESIGN OF HEEL SLAB



BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	moment
Downward Loads.	3	1	4.080	1.700	6.936
	4	0.5	2.856	1.133	3.237
	8	0.5	4.284	2.267	9.710
	8a	1	49.425	1.700	84.023
		0.5	4.284	2.267	4.855
		1	6.206	1.700	10.550
Upward base pressure	Rect.	1	-20.960	1.700	-35.631
	Trian.	0.5	-16.094	1.133	-18.240
TOTAL			34.081		65.439

Bending Moment at face of stem **65.439** tm

Effective depth required 0.762 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3191.65 mm^2
Minimum steel 1672.50 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 669.00 mm^2/m
mainsteel 3191.65 mm^2

12 ϕ , @ 150 C/C
753.9822

Hence provide , 25 ϕ , @ 150 C/C
0 ϕ , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 250 mm^2/m
provide 12 ϕ , @ 150 C/C
753.9822

Check for Shear

(Critical section at face of stem)

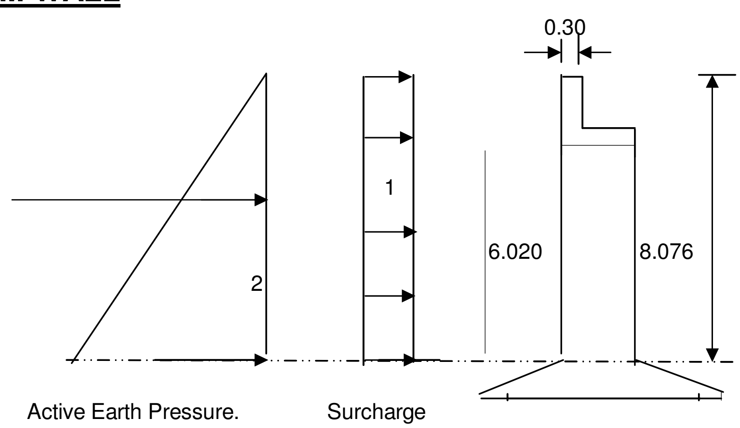
Shear force at face of stem 34.08 t

tan β 0.206
Bending moment at face of stem 65.439 tm
Net shear force S-M*tanβ /d 22.00 t

Hence, Shear stress 19.73 t/m^2
% of reinforcement 0.29

Permissible shear stress 24.63 t/m^2 **Hence O.K.**

DESIGN OF STEM WALL



SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT

R.L. OF SECTION = 227.024 m

DRY Condition

a) Service Condition

Element No.	Area factor	Height of E.P. diagram	Earth Pressure	Force	L.A.	Moment tm
1	1	7.726	0.603	55.9	3.863	216.13
2	0.5	7.726	3.885	180.1	3.245	584.44
HORIZONTAL FORCE				16.00	6.620	105.92
TOTAL				252.06		**906.488**

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	208.051	+	12.15	+	10.37	+	211.04
	=	441.604 t						
Longitudinal Moment	=	906.488 t-m						
Transverse Moment	=	164.933 t-m						

b) Span Dislodged Condition

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	208.051	+	12.15	+	10.37	+	0.00
	=	230.567 t						
Longitudinal Moment	=	800.568 t-m						
Transverse Moment	=	0.000 t-m						

H.F.L. Condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.603	55.95	3.863	216.13
2	Dry Earth	0.5	2.024	1.13	13.73	6.321	86.81
3		1.0	5.646	1.13	76.62	2.823	216.31
4	SubmgEarth	0.5	5.646	1.58	53.44	1.882	100.57
TOTAL					**199.74**		**619.815**

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	126.749	+	12.15	+	10.37	+	211.04
	=	360.301 t						
Longitudinal Moment	=	725.735 t-m						
Transverse Moment	=	164.933 t-m						

b) Span Dislodged Condition

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	126.749	+	12.15	+	10.37	+	0.00
	=	149.265 t						
Longitudinal Moment	=	619.815 t-m						
Transverse Moment	=	0.000 t-m						

Cross Sectional area 1.2 m^2

For horizontal reinforcement area of steel required for the stem at the section/metre = 173.9186 mm^2

Providing **12** ,@ 650.29 c/c say, **150** C/C as horizontal reinforcement

Movement of Deck:

Total Length of Bridge = 14.6 m

= 0.0005 x 7300 = 3.65 mm

Longitudinal Force :

Size of bearing = 500 x 250 x 50 mm

Strain in bearing = $\frac{3.650}{50}$ = 0.073

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.073 x 1.0 x 500 x 250 = 9125 N

= 0.930 t

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

= 0.930 x 4 x 1.05 = 3.907 t

Refer IRC : 6 clause 214.5.1.5;

10 % increase for variation in movement of span

Total shear force = 1.1 x 3.907 = 4.297 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 50) = 10.000 t

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

For class A 3 lane F_h = 0.2 x 50 + 0.05 x 50 = 12.5 t

For class 70R wheeled +class A 1 lane

F_h = 0.2 x 100 + 0.05 x 50 = 22.5 t

For class A 3 lane

Longitudinal horizontal force = $\frac{12.500}{2}$ + 4.297 = 10.55 t Say, 11.0 t

For class 70R wheeled +class A 1 lane

Longitudinal horizontal force = $\frac{22.500}{2}$ + 4.297 = 15.5 t Say, 16.0 t

Span dislodged condition

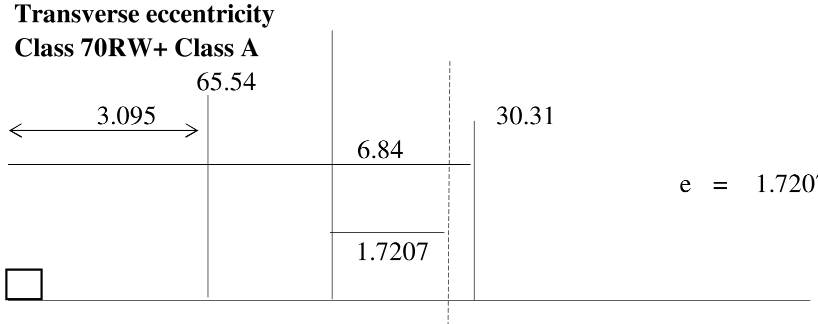
Longitudinal horizontal force = $\frac{0.000}{2}$ + 4.297 = 4.297 t Say, 5.0 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 3 lane	11.00
70R+class A 1lane	16.00
Span dislodged condition	0.00

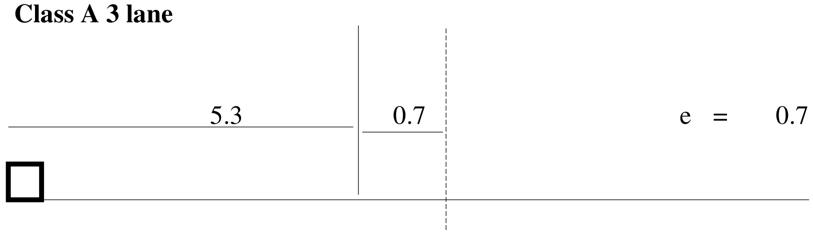
Transverse eccentricity

Class 70RW+ Class A



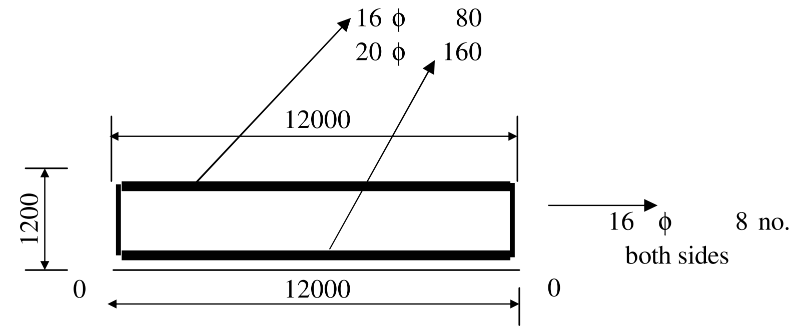
e = 1.7207

Class A 3 lane



Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

	P	M_L	M_T
Dead Load Reaction:			
	90.73	t	
SIDL			
	24.46	t	
L.L			
70R + Class A	95.85	t	164.93
Class A 3 Lane	90.93	t	63.65
	211.04		164.93



PLAN : ABUTMENT SHAFT

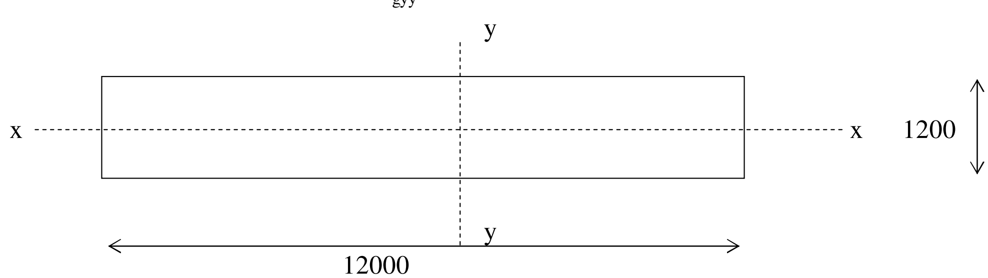
% Reinforcement provided = 69567.428 mm^2

Summary of Loads at Abutment Shaft bottom :

	P	M_L	M_T
1 DRY condition			
With L.L	0.00	0.00	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}	=	1.728E+12	mm^4
Gross M.O.I of section	I_{gyy}	=	1.728E+14	mm^4



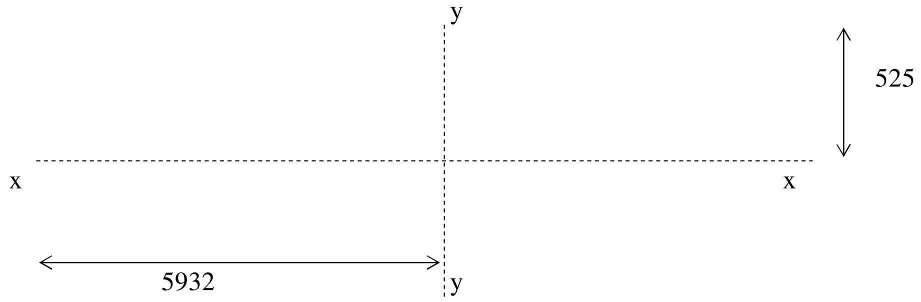
Abutment section

Transformed sectional properties of section:
Adopting

Modular ratio	m	=	10	
Cover			70	68
Dia of Bars		=	20	16
No of bars in tension face (longer)		=	160	
No of bars in compression face		=		80
No of bars in shorter direction		=		8
Total bars in section		=	256	

Steel Area A_s = 69567 mm^2

% of Steel = 0.4831 %



A_{sx} = 50265 mm^2

A_{sy} = 1608.5 mm^2

Area of concrete $A_c = A_g - A_s$ = 14330433 mm^2

C.G of Steel placed on longer face = 530 mm

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

Transformed Area of Section A_{tfm} = 15026107 mm^2

Transformed $M.I_{txx} = I_{gxx} + 2\left[m - 1\right] A_{sx}\, ax^2$

= 1.98215E+12 mm^4

$Z_{xx} = \dfrac{M.Itxx}{d/2}$ = 3.304E+09 mm^3

Transformed $M.I_{tyy} = I_{gyy} + 2\left[m - 1\right] A_{sy}\, ay^2$

= 1.73819E+14 mm^4

$Z_{yy} = \dfrac{M.I_{tyy}}{d/2}$ = 2.897E+10 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.728E+12 mm^4

Area of section = 14400000 mm^2

r = 346.41016 mm

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

Abutment shaft height L = 9.426 m

Effective length L_{eff} = 11.311 m

Slenderness ratio = 32.653 < **50**

Type of member = 1

1 Short Column

Stress reduction coefficient (IRC:21-2000 cl: 306.4.2,3) 2 Long Column

β = 1

Permissible stresses **:** *concrete*

σ_{cbc} = 8.3333 N/mm^2

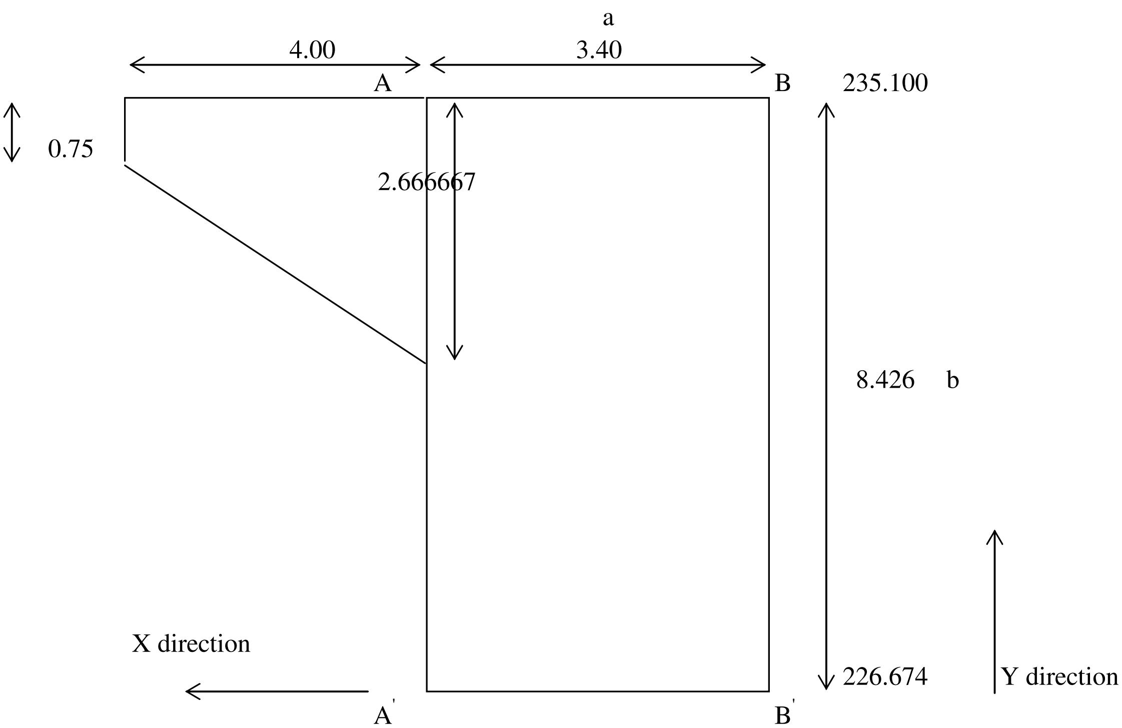
σ_{co} = 6.25 N/mm^2

Tensile stress = 0.61 N/mm^2

Permissible stresses : *Steel*

σ_{st} = 200 N/mm^2

S.No	Item		DRY Case	
	Loads and Moments		**With L.L**	
1	P		441.60	t
2	M_L		906.49	t-m
3	M_T		164.93	t-m
Actual(calculated) Stresses				
4	$\sigma_{co,cal}$	P/A_{tfm}	0.293890878	
5	$\sigma_{cbc,cal}$	M_L/Z_{xx}	2.743949739	
6	$\sigma_{cbc,cal}$	M_T/Z_{yy}	0.056932836	
7	$\sigma_{cbc,cal}$	= 5 + 6	2.800882575	
Permissible Stresses				
8	σ_{cbc}		8.3333333	
9	σ_{co}		6.25	
Check for Minimum steel area	**mm^2**			
10	Conc.Area Required for directstress			
		(1)/(9)	706565.72	
11	0.8% of area required		5652.5257	
12	0.3% of A_g		43200	
13	Governing steel	mm^2	43200	
14	Provided Steel area	mm^2	69567.428	
Check for safety of section				
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$		0.3831284 < 1	
Check for Cracked /Uncracked section				
	$\sigma_{co,cal} - \sigma_{cbc,cal}$		-2.506992	
	Permissible Basic tensile stress in concrete		-0.61	
	Section to be designed as		**Cracked**	



Width of Solid return wall (a)	=	3.40	
Width of Cantilever return wall	=	4.00	
Avg Height of Solid return wall (b)	=	8.426	
Height of Cantilever return at Tip	=	0.75	
Height of Cantilever return at Root	=	2.666667	
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	=	0.333333	
j	=	0.888889	
R	=	151.1111	t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.4035129	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 = 0.4035129 β_1 = 0.416413

β_2 = 0.451376

Live Load Surcharge:

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

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

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

$$\sigma_{bmax} = \frac{0.4164128 \times 0.603504 \times 71.00}{0.25} = 71.36859 \text{ t/m}^2$$

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 $= 71.368586 \times 0.041667 = 2.973691$ t-m/m

$$\sigma_{amax} = \frac{0.4513762 \times 0.603504 \times 71.00}{0.25} = 77.36094 \text{ t/m}^2$$

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 $= 77.360938 \times 0.041667 = 3.223372$ t-m/m

Case (2) For Triangular loading due to earth pressure

a/b = 0.4035129 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 = 0.4035129 β_1 = 0.23846

β_2 = 0.159861

Earth pressure:

$q = 0.2794 \times 1.8 \times 8.426 = 4.237604$ t/m^2

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

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

$$\sigma_{bmax} = \frac{0.23846 \times 4.237604 \times 71.00}{0.25} = 286.9715 \text{ t/m}^2$$

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 = 286.97153 x 0.041667 = **11.95715** t-m/m

$$\sigma_{amax} = \frac{0.1598614 \times 4.237604 \times 71.00}{0.25} = 192.3831 \text{ t/m}^2$$

For 1000 mm of width

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

$= 0.041667$ m^3

Hence Moment /m width along X direction

M_X /m width = 192.38306 x 0.041667 = **8.015961** t-m/m

Total Moment in Solid Return /m height = **11.23933** t-m/m

Along X-direction

Total Moment in Solid Return /m width = **14.93084** t-m/m

Along Y-direction

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A'

$$\begin{aligned} M = \; & 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 \\ & + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 \\ & + 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times dx \times (4.00 - X) \\ & + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times (4.00 - X) \end{aligned}$$

= 3.621024 + 1.13157 + 0.11176 x 21.33333 + 0.653796 x 10.6667

= **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 2.666667 x 0.179776 = 72.4431 t-m

$$A_{st} = \frac{14.11063 \times 10^6}{20400 \times 0.888889 \times 0.424} = 1835.28 \text{ mm}^2$$

A_{st}/m = 688.231 mm^2/m

Provide 12 mm dia @ 150 mm c/c providing 753.9822 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 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 5.40
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 5.40
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (7.40 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (7.40 - X)

= 9.776765 + 3.055239 + 0.11176 x 93.86667 + 0.653796 x 37.8667

= **48.07962** t-m

Moment in Solid Return /m height = 11.23933 + $\frac{48.07962}{8.426}$ = 16.9454 t-m/m

Moment in Solid Return /m width = = 14.9308 t-m/m

Design of face B-B'

Moment in Solid Return /m height = **16.9454** 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 x 0.1764 = 26.656 t-m

A_{st} = $\frac{16.94544 \times 10^6}{20400 \times 0.888889 \times 0.42}$ = 2224.98 mm^2

A_{st}/m = 2224.98 mm^2/m

Provide 20 mm dia @ 150 mm c/c providing 2094.395 mm^2 on earth face.

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

Along Horizantal direction.

Design of face A'-B'

Moment in Solid Return /m width = **14.9308** 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 x 0.1936 = 29.2551 t-m

A_{st} = $\frac{14.93084 \times 10^6}{20400 \times 0.888889 \times 0.44}$ = 1871.35 mm^2

A_{st}/m = 1871.35 mm^2/m

Provide 20 mm dia @ 160 mm c/c providing 1963.495 mm^2 on earth face.

Provide 12 mm dia @ 160 mm c/c providing 706.8583 mm^2 on other face.

Along Vertical direction.

1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure	K_a	=	0.279384
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		=	0.5
Avg. cover to reinforcement. (FOR 2-3 LAYERS)		=	0.150 m

H (From formation level) = 8.076 m

DESIGN FOR BENDING MOMENT

S.NO.	UNITS	1
H (FROM TOP)	m	8.076
WIDTH OF RETURN AT THIS LEVEL	m	3.400
FORCE DUE TO EARTH PRESSURE	t	92.464
MOMENT DUE TO EARTH PRESSURE	tm	313.631
FORCE DUE TO L.L. SURCHARGE	t	27.478
MOMENT DUE TO L.L. SURCHARGE	tm	110.957
TOTAL MOMENT	tm	424.588
DESIGN MOMENT	tm	424.588
REQUIRED EFFECTIVE DEPTH	m	2.650
EFF. DEPTH AVAILABLE	m	3.250
AREA OF STEEL REQUIRED	cm^2	72.036
DIAMETER OF BAR PROVIDED	mm	32
TOTAL NO. OF BARS	no.	12
AREA OF STEEL PROVIDED	cm^2	96.51
		O.K.
		34.0
CHECK FOR SHEAR STRESS		
SHEAR FORCE	t	124.942
SHEAR STRESS	t/m^2	76.888
Ast/bd x 100	%	0.443
PERMISSIBLE SHEAR STRESS	MPa	0.29
PERMISSIBLE SHEAR STRESS	t/m^2	29.77
Asv/sv	cm^2/m	7.107
k_1	...	0.5
k_2	...	1.0
PERMISSIBLE SHEAR STRESS	t/m^2	

10 Tor 2-legged 1

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	=	300	mm			
Assuming a cap thickness of	=	300	mm			
Volume of Abutment cap	=	0.3	x	1.20	x	12
	=	4.32	m^3			
Quantity of steel	=	1 %	of volume			
	=	$\frac{1}{100}$	x	4.32		
	=	0.0432	m^3			
Quantity of steel to be provided at top	=	0.0216	m^3			
Quantity of steel to be provided at bottom	=	0.0216	m^3			

Top & bottom face:

Quantity of steel to be provided in Longitudinal dir			=	0.0108	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					
	$\frac{0.0108}{11.9}$		=	907.563	mm^2

Provide **9** nos of bars **12** mm dia at top & bottom face.

= 1017.88 mm^2

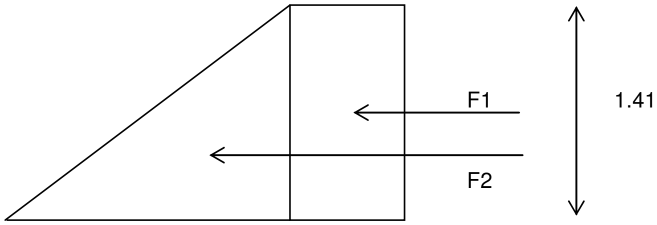
Transverse steel:

Quantity of steel to be provided in Longitudinal dir			=	0.0108	m^3
Assuming a clear cover of			=	50	mm
Assuming a dia of bar			=	12	mm
Length of bar	1.20 -	0.100	=	1.1	m
Volume of each stirrup			=	0.00012	m^3
no of stirrups required for m/length			=	8	nos
Required Spacing			=	$\frac{1000}{8}$	
			=	125	mm

Provide 12 mm dia bar 125 mm c/c stirrups throught in length of abutment cap.

904.779 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	=	0.6706	t/m^2
F1	=	0.67056	x	12.00	x	1.41	=	11.31369	t
Intensity for triangular portion	=	0.2794	x	2.00	x	1.41	=	0.7857	t/m^2
F2	=	0.785673	x	12.00	x	1.41	=	6.627936	t
M1	=	11.31369	x	0.70			=	7.953523	t-m
M2	=	6.627936	x	0.59052			=	3.913929	t-m
M1+ M2							=	**11.86745**	t-m
Total moment at base of dirt wall /m length							=	0.988954	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 $= \dfrac{0.988954}{1.51 \times 1000} = 80.92815$ mm

Ast req $= \dfrac{0.988954}{200 \times 0.889 \times 244} = 227.9579$ mm^2/m

Minimum steel $= \dfrac{0.12}{100} \times 300 \times 1000 = 360$ mm^2/m

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	0.988954	=	0.296686	t-m/m

Ast req $= \dfrac{0.296686}{200 \times 0.889 \times 238} = 70.11142$ mm^2/m

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.

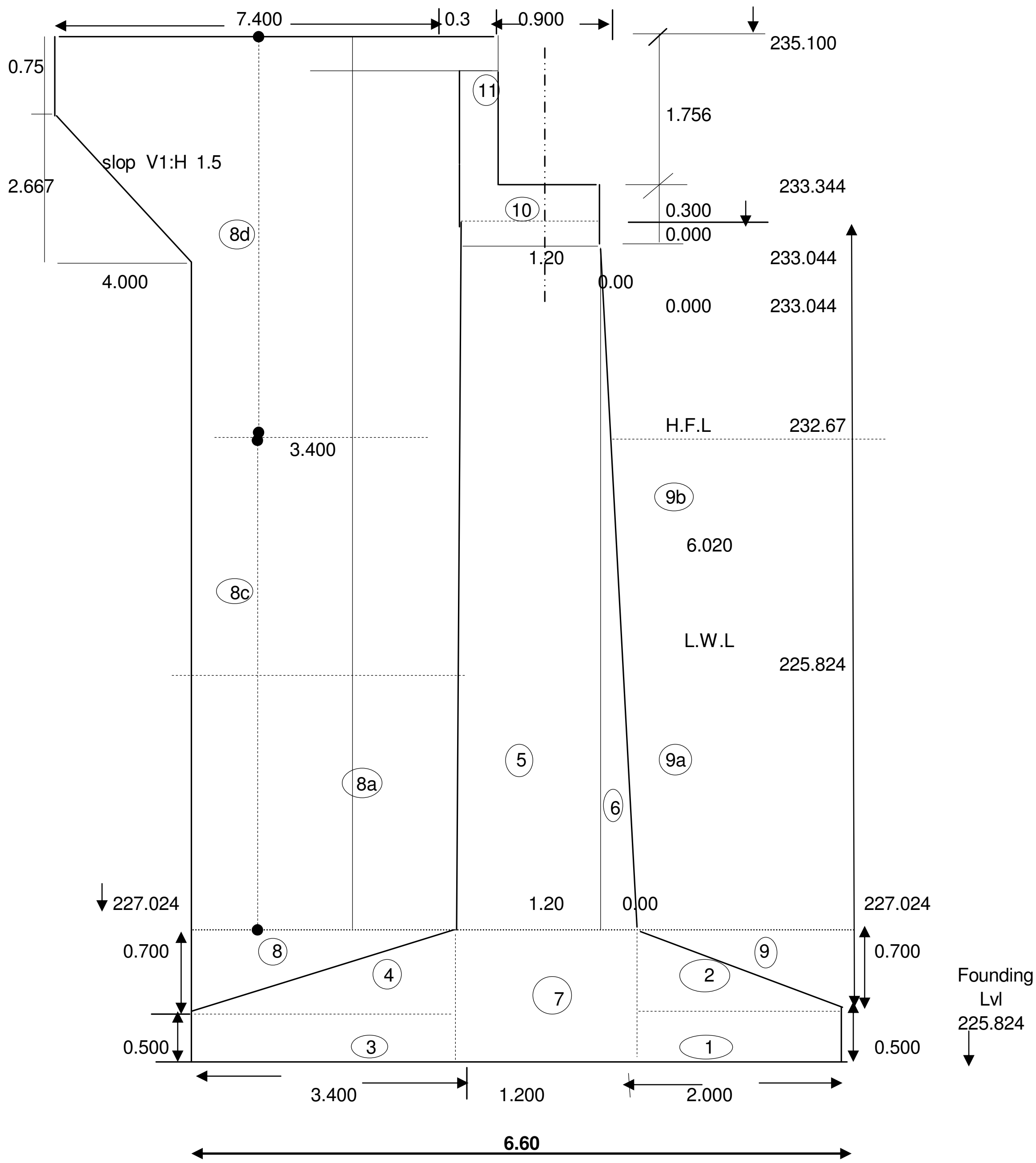
392.6991 mm^2/m

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


7.400
0.3
0.900
235.100
0.75
11
1.756
slop V1:H 1.5
2.667
233.344
10
0.300
8d
0.000
1.20
233.044
4.000
0.00
0.000
233.044
H.F.L
232.67
3.400
9b
6.020
8c
L.W.L
225.824
5
9a
8a
6
227.024
1.20
0.00
227.024
0.700
8
9
0.700
4
2
Founding
Lvl
225.824
7
0.500
3
1
0.500
3.400
1.200
2.000
6.60

Wind Force Calculation

Span c/c of Abutment		14.60	m

Section details

Deck level at abutment		235.100	m
Wearing coat + Girder depth + Bearing thickness + Pedestal ht.		1.552	m
Abutment cap level		233.548	m
Foundation level adopted		225.900	m
Foundation top level		227.100	
Average bed level		229.420	m
Height of dirt wall		1.552	m
Thickness of abutment cap		0.300	m
Ht. of abutment from foundation base to top of cap	233.548-225.900	7.648	m
Thickness of foundation slab at junction		1.200	m
Thickness of foundation slab at edge		0.500	m
Net ht. of abutment from top of fnd. slab to bottom of cap	7.648-1.200-0.300	6.148	m
Ht. of back fill from bottom of foundation	235.10-225.90	9.20	m
Surcharge height as per IRC:78-2000, clause 710.4.4		1.20	m
Thickness of dirt wall (as per clause 710.6.4 of IRC:78-2000)		0.300	m
Width of abutment cap	0.430+0.320+0.300	1.200	m
Height of return wall above top of foundation slab	235.100-225.900-1.200	8.000	m

Wind Forces

On live load

Intensity of wind force on moving live load	300	kg/m
Exposed length of live load	14.60	m
Horizontal wind force	43.8	kN
Height of action of this force above deck	1.5	m
Height of action of this force from top of foundation	9.500	m
Height of action of this force from bottom of foundation	10.700	m

On deck

Girder depth + crash barrier	2.45	m
Exposed area of deck per span	17.89	m2
Average height above foundation top	9.325	m
Wind pressure at this height	88.00	kg/m2
Wind force on deck per span	15.74	kN

Total on superstructure

Wind force on loaded superstructure			29.77	kN
Min. force on loaded superstructure @	450	kg/m	32.85	kN
Hence applicable wind force on loaded superstructure			32.85	kN
Height of combined c.g. of the forces above foundation top			9.454	m
Height of combined c.g. of the forces at foundation bottom			10.654	m
Level of action of this force			236.554	m
Wind force on unloaded superstructure			15.74	kN
Min. force on unloaded superstructure @	240	kg/m2	42.92	kN
Hence applicable wind force on unloaded superstructure			42.92	kN
Height of c.g. of the forces above foundation top			9.32	m
Height of c.g. of the forces at foundation bottom			10.53	m
Level of action of this force			236.43	m

On substructure

Exposed area of abutment & cap			4.95	m2
Average height above top of foundation			5.584	m
Wind pressure at this height			71.00	kg/m2
Wind force on substructure			3.52	kN
Average height from bottom of foundation			5.584	m
Average height from bottom of abutment shaft			4.384	
Moment at the bottom of foundation	Total	Transverse	Longitudinal	
Loaded condition	369.61	369.61	0.00	kN.m
Unloaded condition	471.41	471.41	0.00	kN.m
Transverse moment due to wind forces are say 47 t-m.				
Moment at the bottom of abutment shaft bottom				
Loaded condition	325.97	325.97		
Unloaded condition	415.69	415.69		
Transverse moment due to wind forces are say 42 t-m.				

```
INPUT FILE: 70r.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.3 0 0;3 14.3 0 0;4 14.6 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    22. LOAD GENERATION 175
    23. TYPE 1 -13.4 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 74
    26. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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

      2   74      0.00     34.46      0.00      0.00      0.00      0.00
      3   74      0.00     65.54      0.00      0.00      0.00      0.00
    27. FINISH
```

```
INPUT FILE: CLASS A.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.3 0 0;3 14.3 0 0;4 14.6 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    22. LOAD GENERATION 202
    23. TYPE 1 -18.8 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 74
    26. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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

     2   74      0.00     30.31      0.00      0.00      0.00      0.00
     3   74      0.00     19.69      0.00      0.00      0.00      0.00

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

    27. FINISH
```

```
Abutment SHAFT ..DRY NORMAL

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

along width-compression face- no of bar:  80   tension face- no of bar:
160
Dia (mm)                                   16
20
Cover (cm)                                7.50
10.5
along depth-compression face- no of bar:   8   tension face- no of bar:
8
Dia (mm)                                   16
16
Cover  (cm)                               7.50
7.5

Modular Ratio : Compression  =      10.0
Modular Ratio : Tension      =      10.0
Allowable Concrete Stress    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   441.604 T
Mxx                          =   906.488 Tm
Myy                          =   164.933 Tm


Intercept of Neutral axis : X axis : = 192.893 m
                          : y axis : =    .330 m

          Concrete Stress Governs Design

Stress in Concrete due to Loads =   57.84 Kg/cm^2
Stress in Steel    due to Loads = 1374.71 Kg/cm^2
Percentage of Steel             =     .48 %


Abutment SHAFT ..DRY SPAN DISLODGED

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   230.567 T
Mxx                          =   800.568 Tm
Myy                          =      .000 Tm

Intercept of Neutral axis : X axis : = ******* m
                          : y axis : =    .294 m
```

```
             Steel Stress Governs Design

Stress in Concrete due to Loads =   49.21 Kg/cm^2
Stress in Steel    due to Loads = 1341.50 Kg/cm^2
Percentage of Steel             =     .48 %



Abutment SHAFT ..HFL NORMAL

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   360.301 T
Mxx                          =   725.735 Tm
Myy                          =   164.933 Tm


Intercept of Neutral axis : X axis : = 156.141 m
                          : y axis : =    .334 m

          Concrete Stress Governs Design

Stress in Concrete due to Loads =   46.67 Kg/cm^2
Stress in Steel    due to Loads = 1098.35 Kg/cm^2
Percentage of Steel             =     .48 %

Abutment SHAFT ..HFL SPAN DISLODGED

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   149.265 T
Mxx                          =   619.815 Tm
Myy                          =      .000 Tm

Intercept of Neutral axis : X axis : = ******* m
                          : y axis : =    .288 m

             Steel Stress Governs Design

Stress in Concrete due to Loads =   37.99 Kg/cm^2
Stress in Steel    due to Loads = 1064.81 Kg/cm^2
Percentage of Steel             =     .48 %
```

DESIGN OF SUPERSTRUCTURE

For Design of Superstructure of RCC Girder 14.0 m span refer MOST STANDARD Drawing titled "STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and Slab Superstructure)" Drg. No. SD/250 to SD/256.

BRIDGE AT CH:21+000

DESIGN OF SUBSTRUCTURE

DESIGN DATA

Formation Level	=	221.200	m
Ground Level	=	217.178	m
Lowest Water Level	=	210.678	m
Highest Flood Level	=	218.807	m
Founding Level	=	210.678	m
Thickness of bearing & pedestal	=	0.300	m
Width of abutment	=	12.000	m
Bouyancy factor	=	1.0	
Safe Bearing Capacity	=	45.300	t/sqm
Dry density of earth	=	1.800	t/cum
Submerged density of earth	=	1.0	t/cum
Saturated density of earth	=	2.000	t/cum
Coefficient of base friction	=	0.5	
Span (c/c of exp. joint)	=	12.600	m
Overall Width of deck slab	=	12.000	m
Width of carriageway	=	11.000	m
Width of crash barrier	=	0.500	m
Depth of Superstructure	=	1.200	m
Tkickness of wearing coat	=	0.056	m
Unit wt of concrete	=	2.400	t/m^3
no. of elastomeric bearing	=	4	
size of elastomer brgs.(500 x 250	x	50	mm)
Grade of Concrete	-	M 25	25
Grade of Reinforcement	-	415 (HYSD)	
Live Load	-	One Lane of 70R Wheeled + Class A	
	-	3 lanes of Class A	
Permissible Compressive stress in Concrete	-	850	t/m^2
Permissible Tensile stress in Steel	-	20400	t/m^2
Modular ratio, m	-	10	
factor, k	-	0.294	
Lever arm factor, j	-	0.902	
Moment of Resistance	-	113	t/m^2
Thickness of returnwall		0.5	m

COEFFICIENT OF ACTIVE EARTH PRESSURE

AS PER COULOMB'S THEORY, COEFFICIENT OF ACTIVE EARTH PRESSURE IS

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

WHERE

ϕ = ANGLE OF INTERNAL FRICTION OF EARTH

α = ANGLE OF INCLINATION OF BACK OF WALL

δ = ANGLE OF INTERNAL FRICTION BETWEEN WALL & EARTH

ι = ANGLE OF INCLINATION OF BACKFILL

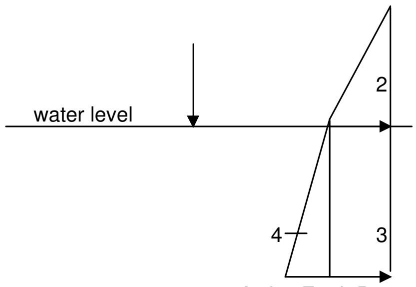
HERE				
	ϕ =	30 ^{o}	=	0.524 Radian
	α =	90 ^{o}	=	1.571 Radian
	δ =	20 ^{o}	=	0.349 Radian
	ι =	0 ^{o}	=	0 Radian
			K_a =	0.2973

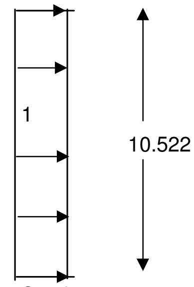
Therefore, Horizontal coefficient of Active earth pressure = **$K_a COS\phi$ = K_{ha} = 0.2794**

HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 10.522 m
For DESIGN purpose, the height of abutment is considered as, say, = **10.520 m**

CALCULATION OF ACTIVE EARTH PRESSURE





DRY. condition

a) Service Condition

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	84.66	5.261	445.41
2	Dry Earth	0.5	10.522	5.29	334.06	4.419	1476.28
3		1.0	0.000	5.29	0.00	0.000	0.00
4	SubmgEarth	0.5	0.000	0.00	0.00	0.000	0.00
TOTAL					**418.72**		**1921.69**

b) Span Dislodge Condition

Net force = 418.72 - 84.66 = **334.06** t
Net moment = 1921.69 - 445.41 = **1476.28** tm

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	84.66	5.261	445.41
2	Dry Earth	0.5	2.393	1.34	19.20	9.134	175.36
3		1.0	8.129	1.34	130.43	4.065	530.15
4	SubmgEarth	0.5	8.129	2.27	110.77	2.710	300.15
TOTAL					**345.07**		**1451.07**

b) Span Dislodge Condition

Net force = 345.07 - 84.66 = **260.40** t
Net moment= 1451.07 - 445.41 = **1005.66** tm

Forces & moments due to Abutment (Concrete) components

DRY Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment about toe
1	Toe Slab	1.0	2.300	12.00	0.500	2.40	33.12	1.150	38.09
2		0.5	2.300	12.00	0.700	2.40	23.18	1.533	35.55
3	Heel Slab	1.0	3.600	12.00	0.500	2.40	51.84	5.300	274.75
4		0.5	3.600	12.00	0.700	2.40	36.29	4.700	170.55
5	Stem Wall	1.0	1.200	12.00	8.666	2.40	299.50	3.100	928.44
6		0.5	0.200	12.00	8.666	2.40	24.96	2.433	60.73
7	stem rect	1.0	1.400	12.00	1.200	2.40	48.38	3.000	145.15
10	Cap	1.0	1.200	12.00	0.300	2.40	10.37	3.100	32.14
11	Dirt Wall	1.0	0.300	12.00	1.206	2.40	10.42	3.550	36.99
8	Retutnwall	0.5	3.600	1.00	0.700	2.40	3.02	6.100	18.45
8a		1.0	3.600	1.00	10.022	2.40	86.59	5.500	476.25
TOTAL							**627.67**		**2217.09**

H.F.L. Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment ,@ Toe
1	Toe Slab	1.0	2.300	12.00	0.500	1.40	19.32	1.150	22.22
2		0.5	2.300	12.00	0.700	1.40	13.52	1.533	20.74
3	Heel Slab	1.0	3.600	12.00	0.500	1.40	30.24	5.300	160.27
4		0.5	3.600	12.00	0.700	1.40	21.17	4.700	99.49
5	Stem Wall	1.0	1.200	12.00	6.929	1.40	139.69	3.100	433.03
6		0.5	0.200	12.00	6.929	1.40	11.64	2.433	28.33
5	Stem Wall	1.0	1.200	12.00	0.537	2.40	18.56	3.100	57.53
6		0.5	0.200	12.00	0.537	2.40	1.55	2.433	3.76
7	stem rect	1.0	1.400	12.00	1.400	1.40	32.93	3.000	98.78
10	Cap	1.0	1.200	12.00	0.30	2.40	10.37	3.100	32.14
11	Dirt Wall	1.0	0.300	12.00	1.21	2.40	10.42	3.550	36.99
8	Returnwall	0.5	3.600	1.00	0.70	1.40	1.76	6.100	10.76
8c		1.0	3.600	1.00	6.93	1.40	34.92	5.500	192.07
8d		1.0	3.600	1.00	2.39	2.40	20.68	5.500	113.72
TOTAL							**366.76**		**1309.84**

Forces & moments due to Earth and LL surcharge

DRY Case : **Self weight of Earth**

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@ Toe
8	DRY EARTH	0.5	3.600	11.00	0.700	1.8	24.95	6.100	152.18
8a		1.0	3.600	11.00	9.322	1.800	664.47	5.500	3654.60
TOTAL							**689.42**		**3806.78**

H.F.L Case :

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@Toe
8	SATURATED SOIL	0.5	3.600	11.000	0.7	1.000	13.86	6.100	84.55
8c		1.0	3.600	11.000	6.929	1.000	274.39	5.5	1509.14
8d	DRY	1.0	3.600	11.000	2.393	1.800	170.57	5.500	938.15
TOTAL							**458.82**		**2531.83**

L.L.SURCHARGE

Element No	Component	Area Factor	Length	Width	Height	Density	Weight	C.G.from toe	Moment `@ Toe
12	DRY EARTH	0.00	3.600	11.00	1.20	1.80	**0.00**	5.500	**0.00**

SUMMARY OF FORCES AND MOMENTS:

LOAD CASE	Case. L.W.L.		Case. H.F.L.		
	Service Cond.	Span dislodged	Service Cond.	Span dislodged	
Vertical load from superstructure including LL	**181.78**	0.00	**181.78**	0.00	
Vertical load from substructure (b)	1317.09	1317.09	825.59	825.59	
Total Vertical Load V = (a) + (b)	1498.87	1317.09	1007.36	825.59	
Total Horizontal Force H =	**429.72**	334.06	**356.07**	260.40	
Moment @ toe due to (a)	**563.50**	0.00	**563.50**	0.00	
Moment @ toe due to (b)	6023.87	6023.87	3841.67	3841.67	
Total Moment @ toe (M)	6587.37	6023.87	4405.17	3841.67	
Dist. of C.G. of V from toe Z = M/V	4.395	4.57	4.37	4.65	
eccentricity (e = Z -b/2)	0.745	0.924	0.723	1.003	
Relieving Moment @ c/l base (M1)	**1116.50**	**1216.48**	**728.31**	**828.28**	
overturning moment due to					
Horz. braking force	**105.78**	0.00	**105.78**	0.00	
Earth Pressure	1921.69	1476.28	1451.07	1005.66	
Total overturning Moment (M2)	**2027.47**	**1476.28**	**1556.85**	**1005.66**	
Net moment (M2-M1) = M_L	**910.96**	**259.80**	**828.54**	**177.38**	
Factor of Safety					
Against overturning (M / M2)	**3.25**	**4.08**	**2.83**	**3.82**	Safe against overturning
Against sliding (μ x V / H)	**1.744**	**1.971**	**1.41**	**1.585**	Not safe

I.R.C 78-2000:cl 706.3.4

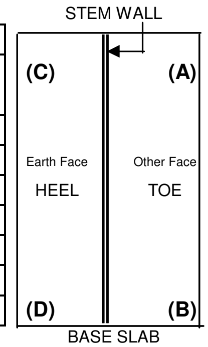
Area of base (A) = 7.300 x 12.00 = 85.20 m^2

Z_L = 106.58 m^3

Z_T = 175.20 m^3

CHECK FOR BASE PRESSURE:

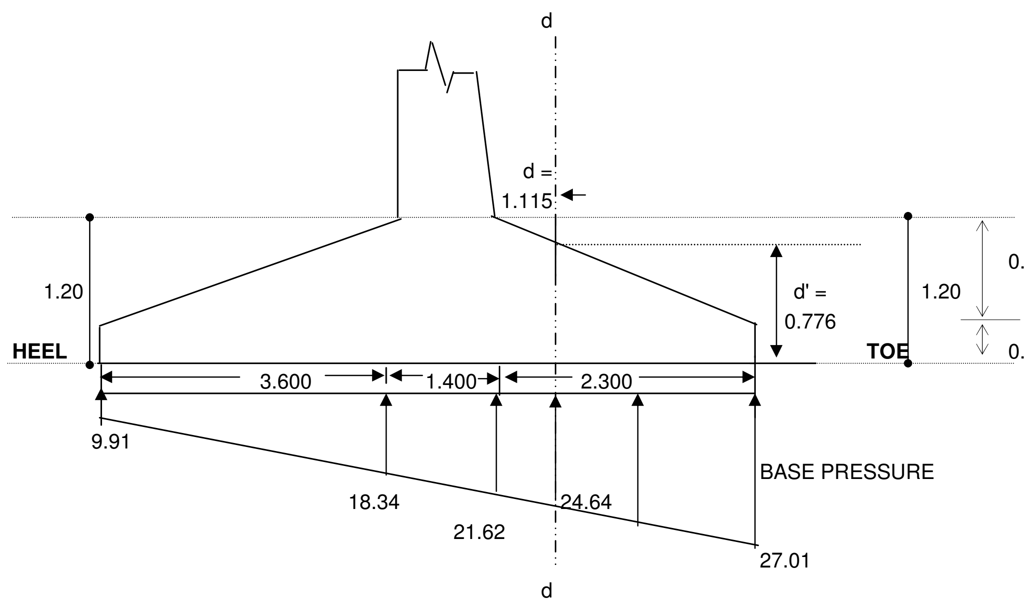
	Base Pressure	LWL CASE		HFL CASE	
		Service Cond.	Span dislodged	Service Cond.	Span dislodged
	P/A	17.59	15.46	11.82	9.69
	M_L/Z_L	8.55	2.44	7.77	1.66
	M_T/Z_T	0.87	0.00	0.87	0.00
(A)	$(P/A + M_L/Z_L + M_T/Z_T)$	**27.01**	**17.90**	**20.47**	**11.35**
(B)	$(P/A + M_L/Z_L - M_T/Z_T)$	25.27	17.90	18.73	11.35
(C)	$(P/A - M_L/Z_L + M_T/Z_T)$	9.91	13.02	4.92	8.03
(D)	$(P/A - M_L/Z_L - M_T/Z_T)$	**8.177**	**13.02**	**3.18**	**8.03**



Max.Base Pressure = **27.01** **t/m^2 <** **45.30** **Hence O.K.**

Min. Base Pressure = **3.18** **t/m^2 >** **0** **Hence O.K.**

DESIGN OF TOE SLAB



BENDING MOMENT AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward Loads	1	1.0	2.760	1.150	3.174
	2	0.5	1.932	0.767	1.481
Upward Base pressure	Rect.	1.0	-49.729	1.150	-57.189
	Trian.	0.5	-6.194	1.533	-9.497
TOTAL			-51.231		-62.031

Bending Moment at face of stem 62.031 tm/m

Effective depth required 0.742 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3024 mm^2

Minimum steel required 1673 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 669 mm^2/m
mainsteel 3024 mm^2 12 ϕ , @ 150 C/C
753.9822

Hence provide , 25 ϕ , @ 150 C/C
0 ϕ , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 250 mm^2/m
provide 12 ϕ , @ 150 C/C
753.9822

Check for Shear

SHEAR FORCE AT "d" FROM FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward loads	1	1.0	1.422	0.593	0.843
	2	0.5	0.995	0.395	0.393
Upward base pressure	Rect.	1.0	-29.193	0.593	-17.297
	Trian.	0.5	-1.405	0.790	-1.110
TOTAL			-28.181		-17.171

Effective depth (d') at distence d 0.776 m

Shear force at critical section 28.2 t

Bending Moment at critical section 17.17 tm

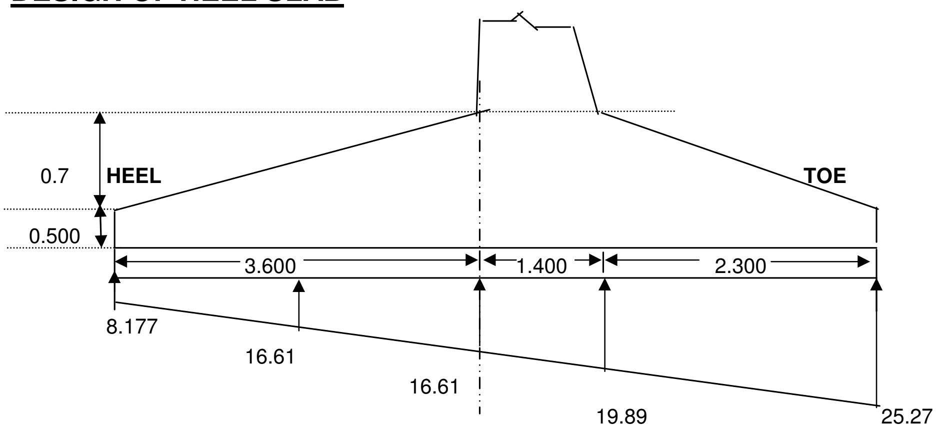
$\tan\beta$ = 0.34

Net shear force S-M*tanβ/d' **20.72** t

Hence, Shear stress 26.71 t/m^2

% of reinforcement 0.42
Permissible shear stress 29.05 t/m^2 **Hence O.K.**

DESIGN OF HEEL SLAB



BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	moment
Downward Loads.	3	1	4.320	1.800	7.776
	4	0.5	3.024	1.200	3.629
	8	0.5	4.536	2.400	10.886
	8a	1	60.407	1.800	108.732
		0.5	4.536	2.400	5.443
		1	7.468	1.800	13.442
Upward base pressure	Rect.	1	-29.438	1.800	-52.989
	Trian.	0.5	-15.174	1.200	-18.209
TOTAL			39.678		78.710

Bending Moment at face of stem **78.710** tm

Effective depth required 0.836 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3838.91 mm^2
Minimum steel 1672.50 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 669.00 mm^2/m
mainsteel 3838.91 mm^2

12 ϕ , @ 150 C/C

Hence provide , 25 ϕ , @ 125 C/C 753.9822

0 ϕ , @ 125 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 250 mm^2/m

provide 12 ϕ , @ 150 C/C

753.9822

Check for Shear (Critical section at face of stem)

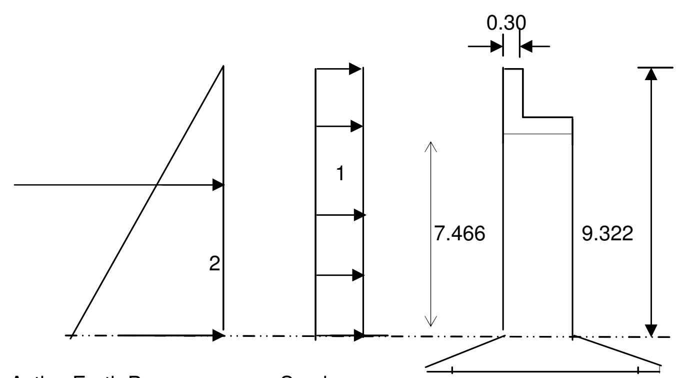
Shear force at face of stem 39.68 t

tan β 0.194
Bending moment at face of stem 78.710 tm
Net shear force S-M*tanβ /d 25.95 t

Hence, Shear stress 23.27 t/m^2
% of reinforcement 0.34

Permissible shear stress 26.52 t/m^2 **Hence O.K.**

DESIGN OF STEM WALL



SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT

R.L. OF SECTION = 211.878 m

DRY Condition

a) Service Condition

Element No.	Area factor	Height of E.P. diagram	Earth Pressure	Force	L.A.	Moment tm
1	1	8.972	0.603	65.0	4.486	291.46
2	0.5	8.972	4.512	242.9	3.768	915.25
HORIZONTAL FORCE				11.00	8.066	88.73
TOTAL				318.86		**1295.44**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 301.029 + 10.42 + 10.37 + 181.78
= 503.592 t
Longitudinal Moment = 1295.444 t-m
Transverse Moment = 152.042 t-m

b) Span Dislodged case

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	301.029	+	10.42	+	10.37	+	0.00
	=	321.817 t						
Longitudinal Moment	=	1206.718 t-m						
Transverse Moment	=	0.000 t-m						

H.F.L. Condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.603	64.97	4.486	291.46
2	Dry Earth	0.5	1.987	1.11	13.24	7.591	100.48
3		1.0	6.929	1.11	92.32	3.465	319.83
4	SubmgEarth	0.5	6.929	1.94	80.48	2.310	185.88
TOTAL					**251.01**		**897.66**

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	169.888	+	10.42	+	10.37	+	181.78
	=	372.451 t						
Longitudinal Moment	=	986.389 t-m						
Transverse Moment	=	152.042 t-m						

b) Span Dislodged case

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	169.888	+	10.42	+	10.37	+	0.00
	=	190.676 t						
Longitudinal Moment	=	897.663 t-m						
Transverse Moment	=	0.000 t-m						

Cross Sectional area 1.3 m^2

For horizontal reinforcement area of steel required for the stem at the section/metre = 215.3876 mm^2

Providing **12** ,@ 525.09 c/c say, **150** C/C as horizontal reinforcement

Movement of Deck:

Total Length of Bridge = 12.6 m

= 0.0005 x 6300 = 3.15 mm

Longitudinal Force :

Size of bearing = 500 x 250 x 50 mm

Strain in bearing = $\frac{3.150}{50}$ = 0.063

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.063 x 1.0 x 500 x 250 = 7875 N

= 0.803 t

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

= 0.803 x 4 x 1.05 = 3.372 t

Refer IRC : 6 clause 214.5.1.5;

10 % increase for variation in movement of span

Total shear force = 1.1 x 3.372 = 3.709 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 28.3] = 5.666 t

For class 70R wheeled : F_h = [0.2 x 60.5] = 12.106 t

For class A 3 lane F_h = 0.2 x 28.3 + 0.05 x 28.3 = 7.0825 t

For class 70R wheeled +class A 1 lane

F_h = 0.2 x 60.5 + 0.05 x 28.3 = 13.523 t

For class A 3 lane

Longitudinal horizontal force = $\frac{7.083}{2}$ + 3.709 = 7.25 t Say, 8.0 t

For class 70R wheeled +class A 1 lane

Longitudinal horizontal force = $\frac{13.523}{2}$ + 3.709 = 10.5 t Say, 11.0 t

Span dislodged condition

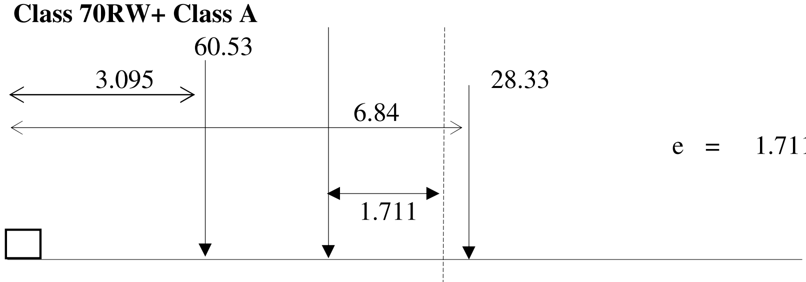
Longitudinal horizontal force = $\frac{0.000}{2}$ + 3.709 = 3.709 t Say, 4.0 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 3 lane	**8.00**
70R+class A 1lane	**11.00**
Span dislodged condition	**0.00**

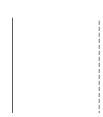
Transverse eccentricity

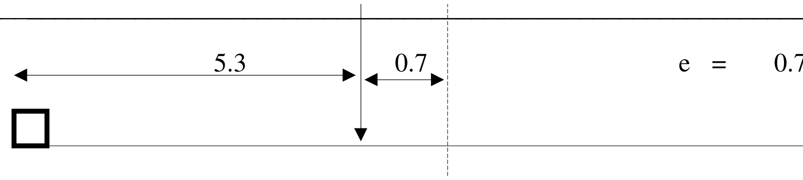
Class 70RW+ Class A



e = 1.711

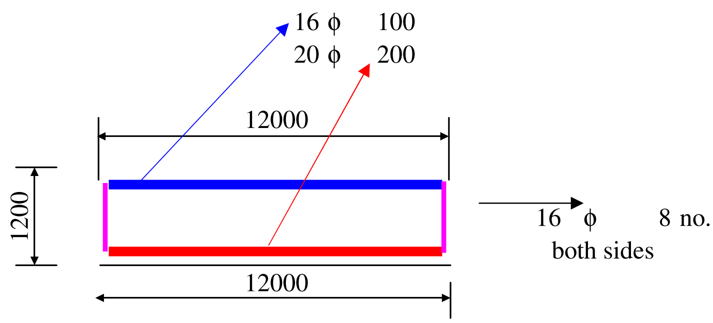
Class A 3 lane





Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

	P		M_L	M_T
Dead Load Reaction:				
	71.81	t		
SIDL	21.10	t		
L.L Max Reaction Case				
70R + Class A	88.86	t		152.04
Class A 3 Lane	84.99	t		59.49
	181.78			**152.04**



PLAN : ABUTMENT SHAFT

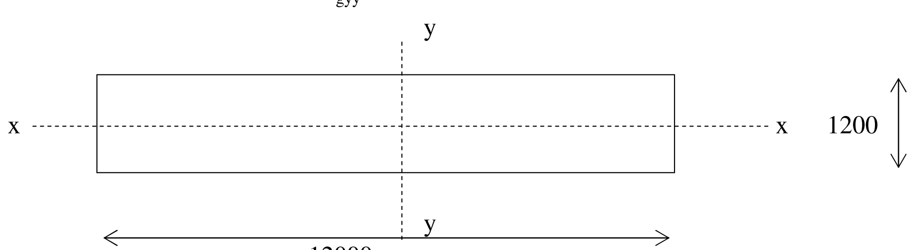
% Reinforcement provided = 86155.037 mm^2

Summary of Loads at Abutment Shaft bottom :

	P	M_L	M_T
1 DRY condition			
With L.L	503.59	1295.44	152.04

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}	=	1.728E+12	mm^4
Gross M.O.I of section	I_{gyy}	=	1.728E+14	mm^4



Abutment section

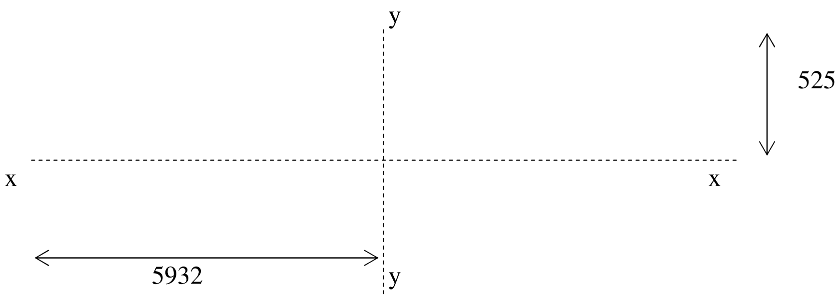
Transformed sectional properties of section:

Adopting

Modular ratio	m	=	10	
Cover			70	68
Dia of Bars		=	20	16
No of bars in tension face (longer)		=	200	
No of bars in compression face		=		100
No of bars in shorter direction		=		8
Total bars in section		=	316	

Steel Area A_s = 86155 mm^2

% of Steel = 0.5983 %



A_{sx} = 62832 mm^2

A_{sy} = 1608.5 mm^2

Area of concrete $A_c = A_g - A_s$ = 14313845 mm^2

C.G of Steel placed on longer face = 530 mm

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

Transformed Area of Section A_{tfm} = 15175395 mm^2

Transformed M.I$_{txx}$ = $I_{gxx} + 2 [m - 1] A_{s} ax^2$

= 2.04569E+12 mm^4

$Z_{xx} = \dfrac{M.Itxx}{d/2}$ = 3.409E+09 mm

Transformed M.I$_{tyy}$ = $I_{gyy} + 2 [m - 1] A_{s} ay^2$

= 1.73819E+14 mm^4

$Z_{yy} = \dfrac{M.I_{tyy}}{d/2}$ = 2.897E+10 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.728E+12 mm^4

Area of section = 14400000 mm^2

r = 346.41016 mm

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

Abutment shaft height L = 9.426 m

Effective length L_{eff} = 11.311 m

Slenderness ratio = 32.653 < **50**

Type of member = 1

1 Short Column

2 Long Column

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

β = 1

Permissible stresses **:** *concrete*

σ_{cbc} = 8.3333 N/mm^2

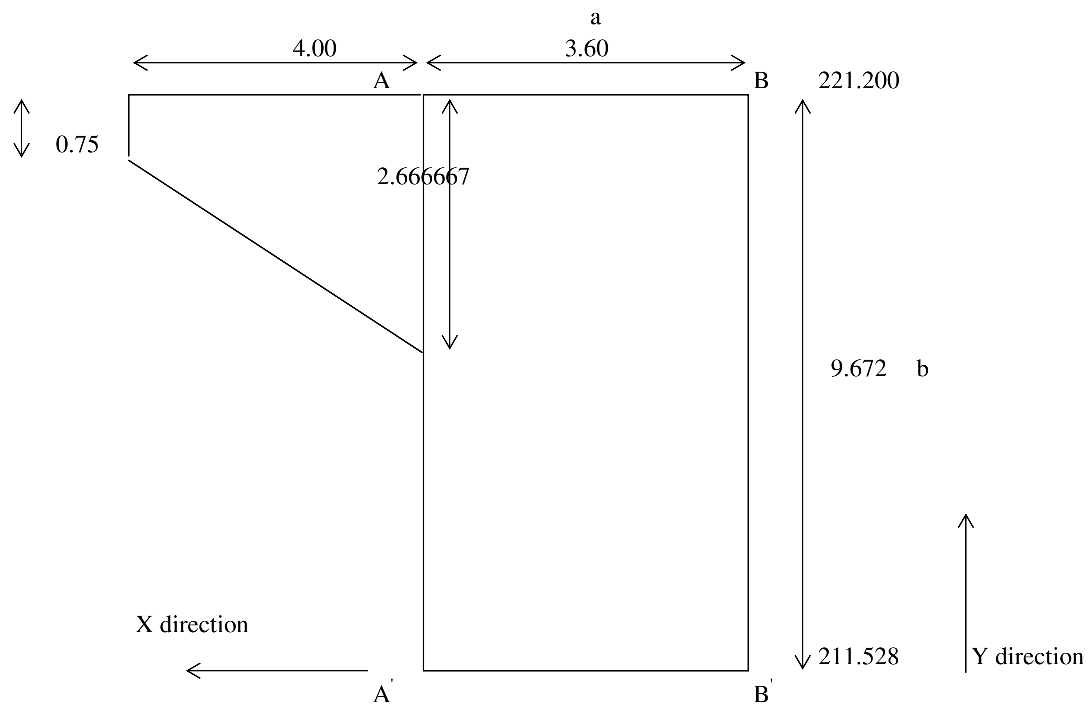
σ_{co} = 6.25 N/mm^2

Tensile stress = 0.61 N/mm^2

Permissible stresses : *Steel*

σ_{st} = 200 N/mm^2

S.No Item	DRY Case
Loads and Moments	With L.L
1 P	503.59 t
2 M_L	1295.44 t-m
3 M_T	152.042 t-m
Actual(calculated) Stresses	
4 $\sigma_{co,cal}$ P/A_{tfm}	0.331847713
5 $\sigma_{cbc,cal}$ M_L/Z_{xx}	3.799531164
6 $\sigma_{cbc,cal}$ M_T/Z_{yy}	0.052483082
7 $\sigma_{cbc,cal}$ = 5 + 6	3.852014246
Permissible Stresses	
8 σ_{cbc}	8.3333333
9 σ_{co}	6.25
Check for Minimum steel area mm^2	
10 Conc.Area Required for directstress	
(1)/(9)	805747.24
11 0.8% of area required	6445.9779
12 0.3% of A_g	43200
13 Governing steel mm^2	43200
14 Provided Steel area mm^2	86155.037
Check for safety of section $\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$ *Check for Cracked /Uncracked section*	0.5153373 < 1
$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-3.520167
Permissible Basic tensile stress in concrete	-0.61
Section to be designed as	**Cracked**



Width of Solid return wall (a)	=	3.60
Width of Cantilever return wall	=	4.00
Avg Height of Solid return wall (b)	=	9.672
Height of Cantilever return at Tip	=	0.75
Height of Cantilever return at Root	=	2.666667
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	=	0.333333
j	=	0.888889
R	=	151.1111 t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.3722084	For a/b	=	0.25	β_1	=	0.182	β_2	=	0.188
			For a/b	=	0.375	β_1	=	0.353	β_2	=	0.398

a/b = 0.3722084 β_1 = 0.349181

β_2 = 0.39331

Live Load Surcharge:

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

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

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

$\sigma_{bmax} = \dfrac{0.3491811 \times 0.603504 \times 93.55}{0.25} = 78.85396$ 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 $= 78.853959 \times 0.041667 = 3.285582$ t-m/m

$\sigma_{amax} = \dfrac{0.3933102 \times 0.603504 \times 93.55}{0.25} = 88.81941$ 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 $= 88.819413 \times 0.041667 = 3.700809$ t-m/m

Case (2) For Triangular loading due to earth pressure

$a/b = 0.3722084$ For $a/b = 0.25$ $\beta_1 = 0.133$ $\beta_2 = 0.09$

For $a/b = 0.375$ $\beta_1 = 0.212$ $\beta_2 = 0.148$

$a/b = 0.3722084$ $\beta_1 = 0.210236$

$\beta_2 = 0.146705$

Earth pressure:

$q = 0.2794 \times 1.8 \times 9.672 = 4.864242$ t/m^2

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

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

$\sigma_{bmax} = \dfrac{0.2102357 \times 4.864242 \times 93.55}{0.25} = 382.6611$ 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 = 382.66108 x 0.041667 = **15.94421** t-m/m

$$\sigma_{amax} = \frac{0.1467047 \times 4.864242 \times 93.55}{0.25} = 267.0249 \text{ t/m}^2$$

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 = 267.02494 x 0.041667 = **11.12604** t-m/m

Total Moment in Solid Return /m height = **14.82685** t-m/m
Along X-direction

Total Moment in Solid Return /m width = **19.22979** t-m/m
Along Y-direction

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A′

$$\begin{aligned} M = {} & 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 \\ & + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 \\ & + 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times dx \times (4.00 - X) \\ & + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times (4.00 - X) \end{aligned}$$

$$= 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.66667$$

$$= \mathbf{14.11063} \text{ t-m}$$

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

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

$$M = R \times b \times d^2$$
$$= 151.1111 \times 2.666667 \times 0.179776 = 72.44307 \text{ t-m}$$

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

A_{st}/m = 688.231 mm^2/m

Provide 12 mm dia @ 150 mm c/c providing 753.9822 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 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 5.60
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 5.60
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (7.60 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (7.60 - X)

= 10.13887 + 3.168396 + 0.11176 x 98.13333 + 0.653796 x 39.46667

= **50.07779** t-m

Moment in Solid Return /m height = 14.82685 + $\frac{50.07779}{9.672}$ = 20.00445 t-m/m

Moment in Solid Return /m width = = 19.22979 t-m/m

Design of face B-B'

Moment in Solid Return /m height = **20.00445** t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 20 - 13 = 418 mm

M = R x b x d^2
= 151.1111 x 1 x 0.174306 = 26.33961 t-m

A_{st} = $\frac{20.00445 \times 10^6}{20400 \times 0.888889 \times 0.4175}$ = 2642.363 mm^2

A_{st}/m = 2642.363 mm^2/m

Provide 25 mm dia @ 150 mm c/c providing 3272.492 mm^2 on earth face.
Provide 16 mm dia @ 150 mm c/c providing 1340.413 mm^2 on other face.

Along Horizantal direction.

Design of face A'-B'

Moment in Solid Return /m width = **19.22979** t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 0 - 13 = 438 mm

M = R x b x d^2
= 151.1111 x 1 x 0.191406 = 28.92361 t-m

A_{st} = $\frac{19.22979 \times 10^6}{20400 \times 0.888889 \times 0.4375}$ = 2423.924 mm^2

A_{st}/m = 2423.924 mm^2/m

Provide 25 mm dia @ 160 mm c/c providing 3067.962 mm^2 on earth face.
Provide 16 mm dia @ 160 mm c/c providing 1256.637 mm^2 on other face.

Along Vertical direction.

1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure	K_a	=	0.279384
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		=	0.5
Avg. cover to reinforcement. (FOR 2-3 LAYERS)		=	0.150 m

H (From formation level) = 9.322 m

DESIGN FOR BENDING MOMENT

S.NO.	UNITS	1
H (FROM TOP)	m	9.322
WIDTH OF RETURN AT THIS LEVEL	m	3.600
FORCE DUE TO EARTH PRESSURE	t	123.197
MOMENT DUE TO EARTH PRESSURE	tm	482.344
FORCE DUE TO L.L. SURCHARGE	t	31.718
MOMENT DUE TO L.L. SURCHARGE	tm	147.836
TOTAL MOMENT	tm	630.180
DESIGN MOMENT	tm	630.180
REQUIRED EFFECTIVE DEPTH	m	3.229
EFF. DEPTH AVAILABLE	m	3.450
AREA OF STEEL REQUIRED	cm^2	100.720
DIAMETER OF BAR PROVIDED	mm	32
TOTAL NO. OF BARS	no.	16
AREA OF STEEL PROVIDED	cm^2	128.68
		O.K.
		27.8
CHECK FOR SHEAR STRESS		
SHEAR FORCE	t	159.914
SHEAR STRESS	t/m^2	92.704
Ast/bd x 100	%	0.584
PERMISSIBLE SHEAR STRESS	MPa	0.33
PERMISSIBLE SHEAR STRESS	t/m^2	33.67
Asv/sv	cm^2/m	8.387

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	=	300	mm			
Assuming a cap thickness of	=	300	mm			
Volume of Abutment cap	=	0.3	x	1.20	x	12
	=	4.32	m^3			
Quantity of steel	=	1 %	of volume			
	=	$\frac{1}{100}$	x	4.32		
	=	0.0432	m^3			
Quantity of steel to be provided at top	=	0.0216	m^3			
Quantity of steel to be provided at bottom	=	0.0216	m^3			

Top & bottom face:

Quantity of steel to be provided in Longitudinal di				=	0.0108	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

$$\frac{0.0108}{11.9} = 907.563 \text{ mm}^2$$

Provide **9** nos of bars **12** mm dia at top & bottom face.

= 1017.88 mm^2

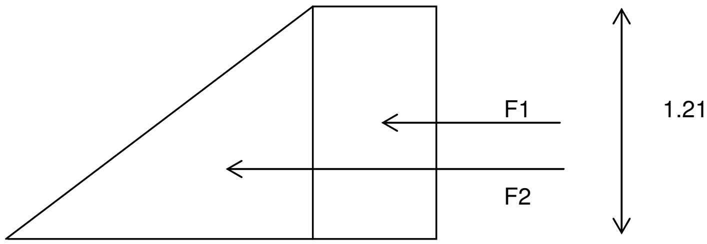
Transverse steel:

Quantity of steel to be provided in Longitudinal di				=	0.0108	m^3
Assuming a clear cover of				=	50	mm
Assuming a dia of bar				=	12	mm
Length of bar	1.20	-	0.100	=	1.1	m
Volume of each stirrup				=	0.00012	m^3
no of stirrups required for m/length				=	8	nos
Required Spacing				=	$\frac{1000}{8}$	
				=	125	mm

Provide 12 mm dia bar 125 mm c/c stirrups throught in length of abutment cap.

904.779 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	1.21	=	9.704344	t
Intensity for triangular portion	=	0.2794	x	2.00	x	1.21	=	0.673913	t/m^2
F2	=	0.673913	x	12.00	x	1.21	=	4.876433	t
M1	=	9.704344	x	0.60			=	5.85172	t-m
M2	=	4.876433	x	0.50652			=	2.470011	t-m
M1+ M2							=	**8.32173**	t-m
Total moment at base of dirt wall /m length							=	0.693478	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 = $\dfrac{0.693478}{1.51 \times 1000}$ = 67.76848 mm

Ast req = $\dfrac{0.693478}{200 \times 0.889 \times 244}$ = 159.8493 mm^2/m

Minimum steel = $\dfrac{0.12}{100} \times 300 \times 1000$ = 360 mm^2/m

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	0.693478	=	0.208043	t-m/m

Ast req = $\dfrac{0.208043}{200 \times 0.889 \times 238}$ = 49.16374 mm^2/m

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.

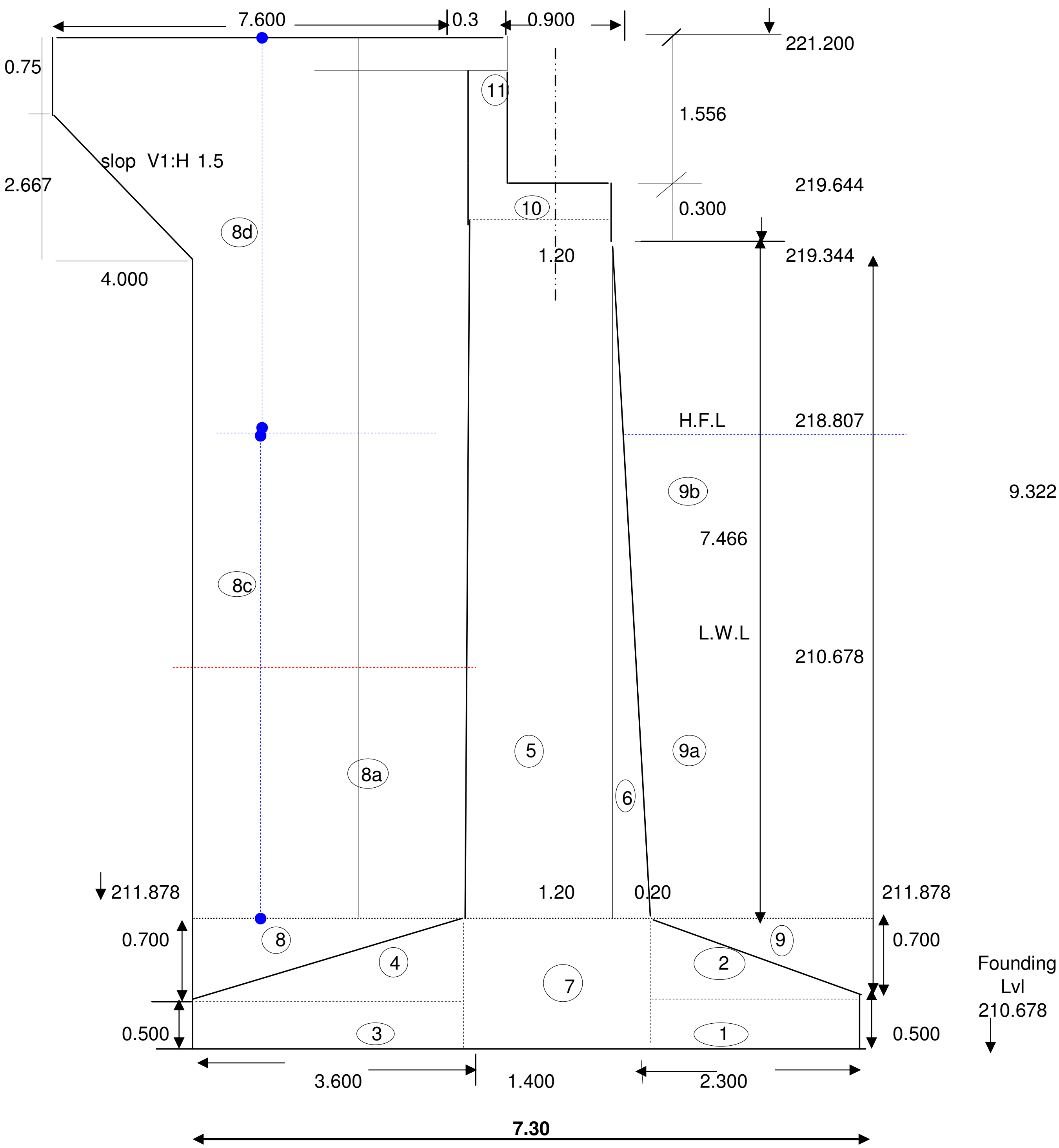
392.6991 mm^2/m

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


7.600
0.3
0.900
221.200
0.75
11
1.556
slop V1:H 1.5
2.667
219.644
10
0.300
8d
219.344
1.20
4.000
H.F.L
218.807
9b
9.322
7.466
8c
L.W.L
210.678
5
9a
8a
6
211.878
1.20
0.20
211.878
0.700
8
4
9
0.700
2
7
Founding
Lvl
210.678
0.500
3
1
0.500
3.600
1.400
2.300
7.30

```
INPUT FILE: 70r.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.3 0 0;3 14.3 0 0;4 14.6 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    22. LOAD GENERATION 175
    23. TYPE 1 -13.4 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 74
    26. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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

      2   74      0.00     34.46      0.00      0.00      0.00      0.00
      3   74      0.00     65.54      0.00      0.00      0.00      0.00
    27. FINISH
```

```
INPUT FILE: CLASS A.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.3 0 0;3 14.3 0 0;4 14.6 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    22. LOAD GENERATION 202
    23. TYPE 1 -18.8 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 74
    26. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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

     2   74      0.00     30.31      0.00      0.00      0.00      0.00
     3   74      0.00     19.69      0.00      0.00      0.00      0.00

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

    27. FINISH
```

```
Abutment SHAFT ..DRY NORMAL

 Depth of Section              =     1.400 m
 Width of Section              =    12.000 m

 along width-compression face- no of bar: 100   tension face- no of bar: 200
 Dia (mm)                                  16                             20
 Cover (cm)                               7.50                           10.5
 along depth-compression face- no of bar:   8   tension face- no of bar:   8
 Dia (mm)                                  16                             16
 Cover  (cm)                              7.50                            7.5

 Modular Ratio : Compression   =      10.0
 Modular Ratio : Tension       =      10.0
 Allowable Concrete Stress     =     85.00 Kg/cm^2
 Allowable Steel Stress        =   2040.00 Kg/cm^2

 Axial Load                    =   503.592 T
 Mxx                           =  1295.444 Tm
 Myy                           =   152.042 Tm


 Intercept of Neutral axis : X axis : = 248.283 m
                           : y axis : =    .390 m

          Concrete Stress Governs Design

 Stress in Concrete due to Loads =   57.50 Kg/cm^2
 Stress in Steel    due to Loads = 1360.23 Kg/cm^2
 Percentage of Steel             =     .51 %

 Abutment SHAFT ..DRY SPAN DISLODGED

 Depth of Section              =     1.400 m
 Width of Section              =    12.000 m

 Modular Ratio : Compression   =      10.0
 Modular Ratio : Tension       =      10.0
 Allowable Concrete Stress     =     85.00 Kg/cm^2
 Allowable Steel Stress        =   2040.00 Kg/cm^2

 Axial Load                    =   321.817 T
 Mxx                           =  1206.718 Tm
 Myy                           =      .000 Tm

 Intercept of Neutral axis : X axis : = ******* m
                           : y axis : =    .358 m

             Steel Stress Governs Design

 Stress in Concrete due to Loads =   51.94 Kg/cm^2
 Stress in Steel    due to Loads = 1358.48 Kg/cm^2
 Percentage of Steel             =     .51 %
```

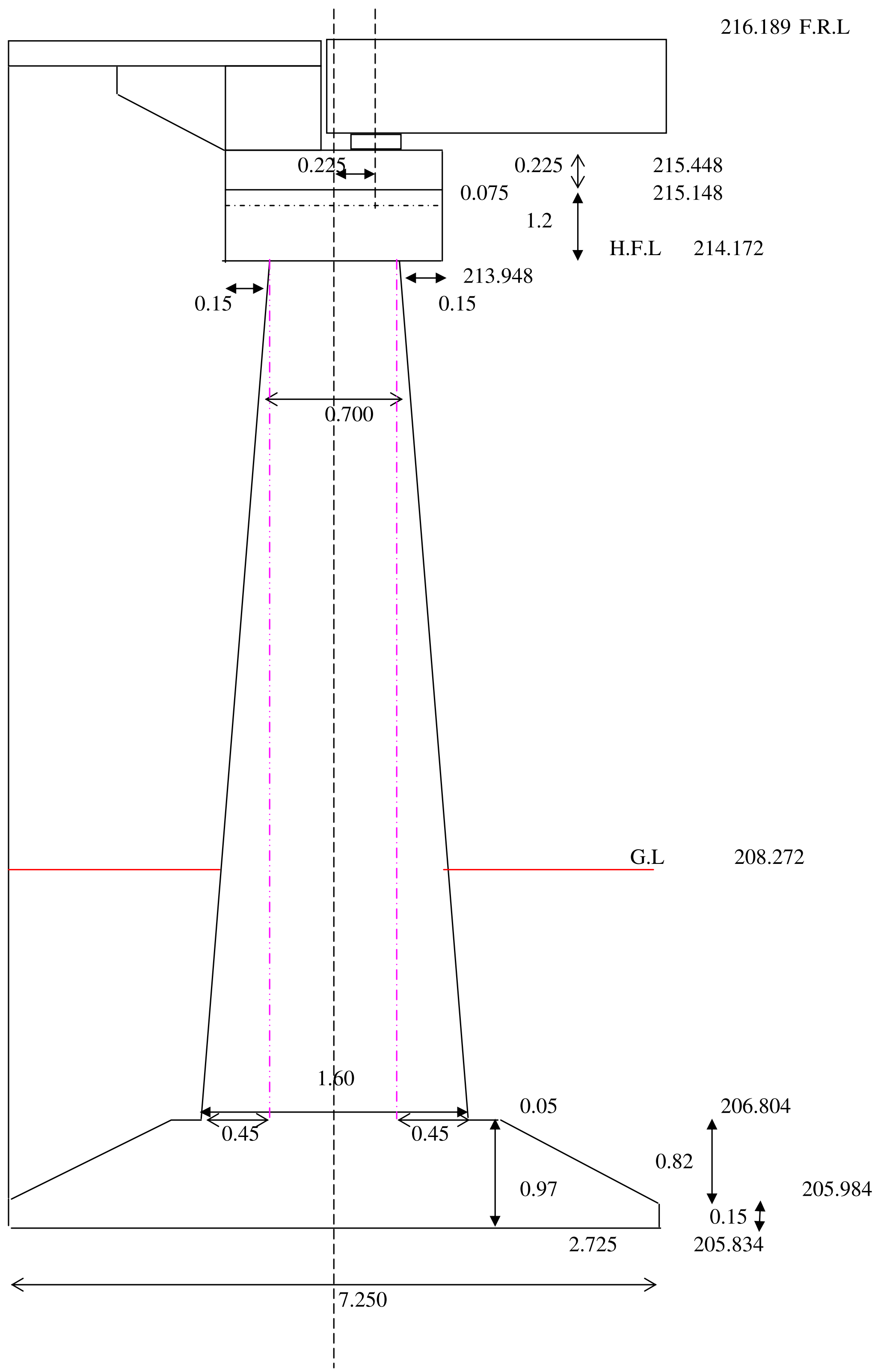
```
Abutment SHAFT ..HFL NORMAL

 Depth of Section              =     1.400 m
 Width of Section              =    12.000 m

 Modular Ratio : Compression   =      10.0
 Modular Ratio : Tension       =      10.0
 Allowable Concrete Stress     =     85.00 Kg/cm^2
 Allowable Steel Stress        =   2040.00 Kg/cm^2

 Axial Load                    =   372.451 T
 Mxx                           =   986.389 Tm
 Myy                           =   152.042 Tm


 Intercept of Neutral axis : X axis : = 189.496 m
                           : y axis : =    .391 m

           Concrete Stress Governs Design



 Stress in Concrete due to Loads =   44.09 Kg/cm^2
 Stress in Steel    due to Loads = 1046.55 Kg/cm^2
 Percentage of Steel             =     .51 %


 Abutment SHAFT ..HFL SPAN DISLODGED

 Depth of Section              =     1.400 m
 Width of Section              =    12.000 m

 Modular Ratio : Compression   =      10.0
 Modular Ratio : Tension       =      10.0
 Allowable Concrete Stress     =     85.00 Kg/cm^2
 Allowable Steel Stress        =   2040.00 Kg/cm^2

 Axial Load                    =   190.676 T
 Mxx                           =   897.663 Tm
 Myy                           =      .000 Tm

 Intercept of Neutral axis : X axis : = ******* m
                           : y axis : =    .349 m

              Steel Stress Governs Design

 Stress in Concrete due to Loads =   38.47 Kg/cm^2
 Stress in Steel    due to Loads = 1044.81 Kg/cm^2
 Percentage of Steel             =     .51 %
```

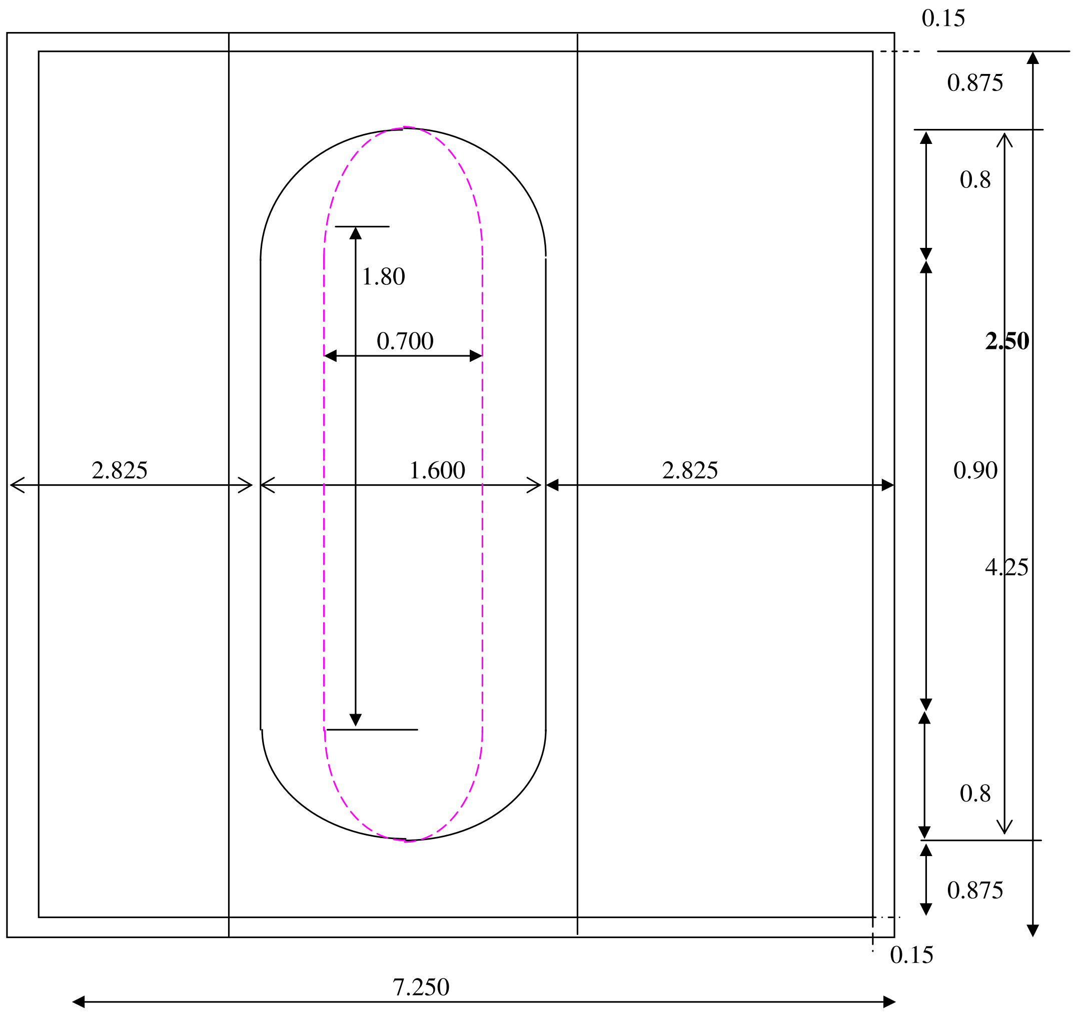
DESIGN OF SUPERSTRUCTURE

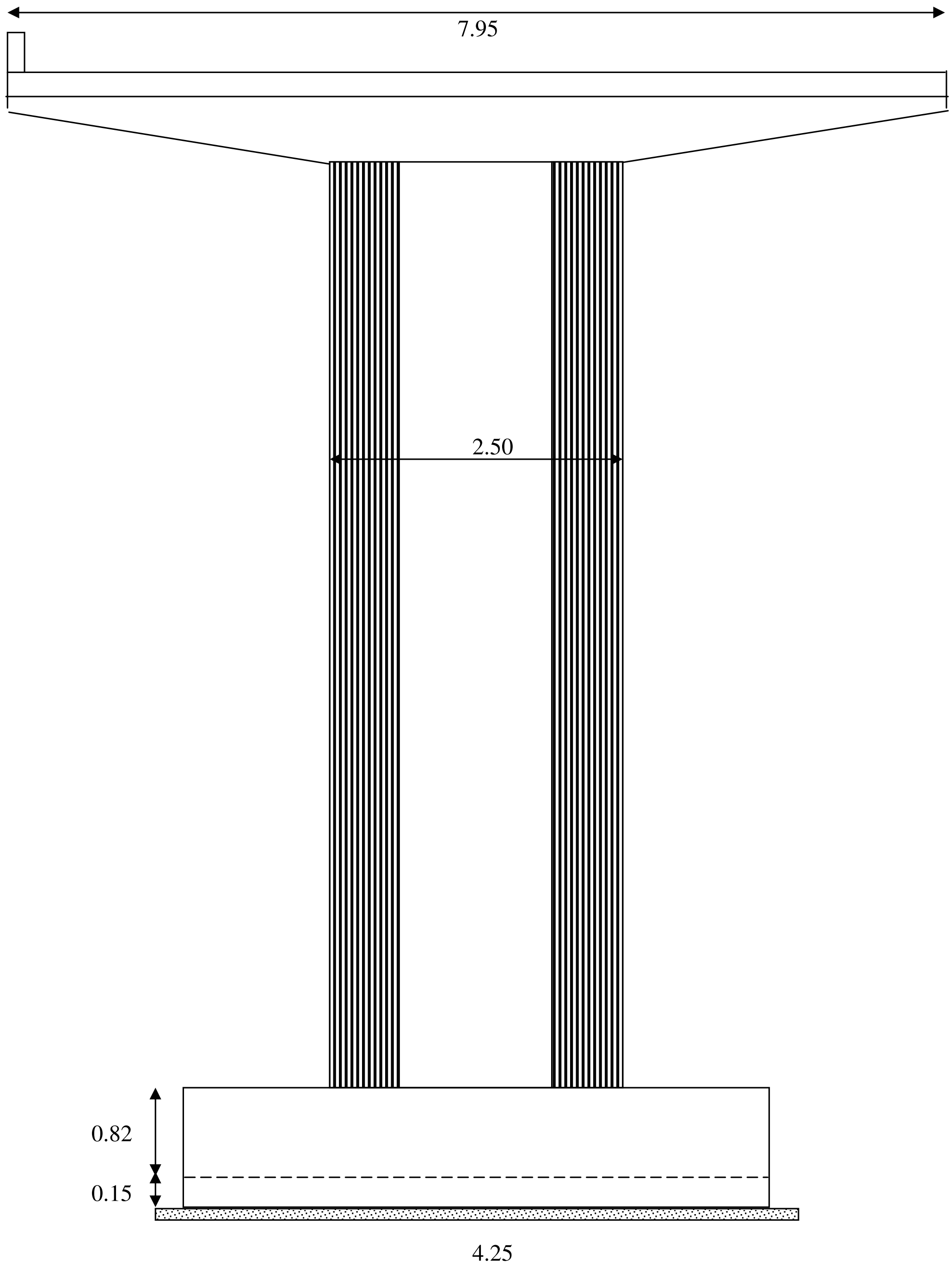
For Design of Superstructure of RCC Girder 12.0 m span refer MOST STANDARD Drawing titled “STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and Slab Superstructure)” Drg. No. SD/260 to SD/266.

BRIDGE AT CH:27+600 & 28+900

STABILITY CHECK & DESIGN OF SUBSTRUCTURE(ABUTMENT PIER)


216.189 F.R.L
0.225
0.225
215.448
0.075
215.148
1.2
H.F.L 214.172
213.948
0.15
0.15
0.700
G.L 208.272
1.60
0.05
206.804
0.45
0.45
0.82
0.97
205.984
0.15
2.725
205.834
7.250


0.15
0.875
0.8
1.80
0.700
2.50
2.825
1.600
2.825
0.90
4.25
0.8
0.875
0.15
7.250


7.95
2.50
0.82
0.15
4.25

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete	=	M - 20	
Abutment Cap	=	M - 25	
Grade of reinforcement steel	=	Fe - 415	
Centre to Centre distance of A / Expansion joints	=	9.200	m
Centre to Centre distance of Bearing	=	8.800	m
Depth of superstructure	=	660	mm
Thickness of wearing coat	=	56.00	mm
Formation level along C of carriage way	=	216.189	m
Soffit level	=	215.448	
Pedestal top level	=	215.473	
Height of bearing and Pedestal	=	0.025	m
L.W.L./Bed level	=	208.272	m
H.F.L	=	214.172	m
M.S.L	=	205.452	m
Founding Level	=	205.834	m
Abutment cap top level	=	215.448	m

Live Load **(a)** Class A two Lane
(b) Class 70R wheeled

Bearing neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:

1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000

Longitudinal translation due to creep, shrinkage & temperature = 0.0005
Horizontal movement = 0.0005 x 4.60 x 1000 = 2.300 mm

Longitudinal Force :

Size of bearing = 400 x 250 x 50 mm

Strain in bearing = $\frac{2.300}{50}$ = 0.046

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.046 x 1.0 x 400 x 250 = 4600 N
= 0.469 t

Total shear force for 4 bearings (with 5 % increase)
= 0.469 x 4 x 1.05 = 1.969 t

Refer IRC : 6 clause 214.5.1.5;
10 % increase for variation in movement of span
Total shear force = 1.1 x 1.969 = 2.166 t

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

For Class A 2 lane : F_h = 0.2 x 49.5 = 9.896 t

For class 70R wheeled : F_h = 0.2 x 47.72 = 9.544 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 2 lane	7.11
70R	6.94
Span dislodged condition	0.00

Dry Condition with L.L					
a) Vertical load and their moments about C/L of Foundation base.					
I					
1 D.L Reaction	**P**	**e_L**	**M_L**	**e_T**	**M_T**
a Left span	58.00	0.225	13.05	0	0
2 S.I.D.L					
a Left span	13.20	0.225	2.97	0	0
	71.2		**16.02**		**0**
4 **L.L**					
a Left span					
70R Wheeled	47.72	0.225	10.737	1.1	52.492
Class A 2 Lane	49.48	0.225	11.133	0.625	30.925
	118.92		**26.757**		**52.492**

II Substructure	**V**	**ρ**	**P**	**e_L**	**M_L**	**e_T**	**M_T**
1 Pedestal							
a Left span	0.0096	2.4	0.0231	0.225	0.0052	0	0
2 Abut cap R	2.385	2.4	5.724	0	0	0	0
Abut cap Tr	5.37	2.4	12.888	0	0	0	0
3 Abut shaft							
up to G.L	14.461	2.4	34.706	0	0	0	0
below G.L	3.7401	2.4	8.9762	0	0	0	0
4 Footing	20.043	2.4	48.103	0	0	0	0
5 Earth above footing	51.338	1.80	92.408	0	0	0	0
6							
7 Earth wt on heel side	83.277	1.8	149.898	-2.213	-331.73		
			202.83		**-332**		**0**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L	**H_T**	**e_L**	**M_L**	**e_T**	**M_T**
	6.94	0	9.639	66.879	9.614	0
2 Earth pressure				369.35		
	6.94	0		**436.22**		**0**

Summary

P = 321.75 t

M_L = 131 t-m

M_T = 52.492 t-m

A	=	30.813
Z_L	=	37.232
Z_T	=	21.826

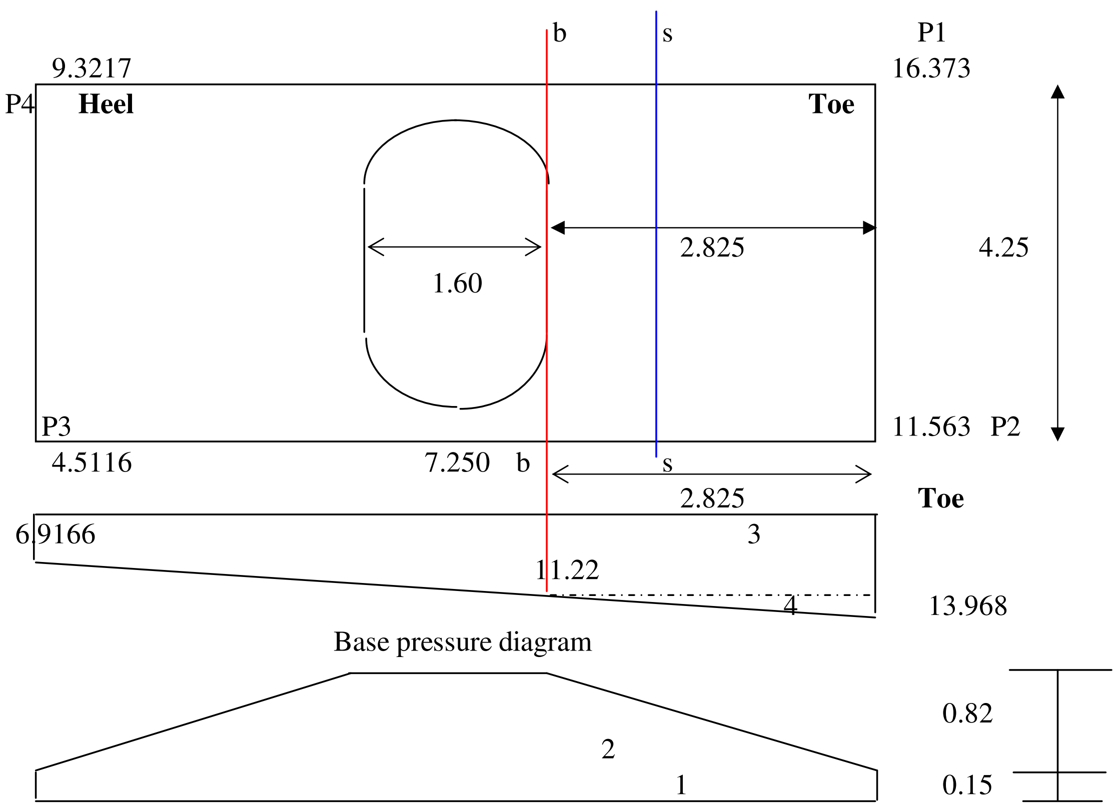
Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{321.75}{30.813} + \frac{131.26}{37.232} + \frac{52.492}{21.826} = 16.373 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{321.75}{30.813} - \frac{131.26}{37.232} - \frac{52.492}{21.826} = 4.5116 \quad t/m^2$$



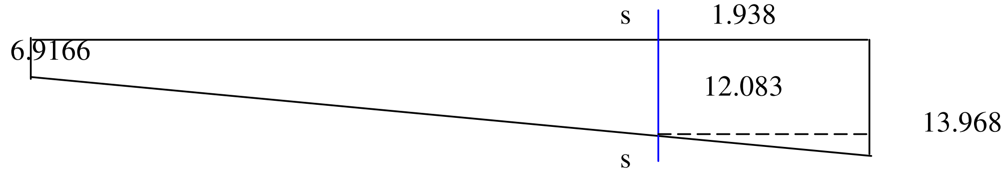
Design of Toe slab

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		1.017		1.4125		1.4365
	2		0.5		2.7798		0.9417		2.6176
Upward base pressure									
	3		1		-11.22		1.4125		-15.849
	4		0.5		-3.8808		0.9417		-3.6544

Net bending moment at face of stem	=	15.449 t-m/m
Effective depth required	=	0.4453 m
Effective depth Provided at face of stem	=	0.885 m
Area of reinforcement required	=	933.49 mm^2
Minimum steel req	=	1327.5 mm^2
Provide **16** ϕ **140** mm c/c	=	1436.2 mm^2

Shear

Effective depth d of footing	=	0.887 m
Total depth of section at distance d from Abutment	=	0.7125 m
Effective depth at distance d from face of Abutment	=	0.6215 m



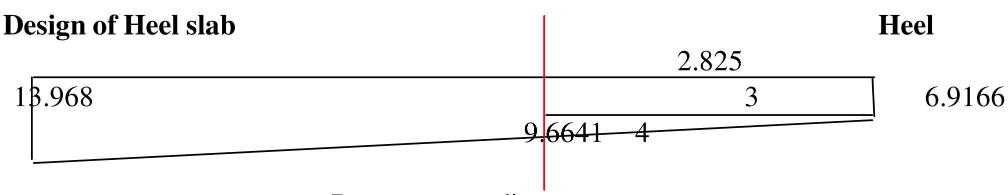
width b = 4.25

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		0.6977		0.969		0.6761
	2		0.5		1.907		0.646		1.2319
Upward base pressure									
	3		1		-12.083		0.969		-11.708
	4		0.5		-1.8264		0.646		-1.1798
					11.30				**10.98**

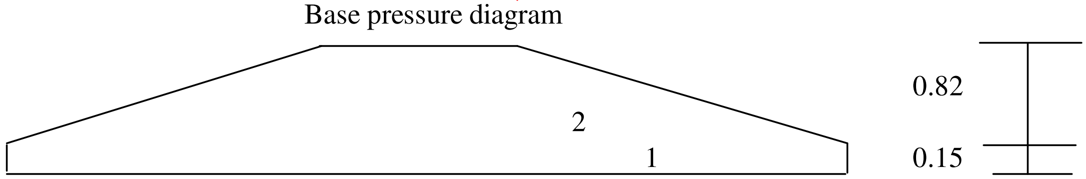
Effective depth at a distance of d eff	=	0.6215 m
Shear at critical section	=	11.30 t
Bending moment at critical section	=	10.98 t-m
tan(β) = 0.423		
Net shear force	=	3.83 t
Shear stress	=	6.1617 t/m^2
% of reinforcement	=	0.2311
Permissible shear stress	=	26.279 t/m^2

NO SHEAR REIN FORCEMENT REQ

Design of Heel slab **Heel**



Base pressure diagram

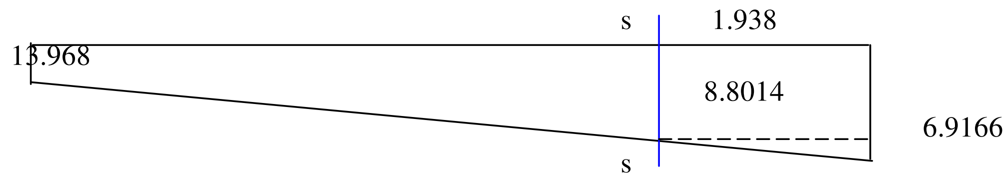


Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		1.017		1.4125		1.4365
	2		0.5		2.7798		0.9417		2.6176
	earth				51.892		1.4125		73.298
Upward base pressure									
	3		1		-6.9166		1.4125		-9.7697
	4		0.5		-3.8808		0.9417		-3.6544

Net bending moment at face of stem = 63.928 t-m/m
Effective depth required = 0.9058 m
Effective depth Provided at face of stem = 0.9175 m
Area of reinforcement required = 3726 mm^2
Minimum steel req = 1376.3 mm^2
Provide **25** ϕ **125** mm c/c = 3927 mm^2

Shear

Effective depth d of footing = 0.8825 m
Total depth of section at distance d from Abutment = 0.7138 m
Effective depth at distance d from face of Abutment = 0.6138 m



width b = 4.25

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		0.6977		0.969		0.6761
	2		0.5		1.907		0.646		1.2319
	earth				35.599		0.969		34.496
Upward base pressure									
	3		1		-6.9166		0.969		-6.7022
	4		0.5		-1.8264		0.646		-1.1798
					29.46				**28.52**

Effective depth at a distance of d eff = 0.6138 m
Shear at critical section = 29.46 t
Bending moment at critical section = 28.52 t-m
$\tan(\beta)$ = 0.423
Net shear force = 9.80 t
Shear stress = 15.967 t/m^2
% of reinforcement = 0.6397
Permissible shear stress = 31.411 t/m^2
NO SHEAR REIN FORCEMENT REQ

Dry Condition with L.L												
a) Vertical load and their moments about Abutment Shaft bottom												
II	Substructure	**V**		**ρ**	**P**		**e_L**	**M_L**		**e_T**		**M_T**
1	Pedestal											
a	Left span	0.0096		2.4	0.0231		0.225	0.0052		0		0
2	Abut cap R	1.7888		2.4	4.293		0	0		0		0
	Abut cap Tr	5.37		2.4	12.888		0	0		0		0
3	Abutment shaft											
	up to G.L	14.461		2.4	34.706		0	0		0		0
	below G.L	3.7401		2.4	8.9762		0	0		0		0
4	Earth pressure							280				
					60.886			**280**				**0**

b) Horizantal Forces and Moments withrespect to Abutment Shaft bottom										
1 Longitudinal Forces at bearing level										
	H_L		**H_T**		**e_L**	**M_L**		**e_T**		**M_T**
	6.94		0		8.669	60.149		8.644		0
	6.94		**0**			**60.149**				**0**

Summary *for design of Abutment*

P	=	179.81	t
M_L	=	367.35	t-m
M_T	=	52.492	t-m

H.F.L Condition with L.L

a) Vertical load and their moments about C/L of Foundation base.

I

		P	e_L	M_L	e_T	M_T
1	**D.L Reaction**					
a	Left span	58.000	0.225	13.050	0.000	0.000
2	**S.I.D.L**					
a	Left span	13.200	0.225	2.970	0.000	0.000
		71.200		***16.020***		***0.000***
3	**L.L**					
a	Left span					
	70R Wheeled	47.720	0.225	10.737	1.100	52.492
	Class A 2 Lane	49.480	0.225	11.133	0.625	30.925
		118.920		**26.757**		**52.492**

II	Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1	Pedestal							
a	Left span	0.010	2.400	0.023	0.225	0.005	0.000	0.000
2	Abut cap R	2.385	2.400	5.724	0.000	0.000	0.000	0.000
	Abut cap Tr	5.370	2.400	12.888	0.000	0.000	0.000	0.000
3	Abut shaft							
	up to H.F.L	0.000	2.400	0.000	0.000	0.000	0.000	0.000
	below H.F.L	18.201	1.400	25.481	0.000	0.000	0.000	0.000
4	Footing	20.043	1.400	28.060	0.000	0.000	0.000	0.000
5	Earth above footing	51.338	1.000	51.338	0.000	0.000	0.000	0.000
6	Earth Pressure					230.383		
7	Earth wt on heel side	83.277	1.000	83.277	-2.213	-184.29		
				123.515		**46.096**		**0.000**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L	H_T	e_L	M_L	e_T	M_T
	6.938	0.000	9.639	66.879	9.614	0.000
	6.938	0.000		**66.879**		**0.000**

Summary

P	=	242.435	t
M_L	=	139.732	t-m
M_T	=	52.492	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{242.435}{30.813} + \frac{139.732}{37.232} + \frac{52.492}{21.826} = 14.026 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{242.435}{30.813} - \frac{139.732}{37.232} - \frac{52.492}{21.826} = 1.710 \quad t/m^2$$

H.F.L Condition with L.L

a) Vertical load and their moments about Abutment Shaft bottom

II Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1 Pedestal							
a Left span	0.010	2.400	0.023	0.225	0.005	0.000	0.000
2 Abut cap R	1.789	2.400	4.293	0.000	0.000	0.000	0.000
Abut cap Tr	5.370	2.400	12.888	0.000	0.000	0.000	0.000
3 Abut shaft							
up to H.F.L	0.000	2.400	0.000	0.000	0.000	0.000	0.000
below H.F.L	18.201	1.400	25.481	0.000	0.000	0.000	0.000
			42.685		**0.005**		**0.000**

b) Horizantal Forces and Moments withrespect to Abutment Shaft bottom

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	6.938	0.000	8.669	60.149	8.644	0.000
Earth pressure				179.197		
	6.938	0.000		239.346		0.000

Summary for design of Abutment shaft

P	=	161.605	t
M_L	=	266.108	t-m
M_T	=	52.492	t-m

Summary of Loads at Abutment Shaft bottom :

1 DRY condition	**P**		**M_L**		**M_T**
With L.L	179.81		367.35		52.49
2 H.F.L					
With L.L	161.61		266.11		52.49

Dimensions of Substructure & Foundation

1	**Pedestal**											
		Length					=	0.7				
		Width					=	0.55		Volume	=	0.00963
		Height					=	0.025				
2	**Pier cap**											
a	*Top uniform portion*											
		Width					=	1.00				
		Depth					=	0.225		Volume	=	1.78875 m^3
		Length					=	7.95				
b	*Top uniform portion*											
		Width					=	1.00				
		Depth					=	0.075		Volume	=	0.59625 m^3
		Length					=	7.95				
c	*Bottom trapezoidal portion*											
		Width					=	1.00				
		Depth					=	1.2		Volume	=	5.37 m^3
		Length					=	7.95				
	Area at level	215.148 m	=	7.95	x	1	=	7.95	m^2			
	Area at level	213.948 m	=	1	x	1	=	1	m^2		=	**7.755** m^3
	H.F.L											
	Pier cap						=	1.78875	m^3			
							=	0.59625	m^3			
							=	5.37	m^3			
3	**Abutment shaft**											
	Area at abutment shaft bottom						=	3.45062	m^2			
	Area at abutment shaft top						=	1.64485	m^2			
	Average area						=	2.54773	m^2			
	Height of Abutment shaft						=	7.14				
	Height above Ground level						=	5.676	m	Volume	=	14.4609
	Height below Ground level						=	1.468	m	Volume	=	3.74007
											=	**18.201** m^3
	Height above H.F.L						=	0.000	m	Volume	=	0
	Height below H.F.L						=	7.14	m	Volume	=	18.201
											=	**18.201** m^3
4	**Pedestal at footing top**			Width			=	1.6				
				Length			=	2.50		Volume	=	0 m^3
				Height			=	0				

5 **Footing**

at foundation level

Width	=	7.250	m
Length	=	4.25	m
Thickness at Root	=	0.97	m
Thickness at Tip	=	0.15	m
	=	20.043	m^3

Volume of Overburdden earth below ground level

Total volume 7.250 x 4.250 x 2.438	=	75.1209	m^3
Net volume below Ground level	=	51.3378	m^3
Wt of earth on Heel side above ground level	=	83.2769	t

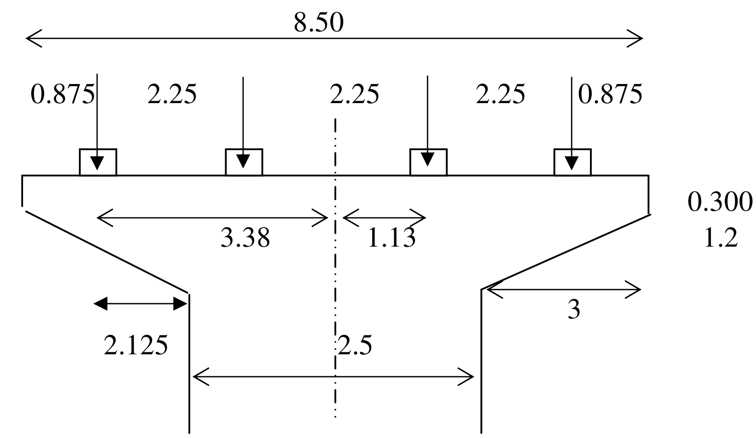
Sectional Properties of Footing

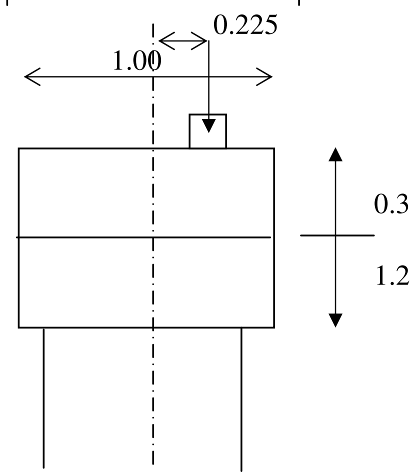
$A = 7.250 \times 4.250 = 30.813\ m^2$

$Z_L = 4.250 \times 8.76042 = 37.232\ m^3$

$Z_T = 7.250 \times 3.01042 = 21.826\ m^3$

DESIGN OF ABUTMENT CAP:

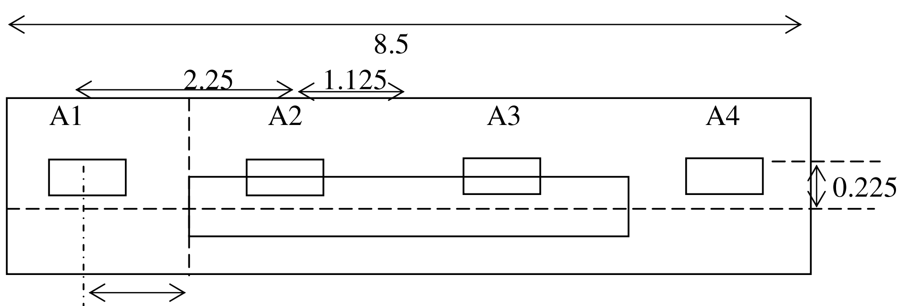




For Outer bearing	a	=	2.125
	D	=	1.50
Available effective depth			
using 25 mm dia of bars	d	=	1.5 - 0.05 - 0.025
		=	1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= **Pier cap designed as Cantilever**



Bearings A1 & A4 are effective.

	A1	
DL	14.5	
SIDL	3.3	
LL		
70RW	11.93	

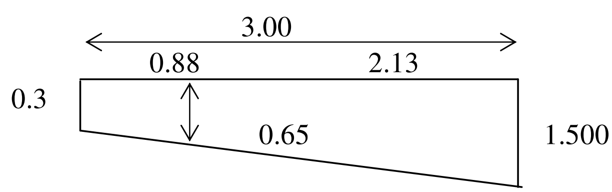
due to Transverse moment A1 = 47.72 x 1.16 = 55.355

$$A1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	A1
DL	14.5
SIDL	3.3
LL	7.3807
	11.93
	37.111



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 37.11 x 2.13 = 78.860223 t-m

Due to selfweight of Abutment cap = 6.48 x 1.1667 = 7.56 t-m

Total moment at the face of support = **86.42** t-m

Torsion at outerbearing section:

= 37.111

Eccentricity = 0.225

Moment = 8.3499 t-m

Longitudinal Reinforcement:

M = 86.42 = 86.42 t-m

For, M - 25 & Fe - 415

σcbc = 8.33 N/mm^2

m = 10

σst = 200 N/mm^2

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \frac{86.42 \times 10^7}{1.105 \times 1000}$

= 884.22 < 1425 O.K

A_{st} required $= \frac{86.42}{200 \times 0.902 \times 1425}$

= 3361.8851 mm^2

Provide 6 nos 20 mm dia bars in 2 layers 3769.9 mm^2

This reinforcement will provided in full length of Abutment cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 37.111

= 37.111 t

Downward Load due to self wt of Abutment cap = 6.48 t

Total downward Load = 43.591 t

After shear correction, S.F $= V - \frac{M \tan\beta}{d}$

$= 43.591 - \frac{86.42 \times 0.4}{1.425}$

= 19.332385 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

$b = \frac{1 + 1}{2}$ = 1 m

$V_e = V + 1.6 \times \frac{T}{b}$ = 32.692 t

Shear stress $= \frac{32.692 \times 10000}{1000 \times 1425}$ = 0.2294 N/mm^2

Area of tension reinforcement = 0.2646 %

τ_c = 0.2671 N/mm^2

Provide minimum shear

EARTH PRESSURE CALCULATION

Formation level	=	216.189 m
Founding level	=	205.834 m
Low Water Level	=	208.272 m
Highest flood level	=	214.172 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb's theory of active earth pressure

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

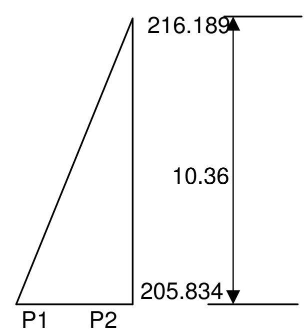
Here	Angle of internal friction,	ϕ	=	30 ^{o}
	Angle of friction between soil and concrete,	δ	=	20 ^{o}
	Surcharge angle	i	=	0 ^{o}
	Angle of wall face with horizontal	α	=	90 ^{o}
	Bulk density of earth	γ	=	1.8 t/m^3
	Submerged density of earth	γ_{sub}	=	1.0 t/m^3
	Width of abutment		=	2.50 m

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

* (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 10.36 = 5.5 t/m^2

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

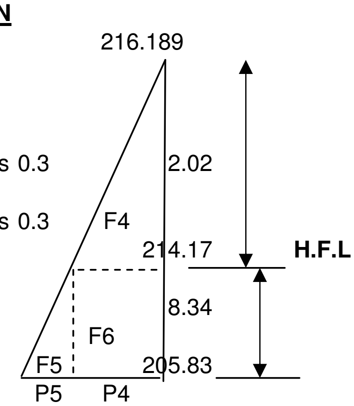
F1 = 0.5 x 5.542 x 10.36 x 2.5 x cos 0.3

= 67.404 t say 68 t

M = 67.404 x 0.42 x 10.36

= 293 t-m

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION



P4 = 0.2973 x 1.8 x 2.02 = 1.1 t/m^2

P5 = 0.2973 x 1.0 x 8.34 = 2.5 t/m^2

F4 = 0.5 x 1.079 x 2.02 x 2.5 x cos 0.3

= 2.5574 t

F5 = 0.5 x 2.479 x 8.34 x 2.5 x cos 0.3

= 24.28 t

F6 = 1.08 x 8.34 x 2.5 x cos 0.3

= 21.14 t

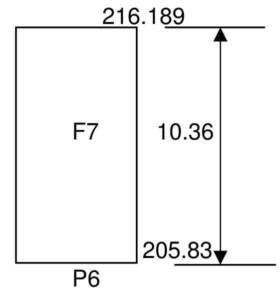
Total force, F = 2.5574 + 24.28 + 21.14

= 48 t

M = 2.5574 x (0.42 x 2.02 + 8.34)

+ 24.28 x 0.33 x 8.34 + 21.1 x $\frac{8.34}{2}$

= 178 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	10.36	x	3	x	cos 0.3				
	=	16 t										
M	=	16	x	$\frac{9.75}{2}$	=	76 t-m						

H.F.L

P6	=	0.2973	x	1.2	x	1.0	=	0.36	t/m^2			
F7	=	0.6422	x	2.02	x	3	x	cos 0.3		=	3.043011	t
	=	0.3568	x	8.34	x	3	x	cos 0.3		=	6.988548	t
										=	10.03156	
M	=	3.043	+	6.988548147	x	5.1775				=	51.9384	tm

EARTH PRESSURE CALCULATION

Formation level	=	216.189 m
Abutment shaft bottom level	=	206.804 m
Low Water Level	=	208.272 m
Highest flood level	=	214.172 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		=	2.50 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 9.38 = 5.02 t/m^2

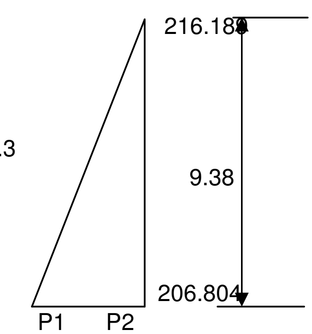
P2 = 0.2973 x 1.0 x 0.00 = 0 t/m^2

F1 = 0.5 x 5.023 x 9.38 x 2.5 x cos 0.3

= 55.367 t say 56 t

M = 55.367 x 0.42 x 9.38

= 218 t-m



CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

P4 = 0.2973 x 1.8 x 2.02 = 1.08 t/m^2

P5 = 0.2973 x 1.0 x 7.37 = 2.19 t/m^2

F4 = 0.5 x 1.079 x 2.02 x 2.5 x cos 0.3

= 2.5574 t

F5 = 0.5 x 2.191 x 7.37 x 3 x cos 0.3

= 18.96 t

F6 = 1.08 x 7.37 x 2.5 x cos 0.3

= 18.68 t

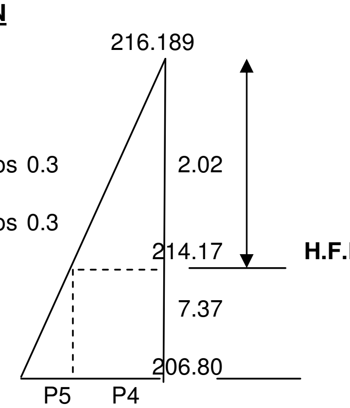
Total force, F = 2.5574 + 18.96 + 18.68

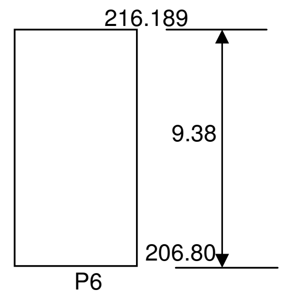
= 40 t

M = 2.5574 x (0.42 x 2.02 + 7.37)

+ 18.96 x 0.33 x 7.37 + 18.7 x $\frac{7.37}{2}$

= 136 t-m





<u>CALCULATION OF LIVE LOAD SURCHARGE</u>

Dry

P6	=	0.2973	x	1.2	x	1.8	=	0.64	t/m^2				
F7	=	0.64	x	9.38	x	3	x	cos	0.3				
	=	14 t											
M	=	14	x	$\frac{8.78}{2}$	=	62 t-m							

H.F.L

P6	=	0.2973	x	1.2	x	1.0	=	0.36	t/m^2				
F7	=	0.6422	x	2.02	x	3	x	cos	0.3	=	3.043011	t	
	=	0.3568	x	7.37	x	3	x	cos	0.3	=	6.175536	t	
										=	9.218547		
M	=	3.043	+	6.175536429	x	4.6925				=	43.25803	tm	

```
INPUT FILE: 9.2 M SPAN DL.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;3 0.4 0.0 0.0 10 8.4 0.0 0.0;11 8.8 0.0 0.0
     9. 12 9.18 0.0 0.0;139.22 0.0 0.0;14 9.6 0.00.0 22 18 0.00.0;2318.380.0 0.0
    10. 24 18.4 0.0 0.0
    11. REPEAT ALL 1 0.0 0.0 1.0
    12. REPEAT 3 0.0 0.0 2.0
    13. REPEAT 1 0.0 0.0 1.0
    14. MEMBER INCIDENCES
    15. 1 1 2 23 1 1
    16. REPEAT 5 23 24
    17. 139 1 25 143 1 24
    18. REPEAT 23 5 1
    19. CONSTANTS
    20. E 2.7386E+006
    21. POISSON 0.15
    22. DENSITY 2.4 MEMB  1 TO 138
    23. DENSITY 0.0001 MEMB 139 TO 258
    24. MEMBER PROPERTY INDIAN
    25. 2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
    26. 25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
    27. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
    28. 1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
    29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    31. SUPPORTS
    32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
    33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
    34. 35 59 83 107 PINNED
    35. 46 70 94 118 PINNED
    36. MEMBER RELEASE
    37. 1 24 47 93 116 END MZ
    38. 23 46 92 115 138 START MZ
    39. 12 35 58 81 104 127 START MZ
    40. 12 35 58 81 104 127 END MZ

41. LOAD 1 SELFWEIGHT
    42. SELF Y -1
    43. PERFORM ANALYSIS
    44. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70
94 118
```

SUPPORT REACTION LIST 27

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = FLOOR

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
27	1	0.00	16.77	0.00	0.00	0.00	0.00
51	1	0.00	12.24	0.00	0.00	0.00	0.00
75	1	0.00	12.24	0.00	0.00	0.00	0.00
99	1	0.00	16.77	0.00	0.00	0.00	0.00
38	1	0.00	16.42	0.00	0.00	0.00	0.00
62	1	0.00	12.60	0.00	0.00	0.00	0.00
86	1	0.00	12.60	0.00	0.00	0.00	0.00
110	1	0.00	16.42	0.00	0.00	0.00	0.00
35	1	0.00	16.58	0.00	0.00	0.00	0.00
59	1	0.00	12.44	0.00	0.00	0.00	0.00
83	1	0.00	12.44	0.00	0.00	0.00	0.00
107	1	0.00	16.58	0.00	0.00	0.00	0.00
46	1	0.00	16.86	0.00	0.00	0.00	0.00
70	1	0.00	12.16	0.00	0.00	0.00	0.00
94	1	0.00	12.16	0.00	0.00	0.00	0.00
118	1	0.00	16.86	0.00	0.00	0.00	0.00

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

45. FINISH

```
INPUT FILE: 9.2 M SPAN SIDL.STD
STAAD FLOOR
START JOB INFORMATION
ENGINEER DATE 09-JUN-06
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER MTON
JOINT COORDINATES
1 0.0 0.0 0.0;2 0.02 0.0 0.0;3 0.4 0.0 0.0 10 8.4 0.0 0.0;11 8.8 0.0 0.0
12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22180.00.0;2318.380.0 0.0
24 18.4 0.0 0.0
REPEAT ALL 1 0.0 0.0 1.0
REPEAT 3 0.0 0.0 2.0
REPEAT 1 0.0 0.0 1.0
MEMBER INCIDENCES
1 1 2 23 1 1
REPEAT 5 23 24
139 1 25 143 1 24
REPEAT 23 5 1
CONSTANTS
E 2.7386E+006
POISSON 0.15
DENSITY 2.4 MEMB  1 TO 138
DENSITY 0.0001 MEMB 139 TO 258
MEMBER PROPERTY INDIAN
2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
SUPPORTS
27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
35 59 83 107 PINNED
46 70 94 118 PINNED
MEMBER RELEASE
1 24 47 93 116 END MZ
23 46 92 115 138 START MZ
12 35 58 81 104 127 START MZ
12 35 58 81 104 127 END MZ
LOAD 1 SIDL
*** CRASH BARRIER
MEMBER LOAD
25 TO 45 UNI GY -1.0
94 TO 114 UNI GY -1.0
25 TO 45 UMOM GX -1.0
94 TO 114 UMOM GX 1.0
LOAD 2 WEARINGCOAT
MEMBER LOAD
25 TO 114 UNI GY -0.2464
PERFORM ANALYSIS
```

```
    52. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70
94 118
  SUPPORT  REACTION LIST     27
     STAAD FLOOR                                                  -- PAGE NO.    3

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

     27    1      0.00      7.42      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     51    1      0.00     -2.84      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     75    1      0.00     -2.84      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     99    1      0.00      7.42      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     38    1      0.00      6.97      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     62    1      0.00     -2.37      0.00      0.00      0.00      0.00
           2      0.00      1.14      0.00      0.00      0.00      0.00
     86    1      0.00     -2.37      0.00      0.00      0.00      0.00
           2      0.00      1.14      0.00      0.00      0.00      0.00
    110    1      0.00      6.97      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     35    1      0.00      7.19      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     59    1      0.00     -2.59      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     83    1      0.00     -2.59      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
    107    1      0.00      7.19      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     46    1      0.00      7.52      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     70    1      0.00     -2.94      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     94    1      0.00     -2.94      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
    118    1      0.00      7.52      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00


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


    53. FINISH
```

```
INPUT FILE: 70RW.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2 3    PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    20. LOAD GENERATION 175
    21. TYPE 1 -13.4 0. 0. XINC .2
    22. PERFORM ANALYSIS
    23. LOAD LIST 30
    24. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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

      2   30      0.00     47.72      0.00      0.00      0.00      0.00
      3   30      0.00     20.28      0.00      0.00      0.00      0.00


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


    25. FINISH
```

```
INPUT FILE: CLASS A.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2 3    PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    20. LOAD GENERATION 100
    21. TYPE 1 -18.8 0. 0. XINC .2
    22. PERFORM ANALYSIS
    23. LOAD LIST 74
    24. PRINT SUPPORT REACTION
        SUPPORT  REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


     2   74      0.00     24.74      0.00      0.00      0.00      0.00
     3   74      0.00     11.66      0.00      0.00      0.00      0.00


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


    25. FINISH
```

STABILITY CHECK & DESIGN OF SUBSTRUCTURE (PIER)

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete		=	M - 20	
	Pier Cap	=	M - 25	
Grade of reinforcement steel		=	Fe - 415	
Centre to Centre distance between Expansion joints		=	9.200	m
Centre to Centre distance of Bearing		=	8.800	m
Depth of superstructure		=	660	mm
Thickness of wearing coat		=	56.00	mm
Formation level along ℄ of carriage way		=	216.189	m
Soffit level		=	215.423	
Pedestal top level		=	215.473	
Height of bearing and Pedestal		=	0.050	m
L.W.L./Bed level		=	208.272	m
H.F.L		=	214.172	m
M.S.L		=	205.452	m
Founding Level		=	205.834	m
Pier cap top level		=	215.423	m

Live Load **(a)** Class A two Lane
(b) Class 70R wheeled

Bearing : neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:

1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000

Longitudinal translation due to creep, shrinkage & temperature = 0.0005
Horizontal movement = 0.0005 x 4.60 x 1000 = 2.300 mm

Longitudinal Force :

Size of bearing = 400 x 250 x 50 mm

Strain in bearing = $\frac{2.300}{50}$ = 0.046

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.046 x 1.0 x 400 x 250 = 4600 N
= 0.469 t

Total shear force for 4 bearings (with 5 % increase)
= 0.469 x 4 x 1.05 = 1.969 t

Refer IRC : 6 clause 214.5.1.5;
10 % increase for variation in movement of span

Total shear force = 1.1 x 1.969 = 2.166 t

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

For Class A 2 lane : $\mathbf{F_h =}$ [0.2 x 59.2] = 11.836 t

For class 70R wheeled : $\mathbf{F_h =}$ [0.2 x 66.1] = 13.210 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 2 lane	**8.08**
70R	**8.77**
Span dislodged condition	**2.17**

Dry Condition with L.L											
a) Vertical load and their moments about C/L of Foundation base.											
I											
1 D.L Reaction	**P**		**e_L**		**M_L**		**e_T**		**M_T**		
a Left span	58.00		-0.4		-23.2		0		0		
b Right span	58		0.4		23.2		0		0		
2 S.I.D.L											
a Left span	13.68		-0.4		-5.472		0		0		
b Right span	13.68		0.4		5.472		0		0		
	143.36				**0**				**0**		
4 **L.L Max Reaction case**											
a Left span											
70R Wheeled + Class A	36		-0.4		-14.4		1.1		39.6		
Class A 3 Lane	33.6		-0.4		-13.44		0.625		21		
b Right span											
70R Wheeled + Class A	30.05		0.4		12.02		1.1		33.055		
Class A 3 Lane	25.58		0.4		10.232		0.625		15.9875		
4 **L.L Max Longitudinal Moment case**											
a Left span											
70R Wheeled + Class A	0		-0.4		0		1.1		0		
Class A 3 Lane	6.88		-0.4		-2.752		0.625		4.3		
b Right span											
70R Wheeled + Class A	47.15		0.4		18.86		1.1		51.865		
Class A 3 Lane	47.74		0.4		19.096		0.625		29.8375		
Max Reaction case	**209.41**				**1.42**				**72.655**		
Max Longitudinal Moment	**190.51**				**18.86**				**51.865**		

II Substructure	V		ρ	P		e_L	M_L		e_T	M_T
1 Pedestal										
a Left span	0		2.4	0		-0.4	0		0	0
b Right span	0		2.4	0		0.4	0		0	0
2 Pier cap R	2.385		2.4	5.724		0	0		0	0
Pier cap Tr	5.37		2.4	12.888		0	0		0	0
3 Pier shaft										
up to G.L	10.76912		2.4	25.846		0	0		0	0
below G.L	2.8356842		2.4	6.8056		0	0		0	0
4 Footing	11.73375		2.4	28.161		0	0		0	0
5 Earth above footing	34.826681		2	69.653		0	0		0	0
				149.08			**0**			**0**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L 8.77	**H_T** 0	**e_L** 9.639	**M_L** 84.547	**e_T** 9.589	**M_T** 0
	8.77	0		**84.547**		**0**

Summary

		Max Reaction		Max Longitudinal
P	=	358.49	t	339.59
M_L	=	85.967	t-m	103.41
M_T	=	72.655	t-m	51.865

A		=	20.3
Z_L		=	19.623
Z_T		=	11.842

Max reaction

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{358.48789}{20.3} + \frac{85.967}{19.623} + \frac{72.655}{11.842} = 28.1759 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{358.48789}{20.3} - \frac{85.967}{19.623} - \frac{72.655}{11.842} = 7.1431 \quad t/m^2$$

Max Long

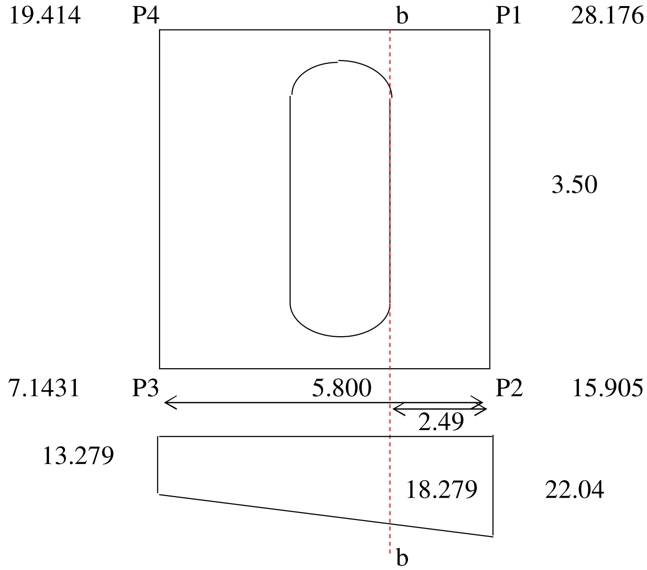
$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{339.58789}{20.3} + \frac{103.41}{19.623} + \frac{51.865}{11.842} = 26.37794 \quad t/m^2$$

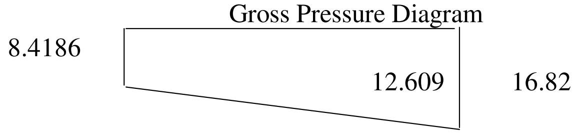
$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{339.58789}{20.3} - \frac{103.41}{19.623} - \frac{51.865}{11.842} = 7.078993 \quad t/m^2$$

Design of Footing in Transverse direction

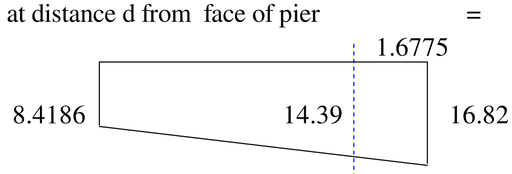


Gross Pressure Diagram



Net Pressure diagram b

Bending moment at the face of pier	b-b	=	**167.27**	t-m
Footing is checked for shear at distance of d from face pier				
Effective depth d of footing		=	0.8125	m
Total depth of section at distance d from pier		=	0.6553	m
Effective depth at distance d from face of pier		=	0.5643	m



Bending moment at a distance of d from face of pier	=	36.54	t-m
Shear force at a distance d from face of pier	=	91.623	t
Net shear force at a distance d from face of pier (after correction)	=	**72.118**	t

Design for Flexure:

M = 20 Fe = 415

Permissible compressive stress	σ_{cbc}	=	6.67	N/mm^2
Permissible tensile stress	σ_{st}	=	200	N/mm^2
	m	=	10	
	k	=	0.25	
	j	=	0.92	
	Q	=	0.7639	

width　b = 3.50

Effective depth required　d　= 790.97 mm
Effective depth provided　= 812.5 mm
A_{st} Required　= 3208.4 mm^2

Provide 7 nos 25 dia bars /m width

Check for shear

Shear stress at effective depth from pier　= 36.517 t/m^2　0.358005
% of steel provided　= 0.3565
Permissible shear stress　= 25.5 t/m^2　0.25

Area of shearreinforcement　IRC:21-2000,cl:304.7.1.4

$$A_{sw} = \frac{V - (\tau_c \times b \times d) \times s}{\sigma_s \times d}$$

= 403.08 mm^2

Minimum Shear Reinforcement :

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

$$\frac{A_{sw}}{s} = \frac{0.4 \times b}{0.87 \times f_y} = 3.8776 \ mm^2/mm$$

Provide 2 Legged 12 @ 200 = 1131

Dry Condition with L.L										
a) Vertical load and their moments about Pier Shaft bottom										
II Substructure	**V**		**ρ**	**P**		**e_L**	**M_L**		**e_T**	**M_T**
1 Pedestal										
a Left span	0		2.4	0		-0.4	0		0	0
b Right span	0		2.4	0		0.4	0		0	0
2 Pier cap R	2.385		2.4	5.724		0	0		0	0
Pier cap Tr	5.37		2.4	12.888		0	0		0	0
3 Pier shaft										
up to G.L	10.76912		2.4	25.846		0	0		0	0
below G.L	2.8356842		2.4	6.8056		0	0		0	0
				51.264			**0**			**0**

b) Horizantal Forces and Moments withrespect to PierShaft bottom

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	8.77	0	8.689	76.214	8.639	0
	8.77	**0**		**76.214**		**0**

Summary	*for design of Pier*		Max Reaction case		Max Longitudinal moment
	P	=	260.67	t	241.77
	M_L	=	77.634	t-m	95.074
	M_T	=	72.655	t-m	51.865

Dry Condition One.Span dislodged

a) Vertical load and their moments about C/L of Foundation base.

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	2.17	0	9.639	20.882	9.589	0
	2.17	0		**20.882**		**0**

Summary

P	=	292.44	t
M_L	=	20.882	t-m
M_T	=	0	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{292.43789}{20.3} + \frac{20.882}{19.623} + \frac{0}{11.842} = 15.46993 \text{ t/m}^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{292.43789}{20.3} - \frac{20.882}{19.623} - \frac{0}{11.842} = 13.34169 \text{ t/m}^2$$

Dry Condition One.Span dislodged

a) Vertical load and their moments about Abutment Shaft bottom.

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	2.17	0	8.689	18.824	8.639	0

Summary *for design of Pier*

P	=	194.62	t
M_L	=	18.824	t-m
M_T	=	0	t-m

H.F.L Condition with L.L						
a)	*Vertical load and their moments about C/L of Foundation base.*					
I						
1.000	**D.L Reaction**	**P**	e_L	M_L	e_T	M_T
a	Left span	58.000	-0.400	-23.200	0.000	0.000
b	Right span	58.000	0.400	23.200	0.000	0.000
2.000	**S.I.D.L**					
a	Left span	13.680	-0.400	-5.472	0.000	0.000
b	Right span	13.680	0.400	5.472	0.000	0.000
		143.360		***0.000***		***0.000***
3.000	L.L Max Reaction case					
a	Left span					
	70R Wheeled	36.000	-0.400	-14.400	1.100	39.600
	Class A 2 Lane	33.600	-0.400	-13.440	0.625	21.000
b	Right span					
	70R Wheeled	30.050	0.400	12.020	1.100	33.055
	Class A 2 Lane	25.580	0.400	10.232	0.625	15.988
4.000	L.L Max Long.Moment case					
a	Left span					
	70R Wheeled	0.000	-0.400	0.000	1.100	0.000
	Class A 2 Lane	6.880	-0.400	-2.752	0.625	4.300
b	Right span					
	70R Wheeled	47.150	0.400	18.860	1.100	51.865
	Class A 2 Lane	47.740	0.400	19.096	0.625	29.838

Max Reaction case:	**209.410**	**1.420**	**72.655**
Max Long. moment case:	**190.510**	**18.860**	**51.865**

II	Substructure	**V**	**ρ**	**P**	e_L	M_L	e_T	M_T
1.000	Pedestal							
a	Left span	0.000	2.400	0.000	-0.400	0.000	0.000	0.000
b	Right span	0.000	2.400	0.000	0.400	0.000	0.000	0.000
2.000	Pier cap R	2.385	2.400	5.724	0.000	0.000	0.000	0.000
	Pier cap Tr	5.370	2.400	12.888	0.000	0.000	0.000	0.000
3.000	Pier shaft							
	up to H.F.L	0.000	2.400	0.000	0.000	0.000	0.000	0.000
	below H.F.L	13.605	1.400	19.047	0.000	0.000	0.000	0.000
4.000	Footing	11.734	1.400	16.427	0.000	0.000	0.000	0.000
5.000	Earth above footing	34.827	1.000	34.827	0.000	0.000	0.000	0.000
6.000	Water current			0.000	0.000	16.274	0.000	6.612
				88.913		**16.274**		**6.612**

b)	*Horizantal Forces and Moments withrespect to Base*					
1.000 Longitudinal Forces at bearing level						
	H_L	H_T	e_L	M_L	e_T	M_T
	8.771	0.000	9.639	84.547	9.589	0.000
	8.771	0.000		**84.547**		**0.000**

Summary

		Max Reaction case		Max Long.moment case
P	=	298.323	t	279.423
M_L	=	102.241	t-m	119.681
M_T	=	79.267	t-m	58.477

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{298.323}{20.300} + \frac{102.241}{19.623} + \frac{79.267}{11.842} = 26.600 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{298.323}{20.300} - \frac{102.241}{19.623} - \frac{79.267}{11.842} = 2.792 \quad t/m^2$$

Max Long

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{279.423}{20.300} + \frac{119.681}{19.623} + \frac{58.477}{11.842} = 24.802 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{279.423}{20.300} - \frac{119.681}{19.623} - \frac{58.477}{11.842} = 2.727 \quad t/m^2$$

H.F.L Condition with L.L

a) Vertical load and their moments about Pier Shaft bottom

II	Substructure	**V**	ρ	**P**		**e_L**	**M_L**		**e_T**		**M_T**
1.000	Pedestal										
a	Left span	0.000	2.400	0.000		-0.400	0.000		0.000		0.000
b	Right span	0.000	2.400	0.000		0.400	0.000		0.000		0.000
2.000	Pier cap R	2.385	2.400	5.724		0.000	0.000		0.000		0.000
	Pier cap Tr	5.370	2.400	12.888		0.000	0.000		0.000		0.000
3.000	Pier shaft										
	up to H.F.L	0.000	1.400	0.000		0.000	0.000		0.000		0.000
	below H.F.L	13.605	1.400	19.047		0.000	0.000		0.000		0.000
4.000	Water current						6.438				16.236
				37.659			**6.438**				**16.236**

b) Horizantal Forces and Moments withrespect to Pier Shaft bottom

1.000 Longitudinal Forces at bearing level										
	H_L		**H_T**		**e_L**		**M_L**	**e_T**		**M_T**
	8.771		0.000		8.689		76.214	8.639		0.000
	8.771		0.000				76.214			0.000

Gross Pressure Diagram

Summary	*for design of Pier*		Max Reaction case			Max Long.moment case	
	P	=	247.069	t		228.169	t
	$\mathbf{M_L}$	=	84.072	t-m		101.512	t-m
Net P	$\mathbf{M_T}$		88.891	t-m		68.101	t-m

H.F.L Condition One.Span dislodged

a) Vertical load and their moments about C/L of Foundation base.

II	Substructure	V		ρ	P		e_L	M_L		e_T		M_T
1.000	Pedestal											
a	Left span	0.000		2.400	0.000		-0.400	0.000		0.000		0.000
b	Right span	0.000		2.400	0.000		0.400	0.000		0.000		0.000
2.000	Pier cap R	2.385		2.400	5.724		0.000	0.000		0.000		0.000
	Pier cap Tr	5.370		2.400	12.888		0.000	0.000		0.000		0.000
3.000	Pier shaft											
	up to H.F.L	0.000		2.400	0.000		0.000	0.000		0.000		0.000
	below H.F.L	13.605		1.400	19.047		0.000	0.000		0.000		0.000
4.000	Footing	11.734		1.400	16.427		0.000	0.000		0.000		0.000
5.000	Earth above footing	34.827		1.000	34.827		0.000	0.000		0.000		0.000
6.000	Water current				0.000		0.000	16.274		0.000		6.612
					88.913			**16.274**				**6.612**

Summary				
	P	=	232.273	t
	$\mathbf{M_L}$	=	16.274	t-m
	$\mathbf{M_T}$	=	6.612	t-m

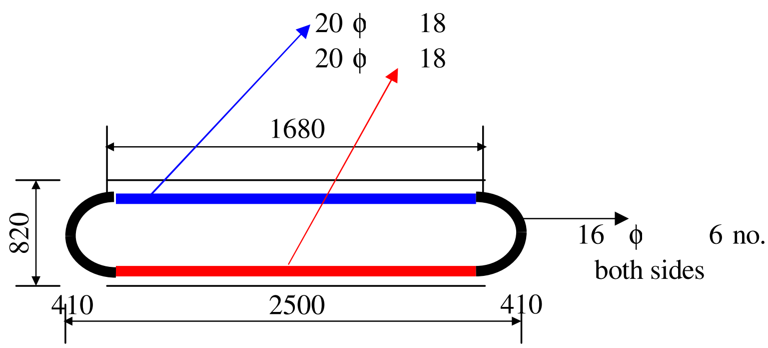
Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{232.273}{20.300} + \frac{16.274}{19.623} + \frac{6.612}{11.842} = 12.830 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{232.273}{20.300} - \frac{16.274}{19.623} - \frac{6.612}{11.842} = 10.054 \quad t/m^2$$

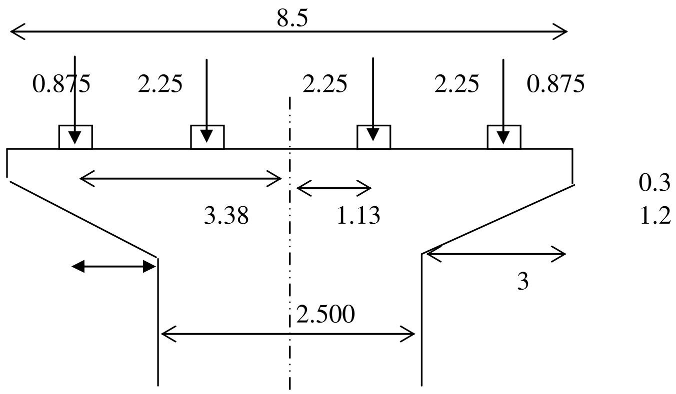


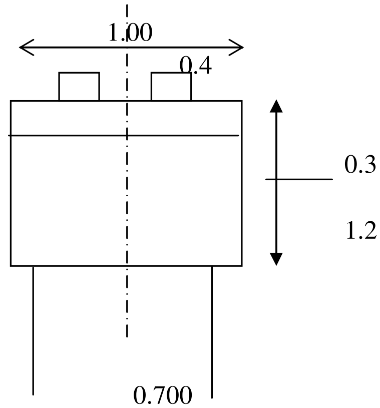
PLAN : PIER SHAFT

Reinforcement provided = 13722.477

Summary of Loads at Pier Shaft bottom :

	P	M_L	M_T
1 DRY condition			
With L.L	260.67	77.63	72.66
Span dislodged	194.62	18.82	0.00
2 H.F.L			
With L.L	247.07	84.07	88.89
Span dislodged	181.02	6.44	16.24





For Outer bearing a = 2.125

D = 1.5

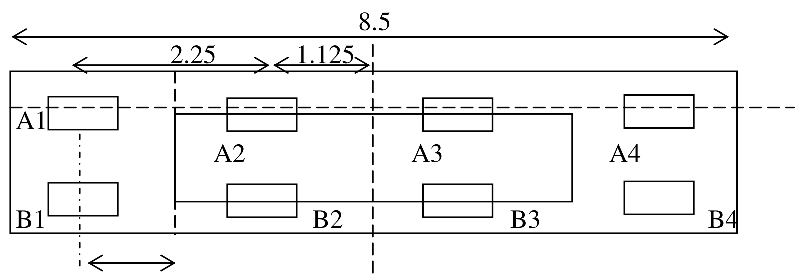
Available effective depth

using 25 mm dia of bars d = 1.5 - 0.05 - 0.025

= 1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= Pier cap designed as Cantilever



Loads on bearing

Bearings A1 & B1 are effective.

	A1	B1
DL	14.5	14.5
SIDL	3.3	3.3
LL		
70RW	7.5125	9

due to Transverse moment A1 = 30.05 x 1.16 = 34.858
B1 = 36 x 1.16 = 41.76

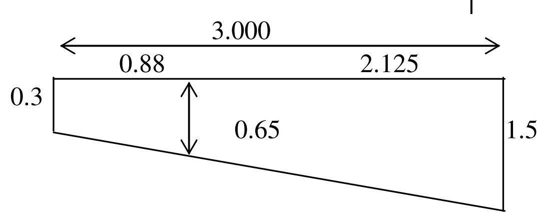
$$A1 = \frac{34.858 \times 3.38}{25.31} = 4.6477 \text{ t}$$

$$B1 = \frac{41.76 \times 3.38}{25.31} = 5.568 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	Left span			Right span	
	B1			A1	
DL	14.5			14.5	
SIDL	3.3			3.3	
LL	5.568			4.6477	
	9			7.5125	
	32.368			**29.96**	



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 62.328 x 2.125

= 132.4475 t-m

Due to selfweight of pier cap = 6.48 x 1.1667 = 7.56 t-m

Total moment at the face of support = 140.01 t-m

Torsion at outerbearing section:

= 32.368 - 29.96

= 2.4078

Eccentricity = 0.4

Moment = 0.9631 t-m

Longitudinal Reinforcement:

M = 140.01 + = 140.01 t-m

For, M - 25 & Fe - 415

σcbc = 8.33 N/mm²

m = 10

σst = 200 N/mm²

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \dfrac{140.01 \times 10^{7}}{1.105 \times 1000}$

= 1125.5 < 1425 O.K

A_{st} required $= \dfrac{140.01}{200 \times 0.902 \times 1425}$

= 5446.5159 mm^2

Provide 6 nos 25 mm dia bars in 2 layers 5890.5 mm^2

This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 32.368 + 29.96

= 62.328 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 68.808 t

After shear correction, S.F $= V + \dfrac{M \tan\beta}{d}$

$= 68.808 + \dfrac{140.01 \times 0.4}{1.425}$

= 29.507884 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

b $= \dfrac{1 + 1}{2}$ = 1 m

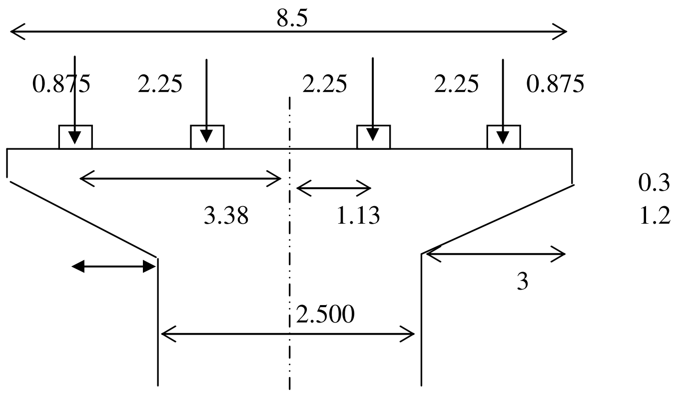
V = 29.508 t

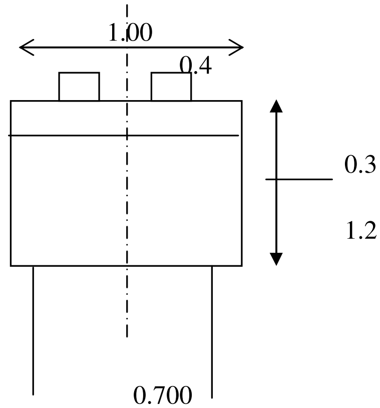
Shear stress $= \dfrac{29.508 \times 10000}{1000 \times 1425}$ = 0.2071 N/mm²

Area of tension reinforcement = 0.4134 %

τ_c = 0.2873 N/mm²

Provide minimum shear

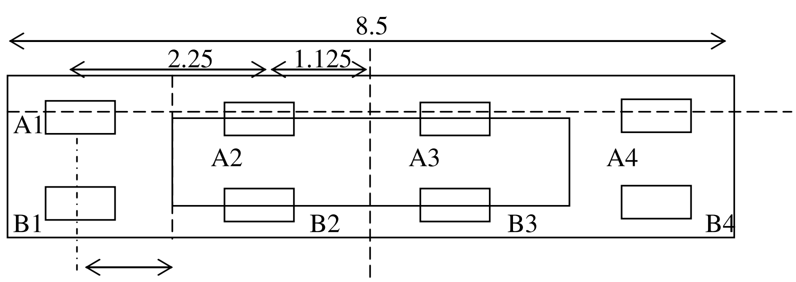




For Outer bearing	a	=	2.125
	D	=	1.5
Available effective depth			
using 25 mm dia of bars	d	=	1.5 - 0.05 - 0.025
		=	1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= Pier cap designed as Cantilever



Loads on bearing

Bearings A1 & B1 are effective.

	A1	B1
DL	14.5	14.5
SIDL	3.3	3.3
LL		
70RW	0	11.93

due to Transverse moment A1 = 0 x 1.16 = 0

B1 = 47.72 x 1.16 = 55.355

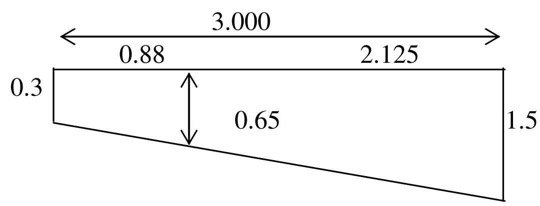
$$A1 = \frac{0 \times 3.38}{25.31} = 0 \text{ t}$$

$$B1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	Left span	Right span
	B1	A1
DL	14.5	14.5
SIDL	3.3	3.3
LL	7.3807	0
	11.93	0
	37.111	**17.8**



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 54.911 x 2.125

= 116.68522 t-m

Due to selfweight of pier cap = 6.48 x 1.1667 = 7.56 t-m

Total moment at the face of support = 124.25 t-m

Torsion at outerbearing section:

= 37.111 - 17.8

= 19.311

Eccentricity = 0.4

Moment = 7.7243 t-m

Longitudinal Reinforcement:

$M_e = M + M_t$

$M_t = T \times \dfrac{1 + D/b}{1.7}$

$= 6.8155$ t-m

$M_e = 124.25 + 6.8155 = 131.06$ t-m

For, M - 25 & Fe - 415

$\sigma cbc = 8.33$ N/mm²

m = 10

$\sigma st = 200$ N/mm²

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \dfrac{131.06 \times 10^7}{1.105 \times 1000}$

$= 1088.9 < 1425$ O.K

A_{st} required $= \dfrac{131.06}{200 \times 0.902 \times 1425}$

$= 5098.4736\ mm^2$

Provide 6 nos 25 mm dia bars in 2 layers 5890.5 mm^2

This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 37.111 + 17.8

= 54.911 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 61.391 t

After shear correction, S.F $= V + \dfrac{M \tan\beta}{d}$

$= 61.391 + \dfrac{124.25 \times 0.4}{1.425}$

= 26.514841 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

$b = \dfrac{1 + 1}{2} = 1$ m

$V_e = V + 1.6 \times \dfrac{T}{b} = 38.874$ t

Shear stress $= \dfrac{38.874 \times 10000}{1000 \times 1425} = 0.2728\ N/mm^2$

Area of tension reinforcement = 0.4134 %

$\tau_c = 0.2873\ N/mm^2$

```
INPUT FILE: 9.2 M SPAN CLASSA 2.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;30.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
    10. 12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38
0.0 0.0
    11. 24 18.4 0.0 0.0
    12. MEMBER INCIDENCES
    13. 1 1 2 23 1 1
    14. CONSTANTS
    15. E 2.7386E+006
    16. POISSON 0.15
    17. DENSITY 2.4 ALL
    18. MEMBER PROPERTY INDIAN
    19. 1 TO 23  PRIS YD 1 ZD 1
    20. SUPPORTS
    21. 3 22 FIXED BUT FX  MZ
    22. 11 14 PINNED
    23. MEMBER RELEASE
    24. 1 START FX FY MZ
    25. 12 START FX FY MZ
    26. 23 END FX FY MZ
    27. DEFINE MOVING LOAD
    28. TYPE 1 LOAD 2.7 2.7 11.4 11.4 6.8 6.8 6.8 6.8
    29. DIST 1.1 3.2 1.2 4.3 3 3 3
    30. **TWO LANE OF CLASS A
    31. LOAD GENERATION 82
    32. TYPE 1 -18.8 0 0 XINC 0.5
    33. PERFORM ANALYSIS
    34. **MAX REA
    35. LOAD LIST 46
    36. PRINT SUPPORT REACTION
 SUPPORT  REACTION
    -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z
      3   46      0.00      4.01      0.00      0.00      0.00      0.00
     22   46      0.00      8.20      0.00      0.00      0.00      0.00
     11   46      0.00     12.79      0.00      0.00      0.00      0.00
     14   46      0.00     16.80      0.00      0.00      0.00      0.00
    37. ** MAX MOM
    38. LOAD LIST 49
    39. PRINT SUPPORT REACTION
      3   49      0.00      1.96      0.00      0.00      0.00      0.00
     22   49      0.00     12.53      0.00      0.00      0.00      0.00
     11   49      0.00      3.44      0.00      0.00      0.00      0.00
     14   49      0.00     23.87      0.00      0.00      0.00      0.00
      40. FINISH
```

```
INPUT FILE: 9.2 M SPAN CLASSA  70R.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;30.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
    10. 12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38
0.0 0.0
    11. 24 18.4 0.0 0.0
    12. MEMBER INCIDENCES
    13. 1 1 2 23 1 1
    14. CONSTANTS
    15. E 2.7386E+006
    16. POISSON 0.15
    17. DENSITY 2.4 ALL
    18. MEMBER PROPERTY INDIAN
    19. 1 TO 23  PRIS YD 1 ZD 1
    20. SUPPORTS
    21. 3 22 FIXED BUT FX  MZ
    22. 11 14 PINNED
    23. MEMBER RELEASE
    24. 1 START FX FY MZ
    25. 12 START FX FY MZ
    26. 23 END FX FY MZ
    27. DEFINE MOVING LOAD
    28. TYPE 1 LOAD 8 12 12 17 17 17 17
    29. DIST 3.96 1.52 2.13 1.37 3.05 1.37
    30. ** CLASS 70 RW
    31. LOAD GENERATION 82
    32. TYPE 1 -13.4 0 0 XINC 0.5
    33. PERFORM ANALYSIS
    34. **MAX REA
    35. LOAD LIST 29
    36. PRINT SUPPORT REACTION
SUPPORT  REACTION
    -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z

      3   29      0.00     18.95      0.00      0.00      0.00      0.00
     22   29      0.00     15.00      0.00      0.00      0.00      0.00
     11   29      0.00     30.05      0.00      0.00      0.00      0.00
     14   29      0.00     36.00      0.00      0.00      0.00      0.00
      37. ** MAX MOMENT
    38. LOAD LIST 19
    39. PRINT SUPPORT REACTION
      3   19      0.00     32.85      0.00      0.00      0.00      0.00
     22   19      0.00      0.00      0.00      0.00      0.00      0.00
     11   19      0.00     47.15      0.00      0.00      0.00      0.00
     14   19      0.00      0.00      0.00      0.00      0.00      0.00
      40. FINISH
```

BRIDGE AT CH:27+800

DESIGN OF SUBSTRUCTURE (PIER)

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete	Pier & Pier Cap	= M - 35
	Pile cap & Pile	= M - 35
Grade of reinforcement steel		= Fe - 415
Centre to Centre distance of A / Expansion joints		= 32.200 m
Centre to Centre distance of Bearing		= 30.000 m
Depth of superstructure		= 2100 mm
Thickness of wearing coat		= 65.00 mm
Formation level along ℄ of carriage way		= 218.139 m
Height of bearing and Pedestal		= 0.300 m
L.W.L./G.L		= 207.905 m
H.F.L		= 214.172 m
M.S.L		= 193.355
Pier cap top level		= 215.674 m
Pile cap top level		= 207.155 m
Depth of pile cap		= 1800 mm
Pile cap bottom level		= 205.355 m
Diameter of bored cast-in-situ piles		= 1200 mm
Length of piles above rock level		= 10.000 m
Length of piles below rock level		= 2.000 m
Total length of pile		= 12.000 m
Load carrying capacity of each pile		= **275** Tons

Live Load **(a)** Class A 2 Lane

(b) Class 70R wheeled

Maximum surface velocity of water $= \sqrt{2} \times 3.000$ m/sec = 4.24 m/sec

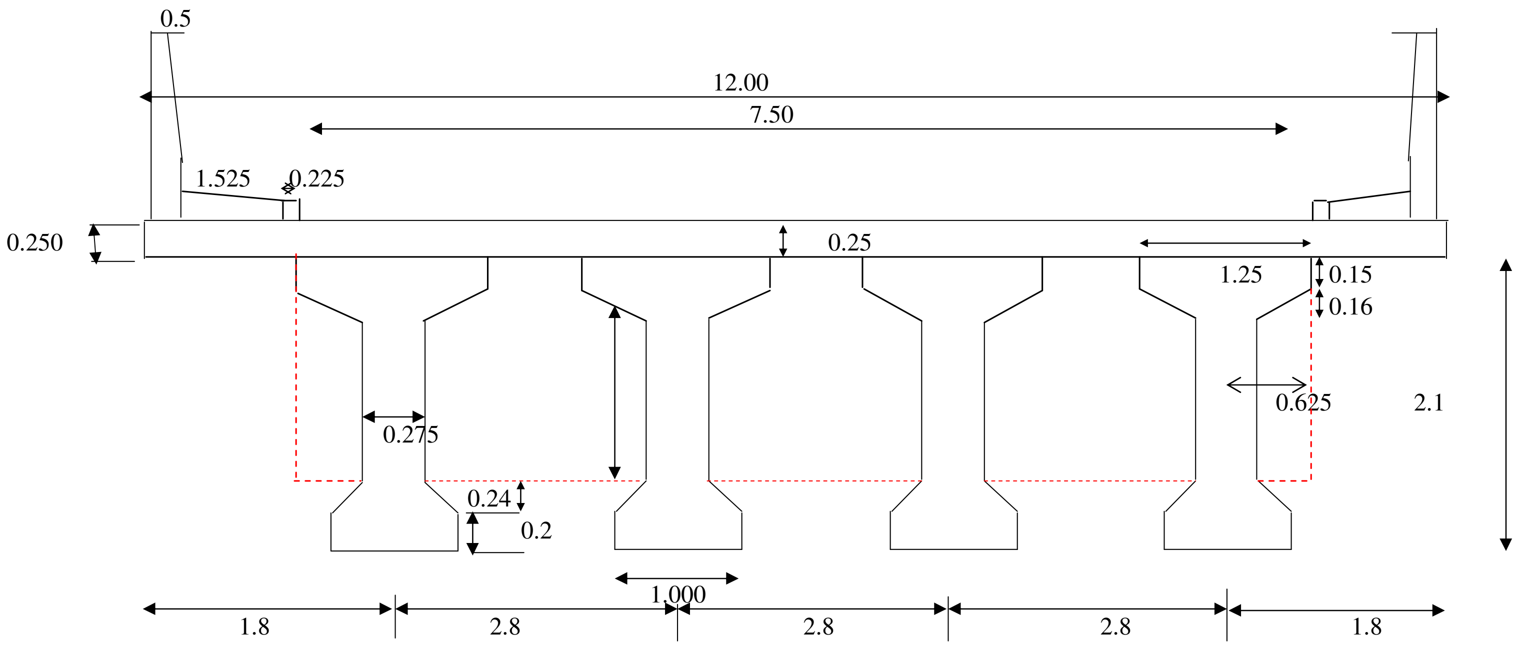
Say, 4.30 m/sec

The following codes are used for the design of substructure:

1 IRC : 6 - 2000

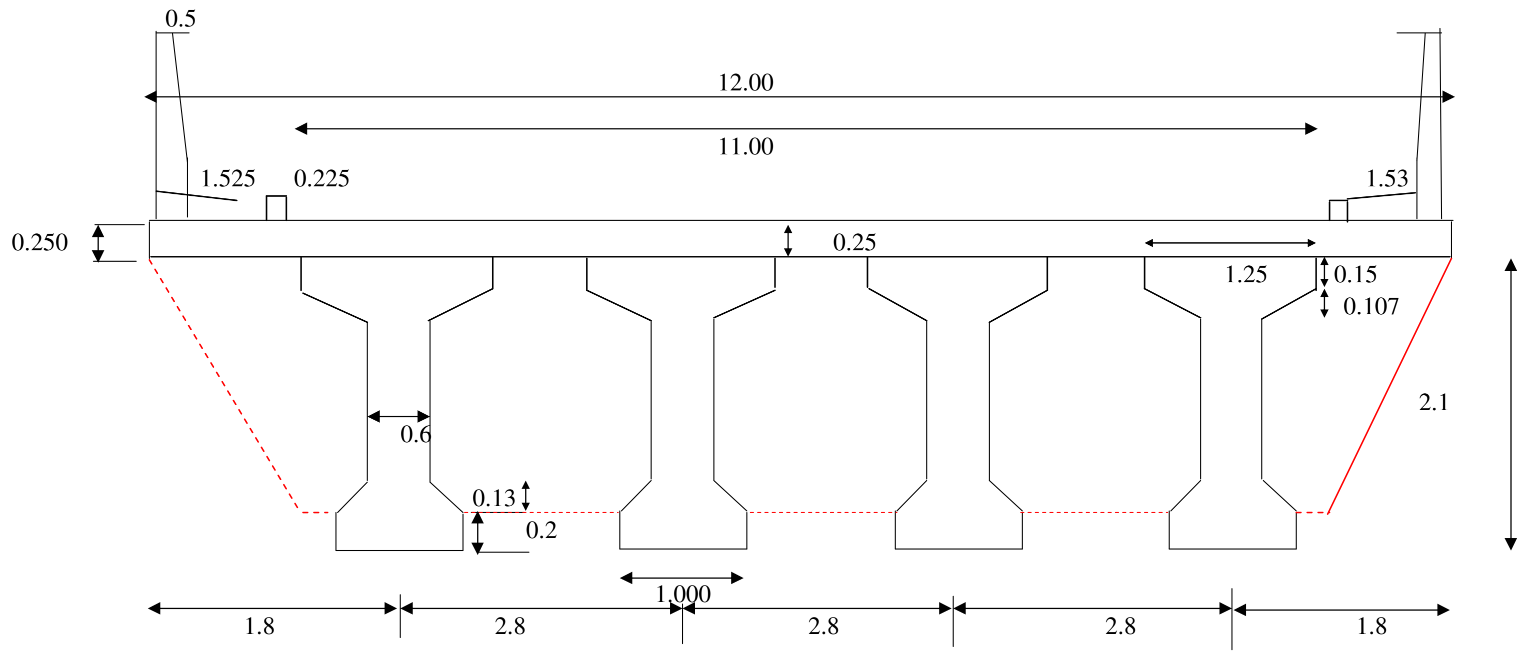
2 IRC : 21 - 2000

3 IRC : 78 - 2000



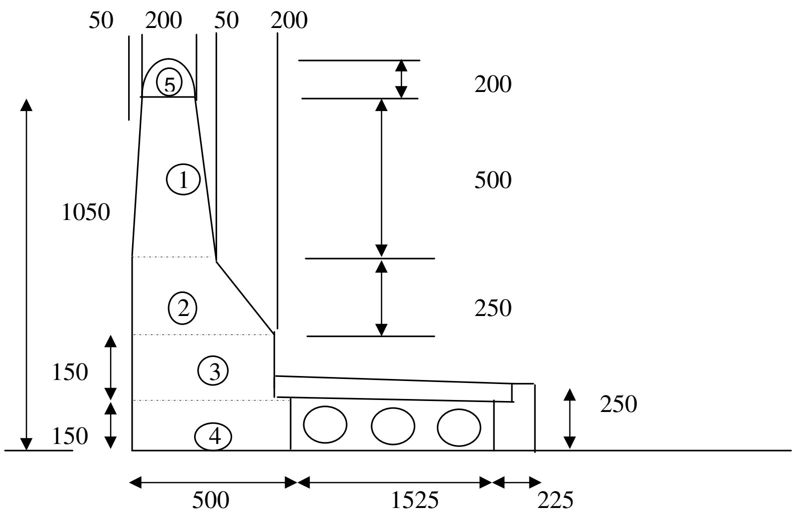
Depth of SuperStructure			=	2.35	
Number of Longitudinal Girders			=	4	
Mid section					
1 slab	=	12*0.25	=	3	m^2
2 Top Flange	=	4*[(0.15*1.25)+(0.5*(1.25+0.275))*0.16	=	1.238	m^2
3 Web	=	4*[(2.1-(0.15+0.16)-(0.2+0.24))*0.275]	=	1.485	m^2
4 Bottom Bulb	=	4*[(1*0.2)+(0.5*(1+0.275))*0.24	=	1.412	m^2
Total			=	**4.135**	m^2

End Section

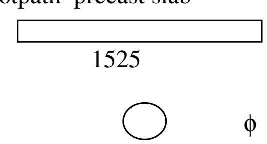


Depth of SuperStructure		=	2.35	
Number of Longitudinal Girders		=	4	
End section				
1 slab	= 12*0.25	=	3	m^2
2 Top Flange	= 4*[(0.15*1.25)+(0.5*(1.25+0.6))*0.107	=	1.1459	m^2
3 Web	= 4*[(2.1-(0.15+0.107)-(0.2+0.132))*0.6]	=	3.6264	m^2
4 Bottom Bulb	= 4*[(1*0.2)+(0.5*(1+0.6))*0.132	=	1.2224	m^2
Total		=	**5.99**	m^2
DL per running meter at Intermediate section		=	10.3375	t/m
DL per running meter at End section		=	14.98675	t/m

Intermediate Cross Girder				
Thickness of Cross Girder		=	0.3	
Number of intermediate cross girders		=	3	
Cross sectional area in elevation		=	12.2585	m^2
Dl of intermediate cross girder		=	**27.58163**	t
End Cross Girder				
Thickness of end Cross Girder		=	0.45	
Number of intermediate cross girders		=	2	
Cross sectional area in elevation		=	18.0277	m^2
Dl of intermediate cross girder		=	**40.56233**	t
S.I.D.L				
I Wearing Coat @ 1.0725 t/m		=	1.0725	t/m
II Crash barrier @ 1.00 t/m				
(provide one side enter 1,bothsides enter 2 , otherwise enter 0)	2	=	2.00	t/m
III R.C.C Post & Railing @ 0.085 t/m				
(provide one side enter 1,bothsides enter 2 , otherwise enter 0)	0	=	0	t/m



footpath precast slab



100
1525
ϕ 100 mm

Wt due to Crashbarrier

Component	Area	Wt
1	0.100	0.24
2	0.0875	0.21
3	0.075	0.18
4	0.075	0.18
5	0.002	0.15413
		0.96 t/m

say, 1.000

Footpath precast slab

0.1525 0.38125 t/m

missllaneous utilities 0.2 t/m

Footpath kerb 0.05625 0.14063 t/m

sandfilling 0.20519 0.36934 t/m

IV Foot path precast s @ 0.38125 t/m
(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.7625 t/m

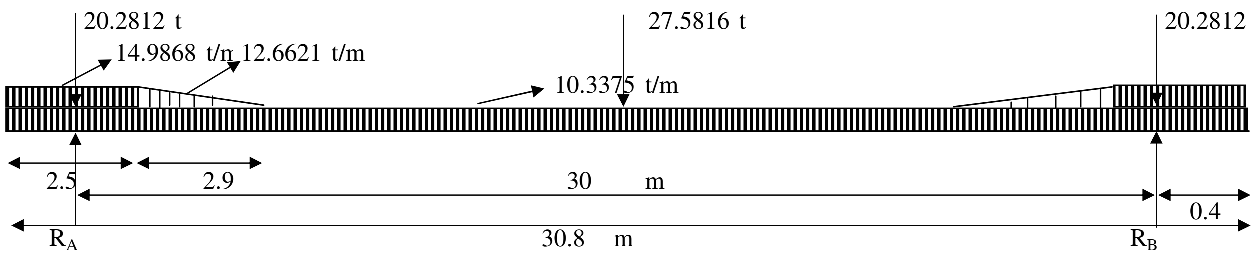
V Footpath Kerb @ 0.14063 t/m
(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.28125 t/m

VI Missillaneous utilities @ 0.2 t/m
(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.4 t/m

VII Sandfilling in footpath @ 0.36934 t/m
(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.738677 t/m

Total = 5.25 t/m

Total S.I.D.L Reaction = 169.2086 t



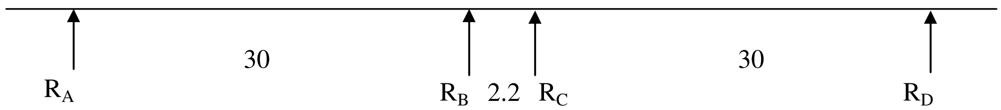
Due to girders:

Reaction R_A = (10.3375*0.5*25.4)+(14.99*2.5)+(0.5*12.66*2.9)+20.28+13.79)
= 221.1852 t

Due to SLAB = 120.75 t

Due to SIDL = **84.6043** t

Total DL + SIDL = **853.079** t



Longitudinal Force :

For Class A Single lane : $F_h = (0.2 \times 55.4) = 11.080$ t

For class 70R wheeled : $F_h = (0.2 \times 100) = 20.000$ t

For class A 2 lane $F_h = 0.2 \times 55.4 = 11.08$ t

	Fixed Bearing	**Free Bearing**
i	$F_h - \mu(R_g+R_q)$	$\mu(R_g+R_q)$
ii	$\frac{F_h}{2} + \mu(R_g+R_q)$	$\mu(R_g+R_q)$

i $20.0 - 0.05 \times 440.54$ 0.05×524.46

$= -2.027$ t $= 26.223$ t

ii $\frac{20.0}{2} + 0.05 \times 440.54$ 0.05×524.46

$= 32.027$ t $= 26.223$ t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)	Longitudinal Moment (t-m)
70R Wheeled	32.03	
Span dislodged	0.00	

Moment in Transverse Direction

Eccentricity of vertical load in transverse direction,

(a) Class A Single lane $= \frac{11.0}{2.00} - 0.40 - \left[\frac{1.80}{2}\right] = 4.2$

(b) Class A 2 lane $= \frac{11.0}{2.00} - 0.40 - \left[\frac{1.80 \times 2 + 1.7}{2}\right] = 2.45$

(c) Class 70R Wheeled

$= \frac{11.0}{2.00} - 1.63 - \left[\frac{1.93}{2}\right] = 2.91$

Footpath Live load :

The effective span of girders = 32.200 m > 30.00 m

Therefore, Intensity of footpath live loac = $P = \left(P' - 260 + \frac{4800}{L}\right)\left(\frac{16.5 - W}{15}\right)$

Where, $P' = 400$ Kg/m^2

Therefore, $\left(400 - 260 + \frac{4800}{32.2}\right)\left(\frac{16.5 - 1.5}{15}\right)$

$= 289.07$ Kg/m^2 $= 0.289$ t/m^2

(a) When one side is loaded;

Load on each span = 0.289 x 1.5 x 32.200 = 13.962 t Say, 14.0 t

(b) When both sides are loaded;

Load on each span = 13.962 x 2 = 27.924 t Say, 28.0 t

Eccentricity of load in transverse direction:

(a) When one side is loaded = $6 - \left(0.5 + \frac{1.5}{2}\right) = 4.75$ m

(b) When both sides loaded = Nil

Summary of Footpath Live load :

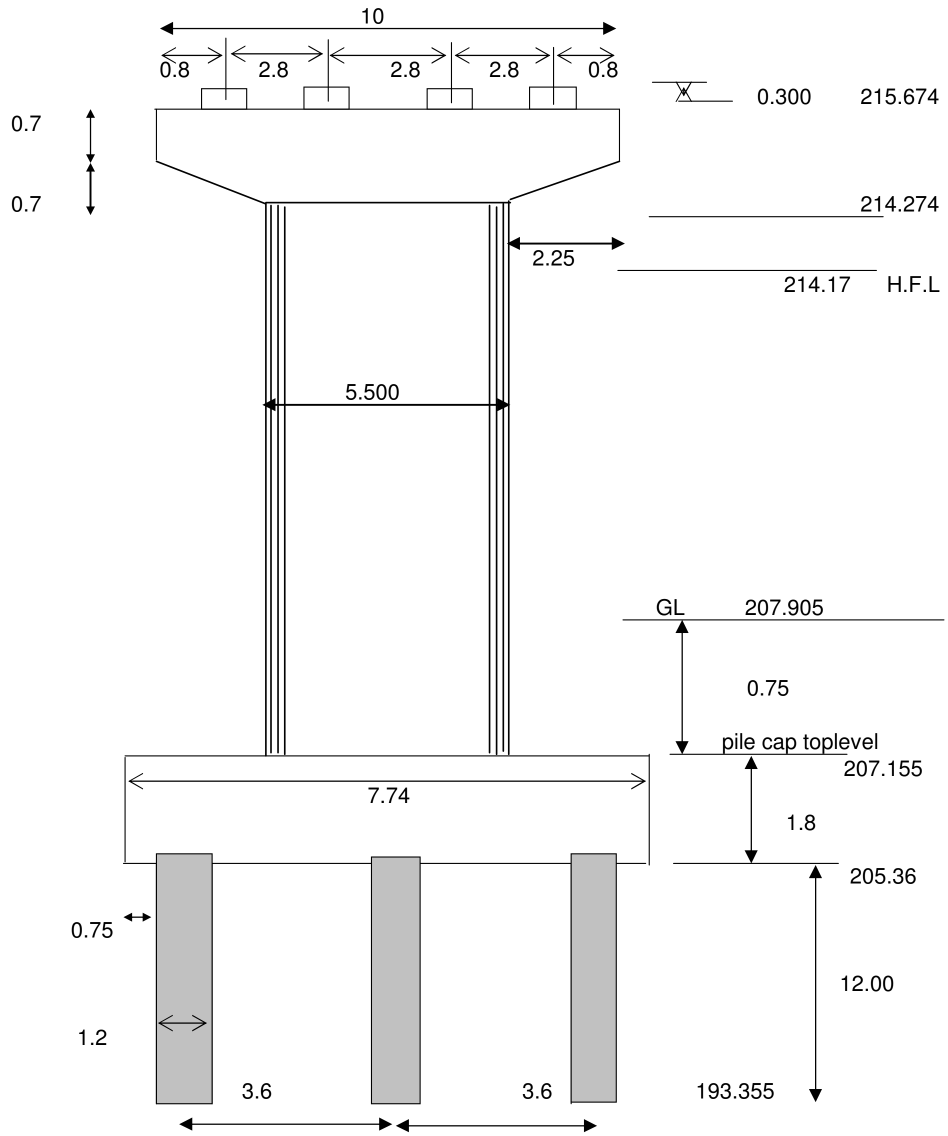
Load Case	Reaction	Longitudinal moment	Transverse moment
When one side is loaded	7.00	0.00	33.25
When both sides loaded	28.00	0.00	0.00

LOADING:	CASE	R_A (t)	R_B(t)	R_C(t)	R_D(t)	M_L	M_T	$R_B + R_C$
Class A 1 Lane	MAX REA	13.17	28.63	19.71	7.49	9.812	203.03	48.34
	MAX LONG	17.19	38.21	0	0	42.031	160.48	38.21
Class A 2 Lane	MAX REA	26.34	57.26	39.42	14.98	**19.624**	**236.87**	**96.68**
	MAX LONG	34.38	76.42	0	0	84.062	187.23	76.42
70R Wheeled	MAX REA	5.3	43.7	46.87	4.13	3.487	263.56	90.57
	MAX LONG	16.08	83.92	0	7.86	**92.312**	**244.21**	**83.92**

SUMMARY OF LOAD ON PIER FROM SUPERSTRUCTURE

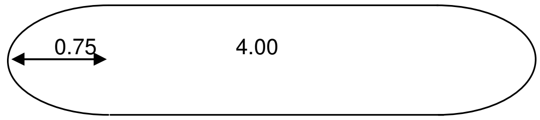
	P	M_L	M_T
1 D.L	683.87	0	0
2 S.I.D.L	197.21	0	0
3 L.L			
Max Rea	96.68	19.624	236.87
Max Long	83.92	92.312	244.21

Wt of Substructure :



Loads from substructure

1 DRY Condition



1 **Wt of Pier cap**

Rectangular										
	10	x	0.7	x	3.20	x	2.4	=	53.76	t
Trapezoidal	7.75	x	0.7	x	3.20	x	2.4	=	41.664	t
								=	95.424	t

2 Wt of Pier Shaft

4.000	x	7.119	x	1.5	x	2.4	=	102.514	
0.7854	x	2.250	x	7.12	x	2.4	=	30.1927	t
							=	132.706	t

3 Wt of Pile cap

56.07	x	1.8	x	2.4	=	242.24	t

4 Weight due to Earth

(56.07	-	7.77)	x	1.8	x	0.75	=	65.2144	

2 H.F.L

Rectangular										
	10	x	0.7	x	3.20	x	2.4	=	53.76	t
Trapezoidal	7.75	x	0.7	x	3.20	x	2.4	=	41.664	t
								=	95.424	t

2 Wt of Pier Shaft

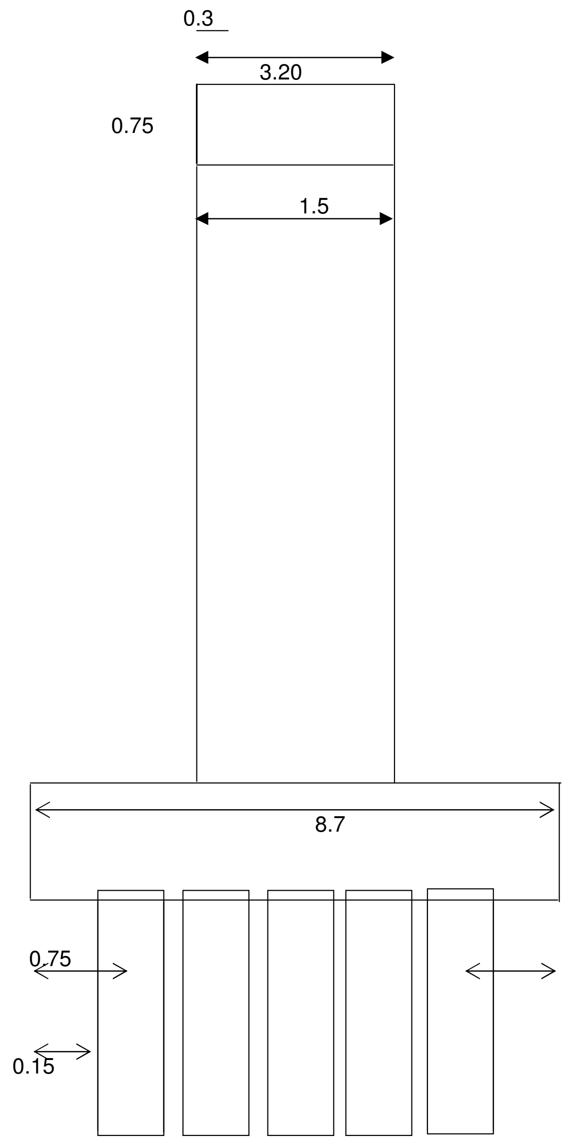
4.000	x	1.5	x	0.10	x	2.4	=	1.4688	t
4	x	1.5	x	7.02	x	1.4	=	58.9428	t
0.7854	x	2.250	x	0.10	x	2.4	=	0.4326	t
0.7854	x	2.250	x	7.02	x	1.4	=	17.3601	t
								78.2043	t

3 Wt of Pile cap

56.07	x	1.8	x	1.4	=	141.307	t

4 Weight due to Earth

(56.07	-	7.77)	x	1.0	x	0.75	=	36.2302	


0.3
3.20
0.75
1.5
8.7
0.75
0.15

H.F.L. Case :

Formation level = 218.139 m
H.F.L. = 214.172 m
Foundation level = 193.355 m
Vertical load on pile foundation;
Vertical load from substructure & Pile cap

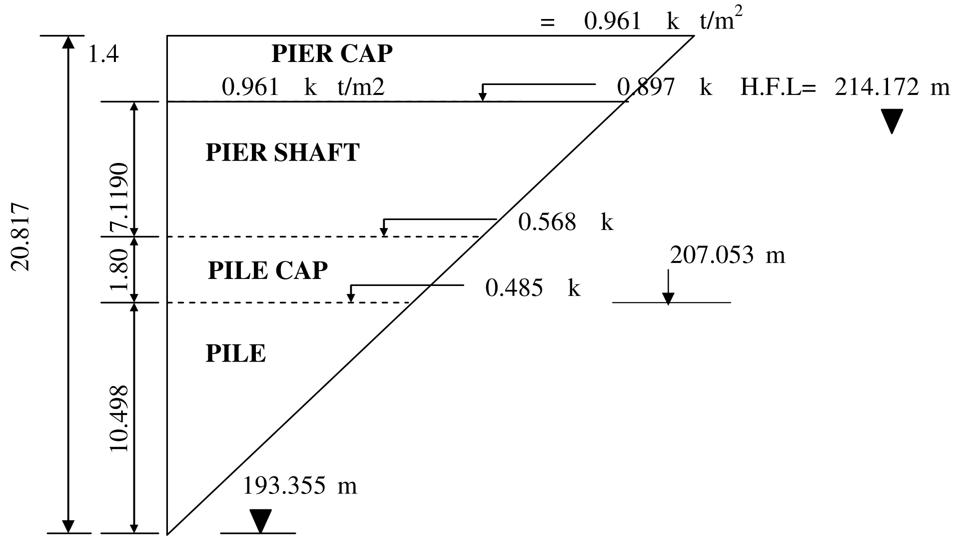
[95.424 +	78.204 +	141.31 +	36.23]	= 351.165 t	Say, 352.00 t
Pier cap	Pier shaft	pile cap	earth	Total load	= 352.00 t

Water current pressure;

Maximum surface velocity at H.F.L. = 4.30 m/sec

So,

$$P = 52 \; kV^2$$
$$= 52 \; x \; k \; x \; 4.30^2 = 961.48 \; k \; kg/m^2$$
$$= 0.961 \; k \; t/m^2$$



Transverse direction;

Pressure per metre depth

(a) Pier cap

0.961 x $\cos 20^0$ x 0.00 x 1.50 = 0 t/m depth

0.897 x $\cos 20^0$ x 0.00 x 1.50 = 0 t/m depth

(b) Pier shaft

0.897 x $\cos 20^0$ x 0.66 x 1.50 = 0.834 t/m depth

0.568 x $\cos 20^0$ x 0.66 x 1.50 = 0.528 t/m depth

(c) Pile cap

0.568 x $\cos 20^0$ x 1.50 x 8.70 = 6.966 t/m depth

0.485 x $\cos 20^0$ x 1.50 x 8.70 = 5.946 t/m depth

Pressure & moment at pile cap bottom;

	0	x	1.4	=	0	t	x	9.619	m	=	0	t-m
$\frac{1}{2}$ x (0 -	0)	x	1.4	=	0.000	t	x	9.852	m	=	0.00	t-m
	0.528	x	7.119	=	3.762	t	x	5.360	m	=	20.16	t-m
$\frac{1}{2}$ x (0.834 -	0.528)	x	7.119	=	1.089	t	x	6.546	m	=	7.13	t-m
	5.946	x	1.800	=	10.703	t	x	0.900	m	=	9.63	t-m
$\frac{1}{2}$ x (6.966 -	5.946)	x	1.800	=	0.918	t	x	1.200	m	=	1.10	t-m
		Total load		=	16.471	t				=	38.022	t-m
			Say,		17.00	t				Say,	39.00	t-m

Longitudinal direction;

Pressure per metre depth

(a) Pier cap

0.961 x $\sin 20^0$ x 0.00 x 10.00 = 0.000 t/m depth

0.897 x $\sin 20^0$ x 0.00 x 10.00 = 0.000 t/m depth

(a) Pier shaft

0.897 x $\sin 20^0$ x 1.50 x 5.50 = 2.531 t/m depth

0.568 x $\sin 20^0$ x 1.50 x 5.50 = 1.603 t/m depth

(b) Pile cap

0.568 x $\sin 20^0$ x 1.50 x 7.74 = 2.254 t/m depth

0.485 x $\sin 20^0$ x 1.50 x 7.74 = 1.924 t/m depth

Pressure & moment at pile cap bottom;

	0.000	x	1.400	=	0.000	t	x	9.619	m	=	0.00	t-m
$\frac{1}{2}$ x (0.000 -	0.000)	x	1.400	=	0.000	t	x	9.852	m	=	0.00	t-m
	1.603	x	7.119	=	11.410	t	x	5.360	m	=	61.15	t-m
$\frac{1}{2}$ x (2.531 -	1.603)	x	7.119	=	3.302	t	x	6.546	m	=	21.62	t-m
	1.924	x	1.800	=	3.464	t	x	0.900	m	=	3.12	t-m
$\frac{1}{2}$ x (2.254 -	1.924)	x	1.800	=	0.297	t	x	1.200	m	=	0.36	t-m
		Total load		=	18.473	t				=	86.243	t-m
			Say,		19.00	t				Say,	87.00	t-m

Therefore, Total vertical load = P = 352.00 t

Longitudinal moment = M_L = 87.00 t-m

Transverse moment = M_T = 39.00 t-m

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

L.W.L. = 207.905 m
Pile cap top level = 207.155 m
Pile cap bottom level = 205.355 m

Vertical load on Pile foundation;
Load from superstructure : Max Reaction case
S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
+ **Live load** = 96.68 t
977.76 t Say, 978.00 t
Load from superstructure : Max Long.Moment case
S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
+ **Live load** = 83.92 t
965.00 t Say, 965.00 t
Load from substructure & pile cap =
95.42 + 132.71 + 242.24 65.214 = 535.59 t Say, 536.00 t
Pier cap Pier shaft pile cap earth
Horizontal longitudinal force at bearing level for pier = 32.027 t
Moment = 32.027 x 10.619 = 340.1
Total = 340.09 Say, 341 t-m

	P	M_L	M_T
Max Rea	**1514.00**	**360.62**	**236.9**
Max Long	**1501.00**	**433.31**	**244.2**

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

Water level = 214.172 m
Pile cap top level = 207.155 m
Pile cap bottom level = 205.355 m

Load from superstructure: Max Reaction case
S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
Live load = 96.68 t
977.76 t Say, 978.00 t
Load from superstructure: Max Long.Moment case
S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
Live load = 83.92 t
965.00 t Say, 965.00 t
Load from substructure & pile cap = 351.165 t Say, 352.00 t

	P	M_L	M_T
Max Rea	**1330.00**	**447.62**	**275.9**
Max Long	**1317.00**	**520.31**	**283.2**

Seismic forces do not govern for Zone II & III .Sample calculations have already been submitted.

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

Vertical load on Pile foundation;
Load from superstructure : Max Reaction case

S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
Live load = 0.00 t
881.08 t Say, 882.00 t

Load from substructure & pile cap = 535.585 t Say, 536.00 t

Horizontal longitudinal force at bearing level for pier = 0.00 t
Moment = 0.00 x 10.619 = 0.000 t-m
Total = 0.000 Say, 0 t-m

Transverse moment = 0.00 Say, 0.0 t-m

	P	M_L	M_T
Max Rea	**1418.00**	**0.00**	**0.0**
Max Long	**1418.00**	**0.00**	**0.0**

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

Vertical load on Pile foundation;
Load from superstructure

S.I.D.L. + D.L. = 197.21 + 683.87 = 881.08 t
Live load = 0.00 t
881.08 t Say, 882.00 t

Load from substructure & pile cap = 351.165 t Say, 352.00 t

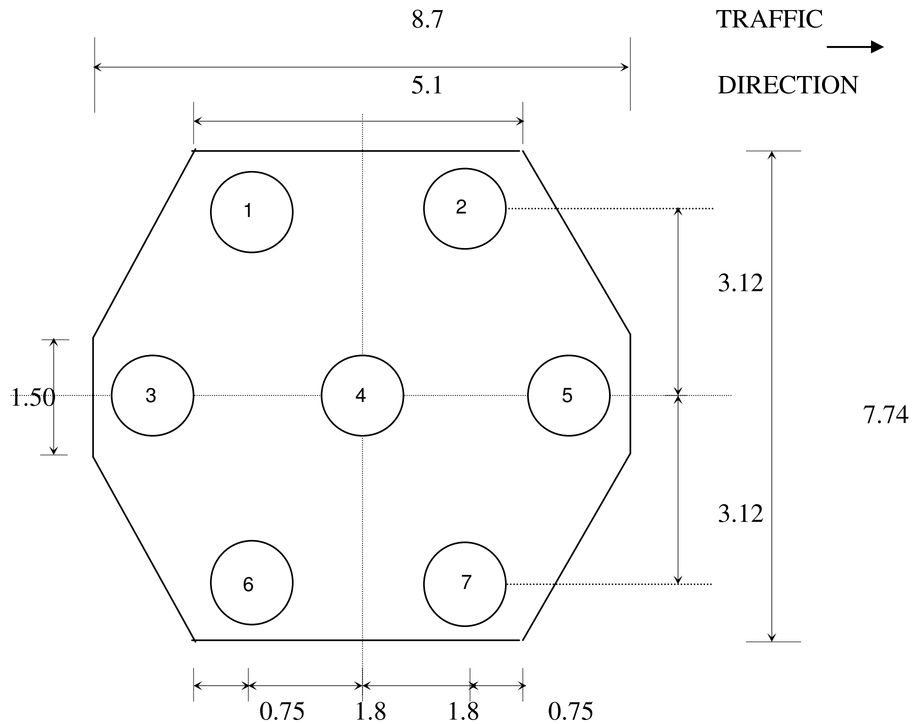
Horizontal longitudinal force at bearing level for pier = 0.00 t
Moment = 0.00 x 10.619 = 0.000 t-m Say, 0

	P	M_L	M_T
Max Rea	**1234.00**	**87.00**	**39.0**
Max Long	**1234.00**	**87.00**	**39.0**

DESIGN OF PILE LAYOUT

Diameter of pile D	=	1.20	m	
c/c distance ratio of pile, K	=	3.00	x	1.20
	=	3.60	m	
Projection of pile cap from the fac	=	0.15	m	

NO. OF PILES : 7



PILE LAYOUT PLAN

PILE NO		X	Z_L		Y	Z_T
1		1.800	21.60		3.118	12.47
2		1.800	21.60		3.118	12.47
3		3.600	10.80		0.000	-
4		0.000	-		0.000	-
5		3.600	10.80		0.000	-
6		1.800	21.60		3.118	12.47
7		1.800	21.60		3.118	12.47
	εx^2	38.88		εy^2	38.88	

Where

x = Distance of pile from C.G of pile group in longitudinal direction

Z_L = $\varepsilon x^2 / x$

y = Distance of pile from C.G of pile group in transverse direction

Z_T = $\varepsilon y^2 / y$

Pile Cap

Area of Pile cap = 56.07 m^2

Dry Case:	P	M_L	M_T	Pile 1	Pile 2	Pile 3
Max Rea	1514.00	360.62	236.87	251.97	218.58	249.68
Max Long	1501.00	433.31	244.21	254.07	213.95	254.55
Span Dislodged	1418.00	0.00	0.00	202.57	202.57	202.57
H.F.L Case:						
Max Rea	1330.00	447.62	275.87	232.84	191.40	231.45

Max Long	1317.00	520.31	283.21	234.94	186.76	236.32
Span Dislodged	1234.00	87.00	39.00	183.44	175.39	184.34
Dry Case:	Pile 4	Pile 5	Pile 6	Pile 7		
Max Rea	216.29	182.8946	213.99	180.60		
Max Long	214.43	174.3071	214.91	174.79		
Span Dislodged	202.57	202.5714	202.57	202.57		
H.F.L Case:						
Max Rea	190.00	148.5533	188.60	147.16		
Max Long	188.14	139.9658	189.52	141.34		
Span Dislodged	176.29	168.2302	177.19	169.13		
Dry Case:	Pile 1 + Pile 2		Pile 1 + Pile 6	Pile 3		
Max Rea	470.56		715.64	249.68		
Max Long	468.02		723.53	254.55		
Span Dislodged	405.14		607.71	202.57		
H.F.L Case:						
Max Rea	424.24		570.00	231.45		
Max Long	421.71		564.43	236.32		
Span Dislodged	358.83		528.86	184.34		

Design of Pile Layout

Load Case	Total Load	Longitudinal Moment	Transverse Moment
	P	M_L	M_T
I **DRY Condition**			
1 With L.L			
Max Rea	1514.00	360.62	236.87
Max Long	1501.00	433.31	244.21
2 Span dislodged	1418.00	0.00	0.00
II **H.F.L Condition**			
1 With LL			
Max Rea	1330.00	447.62	275.87
Max Long	1317.00	520.31	283.21
2 Span dislodged	1234.00	87.00	39.00

Number of Piles **N** = 7.00

Pile Design:

Horizantal load per pile

Dry Condition

Total horizantal force					
with L.L	= 32.03		=	32.03	t
			=	4.5752822	t per pile
span dislodged condition	= 0.00		=	0.00	t
			=	0	t per pile

H.F.L Condition

Longitudinal Direction					
Total horizantal force					
with L.L	= 32.03	+ 19.00	=	51.03	t
			=	7.29	t per pile
span dislodged condition	= 0.00	+ 19.00	=	19.00	t
Transverse Direction			=	2.71	t per pile
Total horizantal force					
with L.L	=	17.00	=	17.00	t
			=	2.43	t per pile
span dislodged condition	=	17.00	=	17.00	t
			=	2.43	t per pile

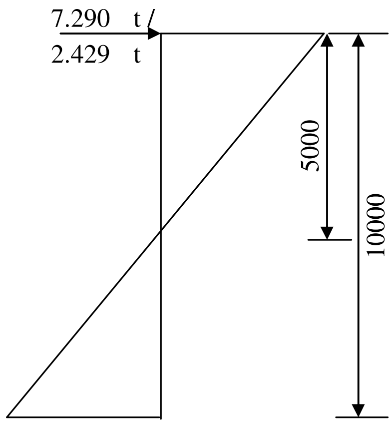
The pile is socketed into the rock and considered as fixed at the bottom for analysis and the Effective height of pile for analysis is taken from bottom of pile cap to top of rock level.

Depth of Rock below bottom of pile cap level = 10.000 m

Self weight of pile = $\frac{\pi \times 1.20^2}{4}$ x 10.000 x 1 nos.

= 11.310 m^3

Buoyant wt. @ 1.40 t/m^3 = 15.834 t



M_L = 7.290 x 5.00 = 36.448 t-m

M_T = 2.429 x 5.00 = 12.14 t-m

M_R = 38.42 t-m

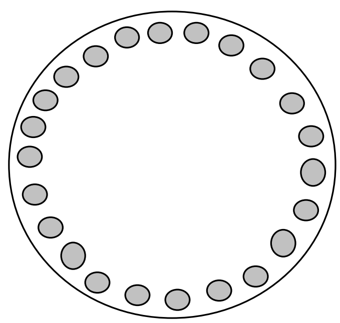
Reaction on each Pile = $\dfrac{38.42 \times 2.0}{7.74}$

= 9.933

Load at Pile base :

Maximum load on a pile at pile cap bottom = 254.55 t

Minimum load on a pile at pile cap bottom = 139.97 t



Provide 23 nos 16 ϕ

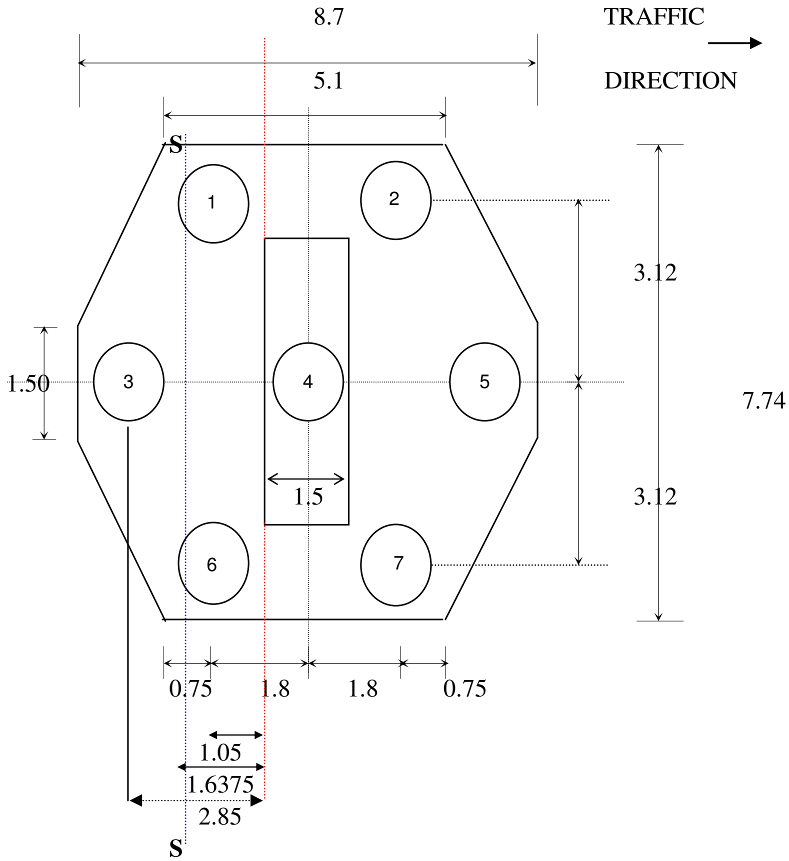
= 46.2 cm^2

= 45.2 cm^2

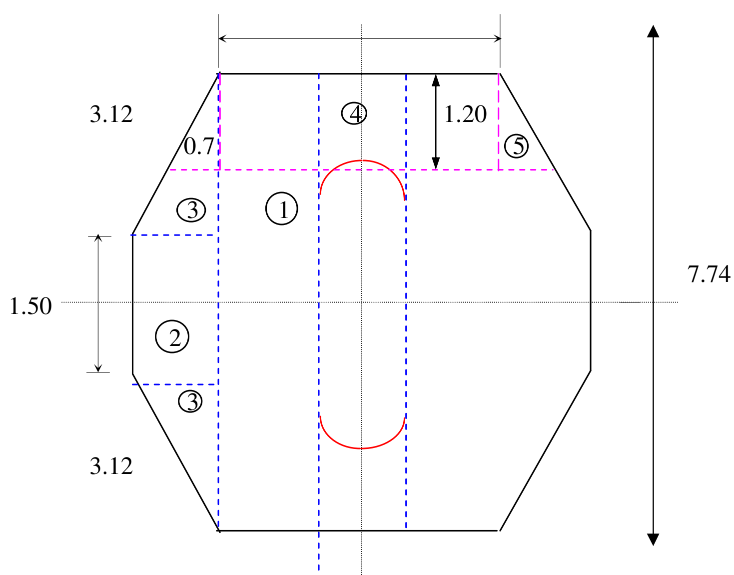
0.4%

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)	82.90	62.14
AS=AREA OF STEEL (in sq. Cm)	46.244	46.244
P=VERTICAL LOAD (in tonnes)	254.55	139.97
BM=BENDING MOMENT (in t-m)	38.42	38.42
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	0.001	0.001
B=BD*PI/180	1.447	1.085
B1=COS(B)	0.124	0.467
A=ALPHA ANGLE=ACOS (RO*B1/RI)	1.429	1.007
RECOSB=RICOSA		
B2=SIN(B)^3	0.977	0.691
B3=SIN(4*B)	-0.476	-0.931
B4=SIN(2*B)	0.245	0.826
A=COS(A)	0.141	0.534
A2=SIN(2*A)	0.280	0.903
A3=SIN(A)	0.990	0.845
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.237	0.336
NUM2=2*(RO^3)/(1+B1)	0.384	0.294
NUM3=(RI^3) *T/(RO+RI*A1)	0.000	0.000
NUM4=(M-1) *PI+A-A2/2	29.56	28.83
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.100	0.105
DENM1=2*(RO^2)/(1+B1)	0.641	0.491
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	0.438	0.808
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.001	0.001
DENM4=(M-1) *PI*A1-A3+A*A1	3.21	14.79
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.284	0.409
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.352**	**0.258**
ACTUAL ECCENTRICITY EC = M/P	**0.151**	**0.274**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	0.074	0.280
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.674	0.880
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**895**	**342**
TS=TENS.STRESS IN STEEL (T/SQM)	**5985**	**950**
PERMISSIBLE COMP.STRESS IN Concrete (in t/sq.m)	1190	1190
PERMISSIBLE TENSILE STRESS IN Steel (in t/sq.m)	20400	20400

PILE CAP DESIGN:



Total Load on Pile in One row Pile 1 & Pile 6 = 723.53 t
Total Load on Pile in Second row Pile 3 = 254.55 t
Lever arm for bending moment due to Pile 1 & Pile 6 = 1.05 m
Lever arm for bending moment due to Pile 3 = 2.85 m
Bending Moment in Transverse direction Pile 1 & Pile 6 = **759.71** t-m
Bending Moment in Transverse direction Pile 3 = **725.4677** t-m
Total Load on Pile 1 & Pile 2 = 470.56 t
Bending Moment in Longitudinal direction = **213.17** t-m



Wt of Pile cap in Transverse direction

Area of strip 1	=	13.92	m^2
Wt due to strip 1	=	60.15	t
Moment due to strip 1	=	54.14	t-m
Area of strip 2	=	2.70	m^2
Wt due to strip 2	=	11.66	t
Moment due to strip 2	=	31.49	t-m
Area of strip 3	=	5.61	m^2
Wt due to strip 3	=	24.24	t
Moment due to strip 3	=	58.18	t-m
Moment due to pile cap (Pile 1 & Pile 6)	=	**85.63**	t-m
Moment due to pile cap (Pile 3)	=	**58.18**	t-m

Wt of Pile cap in Longitudinal direction

Area of strip 4	=	6.14	m^2
Wt due to strip 4	=	26.50	t
Moment due to strip 4	=	15.94	t-m
Area of strip 5	=	0.84	m^2
Wt due to strip 5	=	3.61	t
Moment due to strip 6	=	1.45	t-m
	=	**17.39**	t-m
Total Bending Moment in Transverse direction Pile 1 & Pile 6	=	**674.08**	t-m
Total Bending Moment in Transverse direction Pile 3	=	**667.28**	t-m
Total Bending Moment in Longitudinal direction	=	**195.78**	t-m

Design for Flexure

M	=	35
Fe	=	415
σ_{cbc}	=	11.66667
σ_{st}	=	200
m	=	10
k	=	0.37
j	=	0.88
Q	=	1.89

Depth of Pile Cap	=	1.8	m
Pile shall be embeded by 50 mm into Pile cap & provide			
cover for Reinforcement 75 mm		1.6375	
d_{eff} avilable			
d_{eff} required in Transverse direction pile1& Pile 6	=	0.673187	m
d_{eff} required in Transverse direction Pile 3	=	0.669786	m
d_{eff} required in Longtudinal direction	=	0.39611	m
A_{st} required in Transverse direction due to pile 1 & pile 6	=	23464.05	mm^2
A_{st} required in Transverse direction due to pile 3	=	23227.6	mm^2
A_{st} required in Longtudinal direction	=	6815.029	mm^2
A_{st} required in Transverse direction /m width	=	**3033.341**	mm^2/m
A_{st} required in Transverse direction /m width	=	**3002.773**	mm^2/m
A_{st} required in Transverse direction /m width	=	**6036.114**	mm^2/m
Minimum area of steel required	=	**2160.00**	mm^2/m
		(As per cl. 305.19 of IRC :21-2000)	
A_{st} required in Longtudinal direction /m width	=	**1336.28**	mm^2/m
Minimum area of steel required	=	**2160.00**	mm^2/m

Transverse direction on Bottom face:

Total reinforcement in transverse direction on bottom face						=	6036.114	mm^2
Provide	32	ϕ	125	mm	c/c	=	6433.982	mm^2/m

Transverse direction on Top face:

Minimum area of steel required on top face						=	1080	mm^2/m
Provide	16	ϕ	130	mm	c/c	=	1546.63	mm^2/m

Longitudinal direction on Bottom face:

Provide	20	ϕ	130	mm	c/c	=	2416.61	mm^2/m
Provide	20	ϕ	130	mm	c/c	=	2416.61	mm^2/m

Longitudinal direction on Top face:

Minimum area of steel required on top face						=	1080	mm^2/m
Provide	16	ϕ	130	mm	c/c	=	1546.63	mm^2/m

Check for Shear

Distance of section S-S from face of Pier			
Portion of Pile Cap effective for Shear	=	1.6375	m
Pile 3 is fully effective for shear & Pile 1,Pile 6 are partially effective		1.9625	
Portion of Pile effective for Shear	=	1.20	m
Shear due to Pile 3	=	254.55	t
Shear due to Pile 1 & Pile 6	=	7.54	t
Total Shear	=	262.09	t
Wt of Pile cap upto section S-S	=	39.15	t
Net UpwardShear	=	222.94	t
Shear Stress	=	17.60	t/m^2
Percentage reinforcement provided	=	0.185242	%
Permissible Shear Stress	=	19.13129	t/m^2

No shear reinforcement required Provide Minimum Shear

Design of Pier shaft :

Calculation of Forces at Pier base

(At Top of Pile cap)

Checking the section at R.L. = **207.16** m

Lever arm for Longitudinal forces = 8.819 m i.e(215.97 - 207.16)

DRY ***With L.L***

1 Vertical load - Max Reaction case Max Long.Moment case

S.I.D.L.	= 197.21 t	=	197.21
Dead load	= 683.87 t	=	683.87
Live load	= 96.68 t	=	83.92
Pier cap	= 95.42 t	=	95.42
Pier shaft	= 132.706 t	=	132.706
Total load	= 1205.89 t	=	1193.1 t

2 Longitudinal moment -

Due to Longitudinal force = 32.027 x 8.819 = 282.45 t-m

283.00 t-m

	P	M_L	M_T
Max Rea	1205.89	302.62	236.87
Max Long	1193.13	375.31	244.21

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

1 Vertical load - Max Reaction Case Max Long.Moment Case

S.I.D.L.	= 197.21 t	=	197.21
Dead load	= 683.87 t	=	683.87
Live load	= 96.68 t	=	83.92
Pier cap	= 95.42 t	=	95.42
Pier shaft	= 78.204 t	=	78.204
Total load	= 1151.39 t Say, 1151.39 t	=	1138.6 t

2 Longitudinal moment -

Due to earth = 0.000 t-m

Due to Longitudinal force = 282.45 t-m

Total moment = 282.45 t-m Say, 283.00 t-m

	P	M_L	M_T
Max Rea	1151.39	385.39	264.15
Max Long	1138.6	458.08	271.50

DRY ***spandislodgedcondition***

1 Vertical load -

S.I.D.L. = 197.21 t

Dead load = 683.87 t

Pier cap = 95.42 t

Pier shaft = 132.71 t

Total load = 1109.21 t Say, 1109.21 t

2 Longitudinal moment -

Due to Longitudinal force = 0.000 x 8.819 = 0 t-m

= 0.00 t-m 0.00 t-m

	P	M_L	M_T
Max Rea	1109.21	0.00	0.00
Max Long	1109.2	0.00	0.00

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

1 Vertical load -

S.I.D.L. = 197.21 t

Dead load = 683.87 t

Pier cap = 95.42 t

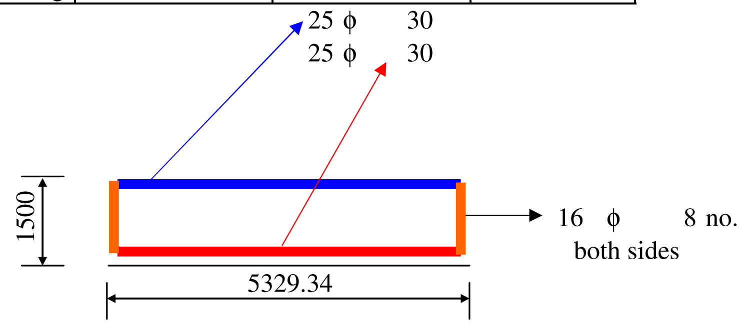
Pier shaft		= 78.204 t			
	Total load	= 1054.71 t	Say,	1054.71 t	

2 Longitudinal moment -

Due to Longitudinal force	=	0.000	x	8.819	=	0	t-m
Due to earth					=	0.00	t-m
					=	0.00	t-m

0.00 t-m

	P	M_L	M_T
Max Rea	1054.71	82.77	0.00
Max Long	1054.7	82.77	0.00



PLAN : PIER SHAFT

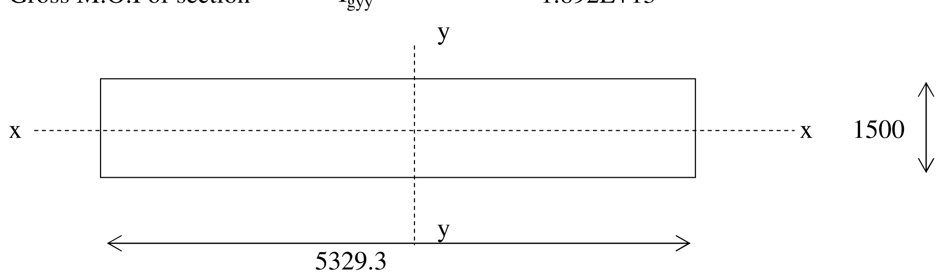
% Reinforcement provided = 32669.422 mm^2

Summary of Loads at Abutment Shaft bottom :

	P	M_L	M_T
1 DRY condition			
With L.L			
Max Rea	1205.89	302.62	236.87
Max Long	1193.13	375.31	244.21
Span dislodged	1109.21	0.00	0.00
2 H.F.L			
With L.L			
Max Rea	1151.39	385.39	264.15
Max Long	1138.63	458.08	271.50
Span dislodged	1054.71	82.77	0.00

Check for Cracked/Uncracked Section

Length of section		=	5329.3404	mm
Width of section		=	1500	mm
Gross Area of section	A_g	=	7994010.6	mm^2
Gross M.O.I of section	I_{gxx}	=	1.499E+12	mm^4
Gross M.O.I of section	I_{gyy}	=	1.892E+13	mm^4

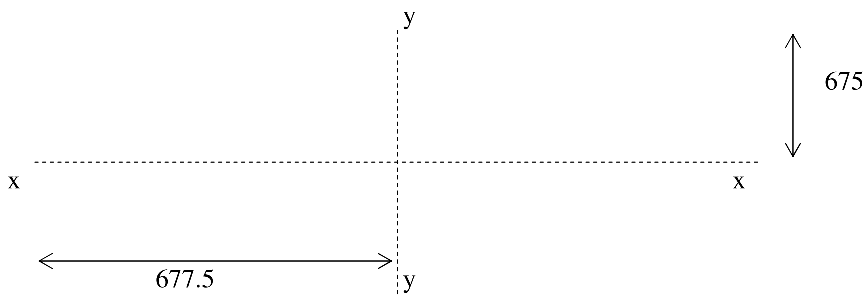


Abutment section

Transformed sectional properties of section:

Adopting

Modular ratio	m	=	10	
Cover			72.5	72.5
Dia of Bars		=	25	25
No of bars in tension face (longer)		=	30	
No of bars in compression face		=		30
No of bars in shorter direction		=		8
Total bars in section		=	76	
Steel Area	A_s	=	37306	mm^2
% of Steel		=	0.4667	%



A_{sx} = -14726 mm^2

A_{sy} = -3927 mm^2

Area of concrete $A_c = A_g - A_s$ = 7956704.2 mm^2

C.G of Steel placed on longer face = 677.5 mm

C.G of Steel placed on shorter face = 2592.2 mm

Transformed Area of Section A_{tfm} = 8329768.3 mm^2

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

= 1.37721E+12 mm^4

$$Z_{xx} = \frac{M.Itxx}{d/2}$$ = 1.836E+09 mm^3

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

= 1.84454E+13 mm^4

$$Z_{yy} = \frac{M.I_{tyy}}{d/2}$$ = 6.922E+09 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.499E+12 mm^4

Area of section = 7994010.6 mm^2

r = 433.0127 mm

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

Pier shaft height L = 7.119 m

Effective length L_{eff} = 12.458 m

Slenderness ratio = 28.771 < **50**

Type of member = Shotr Column

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

β = 1

Permissible stresses : *concrete*

σ_{cbc} = 11.667 N/mm^2

σ_{co} = 8.75 N/mm^2

Tensile stress = 0.67 N/mm^2

Permissible stresses : *Steel*

σ_{st} = 200 N/mm^2

Dry case

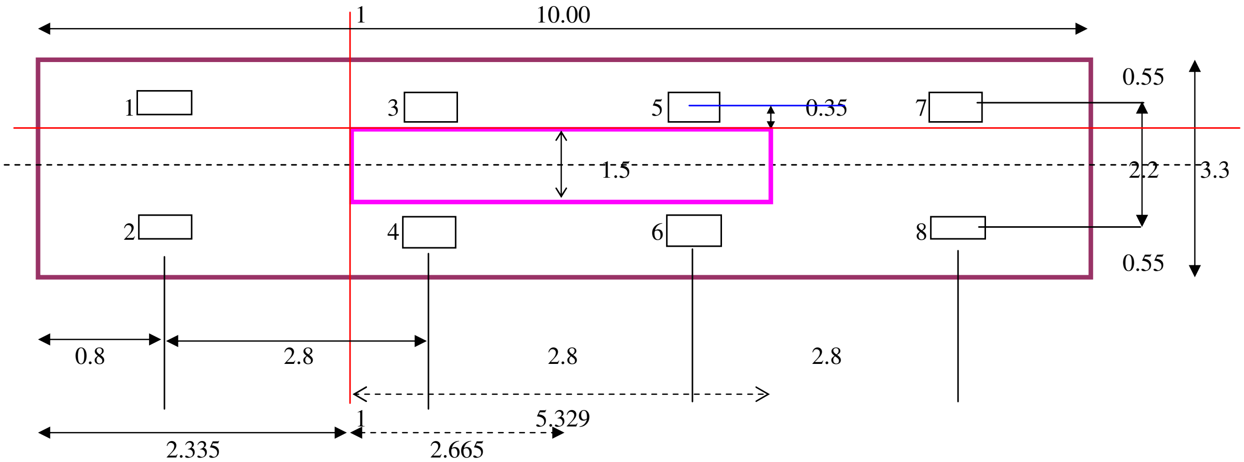
S.No	Item	Max Rea	Max Long	Span Dislodg
	Loads and Moments			
1	P	1205.89 t	1193.13 t	1109.21
2	M_L	302.62 t-m	375.31 t-m	0.00
3	M_T	236.87 t-m	244.21 t-m	0.00
	Actual(calculated) Stresses			
4	$\sigma_{co,cal}$ P/A_{tfm}	1.44769	1.43237	1.33162
5	$\sigma_{cbc,cal}$ M_L/Z_{xx}	1.64803	2.04388	0.00000
6	$\sigma_{cbc,cal}$ M_T/Z_{yy}	0.34218	0.35279	0.00000
7	$\sigma_{cbc,cal}$ = 5 + 6	1.99021	2.39666	0.00000
	Permissible Stresses			
8	σ_{cbc}	11.666667	11.667	11.666667
9	σ_{co}	8.75	8.75	8.75
	Check for Minimum steel area mm^2			
10	Conc.Area Required for directstress (1)/(9)	1378159.3	1363576.4	1267667.8
11	0.8% of area required	11025.274	10909	10141.343
12	0.3% of A_g	23982.032	23982	23982.032
13	Governing steel mm^2	23982.032	23982	23982.032
14	Provided Steel area mm^2	32669.422	32669	32669.422
	Check for safety of section			
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.3360396 < 1	0.3691 < 1	0.1521852 < 1
	Check for Cracked /Uncracked section			
	$\sigma_{co,cal} - \sigma_{cbc,cal}$	-0.542526	-0.9643	1.3316209
	Permissible Basic tensile stress in concrete	-0.67	-0.67	-0.67
	Section to be designed as	**Uncracked**	**Cracked**	**Uncracked**

H.F.L case

S.No	Item		Max Rea	Max Long	Span Dislodg
	Loads and Moments				
1	P		1151.39	1138.63	1054.71
2	M_L		385.39	458.08	82.77
3	M_T		264.15	271.50	0.00
Actual(calculated) Stresses					
4	$\sigma_{co,cal}$	P/A_{tfm}	1.38226	1.36694	1.26619
5	$\sigma_{cbc,cal}$	M_L/Z_{xx}	2.09877	2.49462	0.45074
6	$\sigma_{cbc,cal}$	M_T/Z_{yy}	0.38160	0.39221	0.00000
7	$\sigma_{cbc,cal}$	= 5 + 6	2.48038	2.88683	0.45074
Permissible Stresses					
8	σ_{cbc}		11.666667	11.667	11.666667
9	σ_{co}		8.75	8.75	8.75
Check for Minimum steel area mm^2					
10	Conc.Area Required for directstress	(1)/(9)	1315871.2	1301288.3	1205379.8
11	0.8% of area required		10526.97	10410	9643.0381
12	0.3% of A_g		23982.032	23982	23982.032
13	Governing steel	mm^2	23982.032	23982	23982.032
14	Provided Steel area	mm^2	32669.422	32669	32669.422
Check for safety of section					
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$		0.370576 < 1	0.4037 < 1	0.1833427 < 1
Check for Cracked /Uncracked section					
	$\sigma_{co,cal} - \sigma_{cbc,cal}$		-1.098123	-1.5199	0.8154463
	Permissible Basic tensile stress in concrete		-0.67	-0.67	-0.67
	Section to be designed as		**Cracked**	**Cracked**	**Uncracked**

DESIGN OF PIER CAP

DESIGN OF PIER CAP (along Transverse Direction)



Depth of Pier Cap in Transverse Direction = 1400 mm

considering two layers of 32 ϕ + in longitudinal direction 32 ϕ in transverse

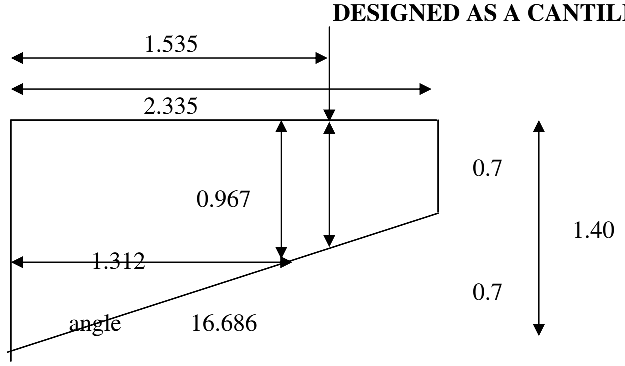
a = distance of cg of bearing from face of equivalent square = 1535 mm

Cover = 40.000 mm

d_{eff} d = 1312 mm

$\frac{a}{d}$ = 1.170221 > 1

DESIGNED AS A CANTILEVER BEAM



Distance of bearing 1 from face of pier = 1.535

Distance of bearing 2 from face of pier = 1.535

Distance of bearing 3 from face of pier = 0

Distance of bearing 4 from face of pier = 0

Loads on bearings:

	Reaction	Ecc	1,2	3,4	5,6	7,8
1 Dead Load	683.8704	0	171.0	171.0	171.0	171.0
2 SIDL	197.2086		49.302	49.302	49.302	49.302
3 LL max Long	83.92	2.91	47.145	29.702	12.258	-5.185

LL reaction with impact factor 50% 1.058901 = 49.92 t

Length of Cantilever at one end = 2.335 m

Length of Cantilever at other end = 2.335 m

Self wt of Cantilever = 19.4206 t

Torsion due to LL Reaction = 54.91412 t-m

Horizantal force due to LL = 8.006744 t

Corresponding moment due horizantal force = 3.736 t-m

Total Torsional Moment = 58.651 t-m

Equivalent bending moment T*(1+D/b)/1.7 = 49.137 t-m

Equivalent Shear	1.6*T/b	=	26.62503 t
Moment at face of pier due to DL		=	338.1867 t-m
Moment at face of pier due to LL		=	76.64662 t-m
Moment at face of pier due to Cantilever load		=	20.157 t-m
Total bending moment		=	**484.127** t-m

M	=	35	N/mm^2
Fe	=	415	N/mm^2
σ_{cbc}	=	11.66667	N/mm^2
σ_{st}	=	200	N/mm^2
m	=	10	
k	=	0.37	
j	=	0.88	
Q	=	1.89	N/mm^2

Depth required $\sqrt{M/Q*b}$ = 0.873 m

OK

Reinforcement required = **21032.97** mm^2

Provide two layers of 32 ϕ 14 nos + 32 ϕ 14 nos in longitudinal direction

= **22518.94** mm^2

Shear force

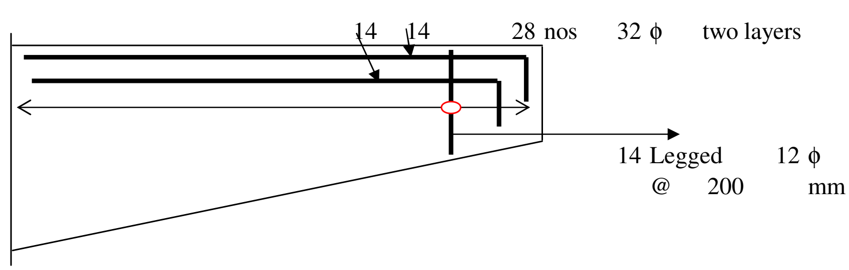
self wt at effective depth d		=	6.98 t
Total Shear force		=	**303.80** t
Shear stress	τ_v = (V-M*Tan(β)/d)/bd	=	48.18 t/m^2
% of reinforcement provided		=	0.706 %
	τ_c	=	31.16 t/m^2
Shear Force carried by Concrete	τ_c*b*d	=	99.46 t
Net design S.F		=	204.34 t

using 14 Legged stirrups at a spacing of 200 mm

Area of steel required /m run = 7634.766 mm^2/m

Providing 12 ϕ 14 legged @ 200 mm

Area of steel provided /m run = 7916.813



CHECK FOR TORSION (one span dislodged condition)

Dead load reaction on bearing		=	85.48 t
Sidl reaction on bearing		=	24.65 t
Horizantal force due to dead load & sidl reaction		=	5.51 t
Torsion	T_u	=	163.55 t-m
	X	=	1.4 m

Y	=	3.3	m
x	=	55.118	inches
y	=	129.921	inches
εX^2*Y	=	394701.6	
f'_c	=	280	kg/cm^2
	=	3984.4	psi
$\phi*(0.5*(f'_c)^{0.5}*\varepsilon X^2*Y)$	=	10588617.424	lb inch
	=	121.875	t-m
	<	163.55	t-m

$A_t/s = T_u/(\alpha_t*x*y*f_y)$

α_t = 0.66 + 0.33 *y/x = 1.437857

A_t/s = = 0.02335 $inch^2/inch$

= 5.93 cm^2/m

DESIGN OF PIER CAP (along Longitudinal Direction)

Depth of pier cap = 1400 mm

considering two layers of 32 ϕ + in longitudinal direction 32 ϕ in transverse

a	=	distance of cg of brg.from face of pier	=	350	mm
b_{eff}		tacking pedestal size for dispersion as 1mx1m	=	1420	mm
d_{eff}			=	1312	mm

$\frac{a}{d}$ = 0.266768

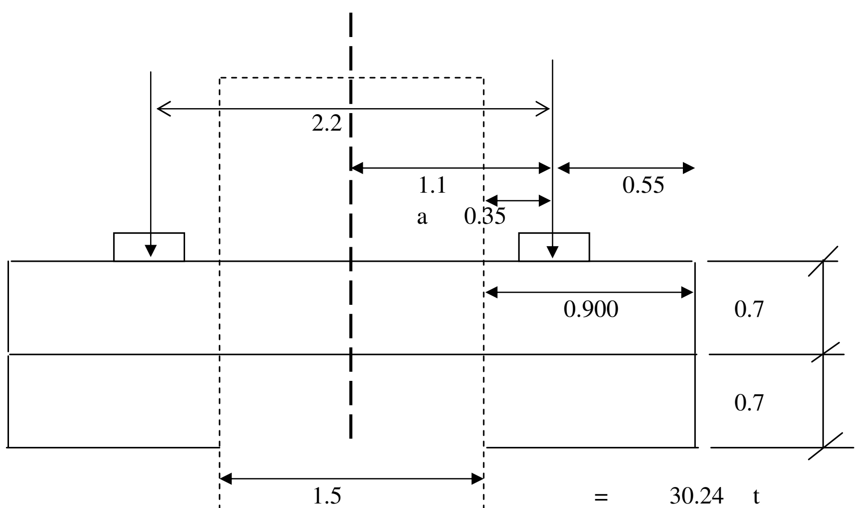
< 1.00

DESIGNED AS A CORBEL

Hence the pier cap has been designed as a bracket.

The design has been done as per "Design proposal for reinforced concrete corbels" by A.H.Mattock.

Dead load reaction coming on corbel	=	341.94	t
Sidl reaction coming on corbel	=	98.60	t
Live load reaction	=	88.86	t



self wt of corbel		=	30.24	t
DL+SIDL+LL + self wt on corbel		=	559.64	t
Ultimate force	V_u 1.4*470.78+1.7*88.86	=	810.1583	t
	(as per load factores given by ACI-4)			
DL+SIDL+LL + self wt on corbel per bearing		=	139.91	t
Ultimate force per bearing	V_u	=	202.5396	t
For Pot-Ptfe bearings	μ'	=	0.05	
Design value of	Nuc	=	10.13	t

Check for Nominal Shearstrength

ϕ	=	0.85	
μ	=	1.4	
$V = V_u/\phi*b*d$	=	127.90	t/m^2
$< 0.2\ F'_c$	=	571.2	t/m^2
$F'_c = 0.8\ f'_c$		**O.K**	

Calculation for Shear Friction Reinforcement

$A_{vf} = V_u/\phi*\mu*f_y$ = 41.012 cm^2

clause 11.9.3.3

Calculation of flexural reinforcement

$M_u = V_u*a+N_{uc}*(D-d_{eff})$	=	71.78	t-m
$A_f = M_u/\phi*f_y*(d_{eff}-x/2)$	=	15.53	cm^2

Where x = depth of rectangular stress block

assume x = 1.876458 cm

Depth of Rectangular Block

$x = A_f*f_y/\phi*f'_c*b_{eff}$ = 1.90725 cm

Calculation for Horizantal Tensile Steel

$A_t = N_{uc}/(\phi*f_y)$ or	=	2.870866	cm^2
$= 2/3*A_{vf}$	=	27.34158	cm^2
$= A_f$	=	15.53	cm^2
whichever is maximum	=	**27.34**	cm^2

Minimum reinforcement as per clause 11.9.5 of ACI-4

$= 0.04*f'_c*b_{eff}*d_{eff}/f_y$ = **50.280** cm^2

Calculation for Torsional Steel

In flexural steel, reinforcement due to torsion is to be added also and to compared with minimum steel requirement.

Torsion due to horizantal force as a result of the live load reaction coming on the bearing.

Horizantal force	=	2.496	t
Bending moment	=	2.745706	t-m
Ultimate torsion	=	4.6677	t-m
X	=	1.4	m
Y	=	1.312	m
x	=	55.118	inches
y	=	51.654	inches
εX^2*Y	=	156923.8	
f'_c	=	280	kg/cm^2
	=	3984.4	psi
$\phi*(0.5*(f'_c)^{0.5}*\varepsilon X^2*Y)$	=	4209777.594	lb inch
	=	48.45454	t-m
	>	4.67	t-m

(Thus reinforcement is provided to cater to the effect of full torsion,neglecting the effect of concrete)

$A_t/s = T_u/(\alpha_t*x*y*f_y)$

α_t = 0.66 + 0.33 *y/x = 0.969257

A_t/s = = 0.025812 $inch^2/inch$

= 6.56 cm^2/m

Longitudinal reinforcement as per ACI:

A_s = $2*A_t*(X1+Y1)/s$ = 33.883 cm^2

Total Steel = 27.342 + 33.88 = **61.225** cm^2 per 1.42

Therefore this is more than minimum steel.

Hence A_s design = 431.159 cm^2

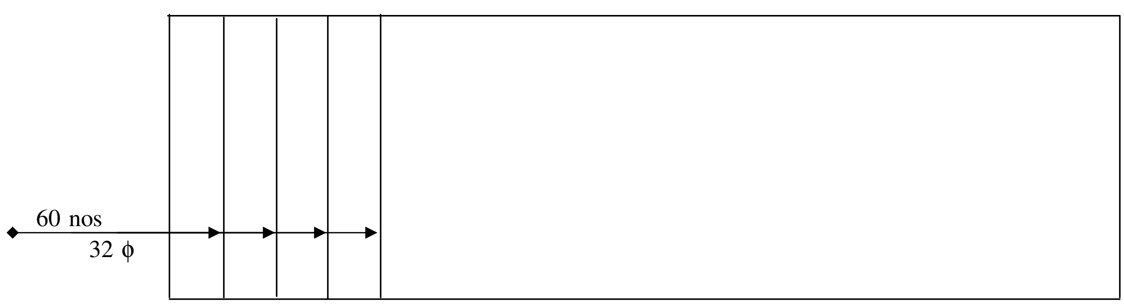
Provide 60 nos 32 ϕ

Area of steel provided = 482.5486 cm^2

Horizantal Stirrups = $0.5*A_s$ = 214.1441 cm^2

Provide **20** nos **2** Legged **16** ϕ in **3** Layers

Area of steel provided = **241.2743** cm^2



```
   PIER SHAFT .case 1

   Depth of Section             =     1.500 m
   Width of Section             =     5.329 m

along width-compression face- no of bar:  30   tension face- no of bar:  30
Dia (mm)                                  25                             25
Cover (cm)                              7.50                            7.5
along depth-compression face- no of bar:   8   tension face- no of bar:   8
Dia (mm)                                  16                             16
Cover  (cm)                             7.50                            7.5

   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                   =  1225.35 T
   Mxx                          =   335.620 Tm
   Myy                          =   236.870 Tm


   Intercept of Neutral axis : X axis : =  27.149 m
                             : y axis : =   1.616 m

             Concrete Stress Governs Design
   Stress in Concrete due to Loads =   33.74 Kg/cm^2
   Stress in Steel    due to Loads =    25.47 Kg/cm^2
   Percentage of Steel             =     .41 %


   PIER SHAFT .case 2

   Depth of Section             =     1.500 m
   Width of Section             =     5.329 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                   =  1212.590 T
   Mxx                          =   408.310 Tm
   Myy                          =   244.210 Tm


   Intercept of Neutral axis : X axis : =  26.643 m
                             : y axis : =   1.428 m
`            Concrete Stress Governs Design

   Stress in Concrete due to Loads =   37.94 Kg/cm^2
   Stress in Steel    due to Loads =   74.00 Kg/cm^2
   Percentage of Steel             =     .41 %
```

```
       PIER SHAFT .case 1

       Depth of Section             =     1.500 m
       Width of Section             =     5.329 m

   along width-compression face- no of bar:  30   tension face- no of bar:  30
   Dia (mm)                                  25                             25
   Cover (cm)                               7.50                           7.5
   along depth-compression face- no of bar:   8   tension face- no of bar:   8
   Dia (mm)                                  16                             16
   Cover  (cm)                              7.50                           7.5

       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                   =  1172.850 T
       Mxx                          =   437.130 Tm
       Myy                          =   270.330 Tm

       Intercept of Neutral axis : X axis : =  23.847 m
                                 : y axis : =   1.346 m

                 Concrete Stress Governs Design

       Stress in Concrete due to Loads =   35.66 Kg/cm^2
       Stress in Steel    due to Loads =   67.03 Kg/cm^2
       Percentage of Steel             =     .41 %

       PIER SHAFT .case 2
23-11-06

       Depth of Section             =     1.500 m
       Width of Section             =     5.329 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                   =  1160.090 T
       Mxx                          =   509.820 Tm
       Myy                          =   277.670 Tm

       Intercept of Neutral axis : X axis : =  23.171 m
                                 : y axis : =   1.160
                 Concrete Stress Governs Design
       Stress in Concrete due to Loads =   46.53 Kg/cm^2
       Stress in Steel    due to Loads =  211.80 Kg/cm^2
       Percentage of Steel             =     .41 %
```

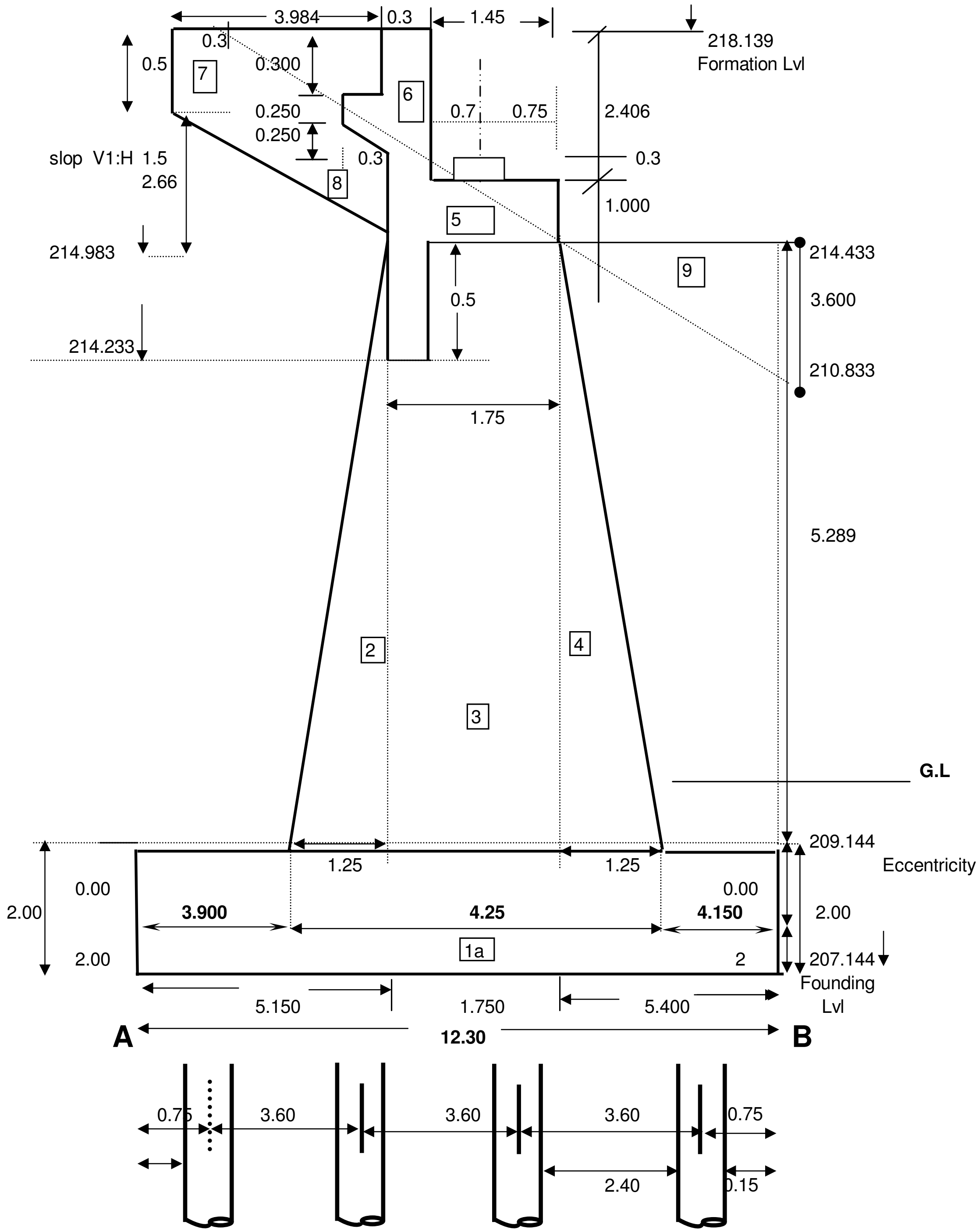
```
INPUT FILE: 70RW.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. JOINT COORDINATES
     6. 1          0.000         .000          .000
     7. 2          0.4           .000          .000
     8. 3         30.4           .000          .000
     9. 4         30.8           .000          .000
    10. 5         30.84         0.0           0.0
    11. 6         31.24          .000          .000
    12. 7         61.24         0.0           0.0
    13. 8         61.64         0             0
    14. MEMBER INCIDENCES
    15. 1         1         2
    16. 2         2         3
    17. 3         3         4
    18. 4         4         5
    19. 5         5         6
    20. 6         6         7
    21. 7         7         8
    22. MEMBER PROPERTY CANADIAN
    23. 1 TO 7 PRI YD 2. ZD 1.
    24. MEMBER RELEASE
    25. 4 END  FY MZ
    26. 5 START FY MZ
    27. CONSTANT
    28. E CONCRETE ALL
    29. DENSITY CONCRETE ALL
    30. POISSON CONCRETE ALL
    31. SUPPORT
    32. 2 3 6 7  PINNED
    33. DEFINE MOVING LOAD
    34. *TYPE 1 LOAD 4*17.0 2*12.0 8.0 DIS 1.37 3.05 1.37 2.13 1.52 3.96
    35. TYPE 1 LOAD 8 2*12 4*17  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    36. LOAD GENERATION 264
    37. TYPE 1 -100.2 0. 0. XINC .5
    38. TYPE 1 -56.8 0. 0. XINC .5
    39. TYPE 1 -13.4 0. 0. XINC .5
    40. PERFORM ANALYSIS
    41. ***MAX REA
    42. LOAD LIST 159
    43. PRINT SUPPORT REACTION
SUPPORT  REACTION
    SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z

      2  159      0.00      5.30      0.00      0.00      0.00      0.00
      3  159      0.00     43.70      0.00      0.00      0.00      0.00
      6  159      0.00     46.87      0.00      0.00      0.00      0.00
      7  159      0.00      4.13      0.00      0.00      0.00      0.00
      44. LOAD LIST 236
      45. PRINT SUPPORT REACTION

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

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

     2  236      0.00     16.08      0.00      0.00      0.00      0.00
     3  236      0.00     83.92      0.00      0.00      0.00      0.00
     6  236      0.00      0.14      0.00      0.00      0.00      0.00
     7  236      0.00      7.86      0.00      0.00      0.00      0.00

   46. FINISH

            *********** END OF THE STAAD.Pro RUN ***********
```

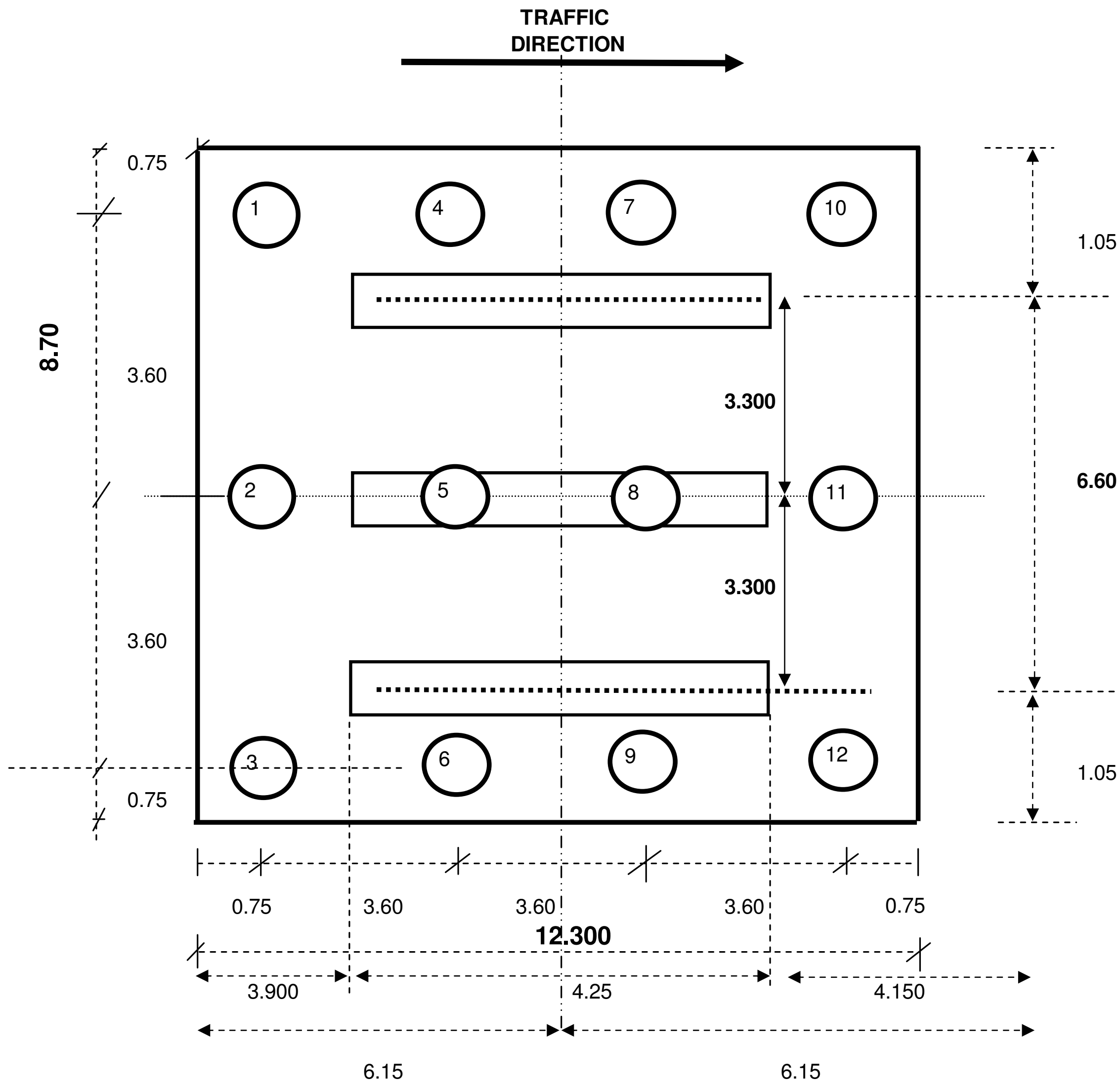
DESIGN OF SUBSTRUCTURE (ABUTMENT)

DESIGN DATA

Formation Level	=	218.139	m
Ground Level	=	210.144	m
Lowest Water Level	=	210.144	m
Highest Flood Level	=	214.172	m
Bottom of pile cap Level	=	207.144	m
Thickness of bearing & pedestal	=	0.300	m
Length of Span expansion joint to expansion joint	=	32.200	m
Clear Span	=	30.000	m
Bouyancy factor	=	1	
Submerged density of earth	=	1.0	t/cum
Saturated density of earth	=	2.000	t/cum
Unit wt of concrete	=	2.400	t/m^3
Depth of Superstructure	=	2.350	m
Tkickness of wearing coat	=	0.056	m
PILE DATA			
Number of piles	=	12	
Diameter of pile	=	1.20	m
Spacing of pile in longitudinal direction	=	3.60	m
No. of spacing in longi. dir.	=	3	
Specing of pile in trans. dir.	=	3.60	m
No. of specing in trans. dir.	=	2	
SEC MODULUS FOR LONG. MOMENTS, Z_L outer	=	36.00	m^3
SEC MODULUS FOR LONG. MOMENTS, Z_L inner	=	108.00	m^3
SEC MODULUS FOR TRANS. MOMENTS, Z_T outer	=	28.80	m^3
SEC MODULUS FOR TRANS. MOMENTS, Z_T inner	=	0.00	m^3
LENGTH OF PILE CAP IN LONG. DIRECTION	=	12.30	m
LENGTH OF PILE CAP IN TRANS. DIRECTION	=	8.70	m
Live Load	-	One Lane of 70R Two Lane of class A	
Grade of Concrete	-	M 35	
Grade of Reinforcement	-	415 (HYSD)	
Permissible Compressive stress in Concrete	-	1190	t/m^2
Permissible Tensile stress in Steel	-	20400	t/m^2
Modular ratio, m	-	10	
factor, k	-	0.368	
Lever arm factor, j	-	0.877	
Moment of Resistance	-	192	t/m^2

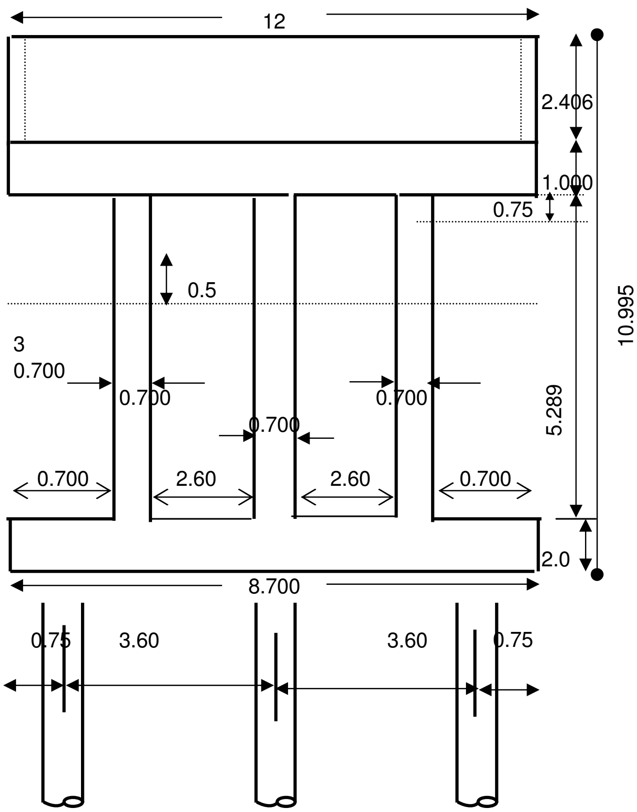


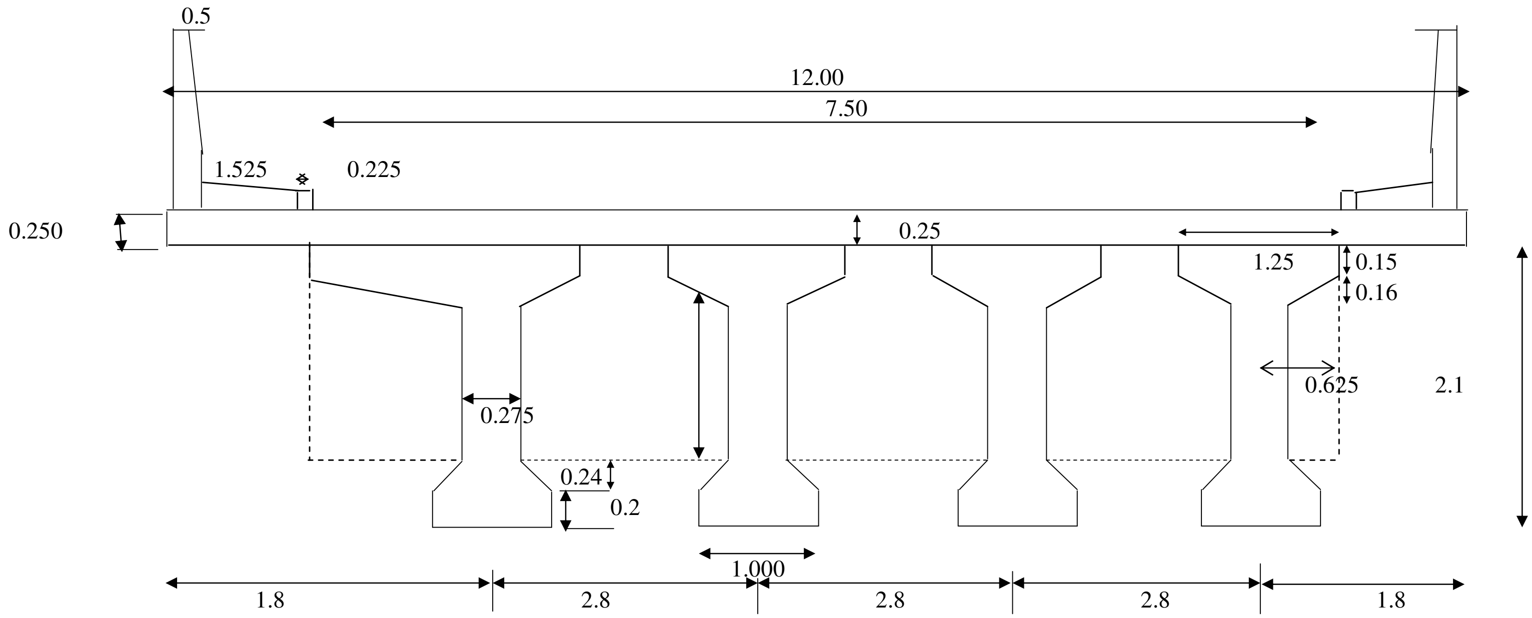
3.984
0.3
1.45
218.139
Formation Lvl
0.3
0.5
0.300
7
6
0.250
0.7
0.75
2.406
0.250
slop V1:H 1.5
0.3
0.3
2.66
8
1.000
5
214.983
214.433
9
3.600
0.5
214.233
210.833
1.75
5.289
2
4
3
G.L
209.144
1.25
1.25
Eccentricity
0.00
0.00
2.00
3.900
4.25
4.150
2.00
1a
2.00
2
207.144
Founding
Lvl
5.150
1.750
5.400
A
12.30
B
0.75
3.60
3.60
3.60
0.75
2.40
0.15



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

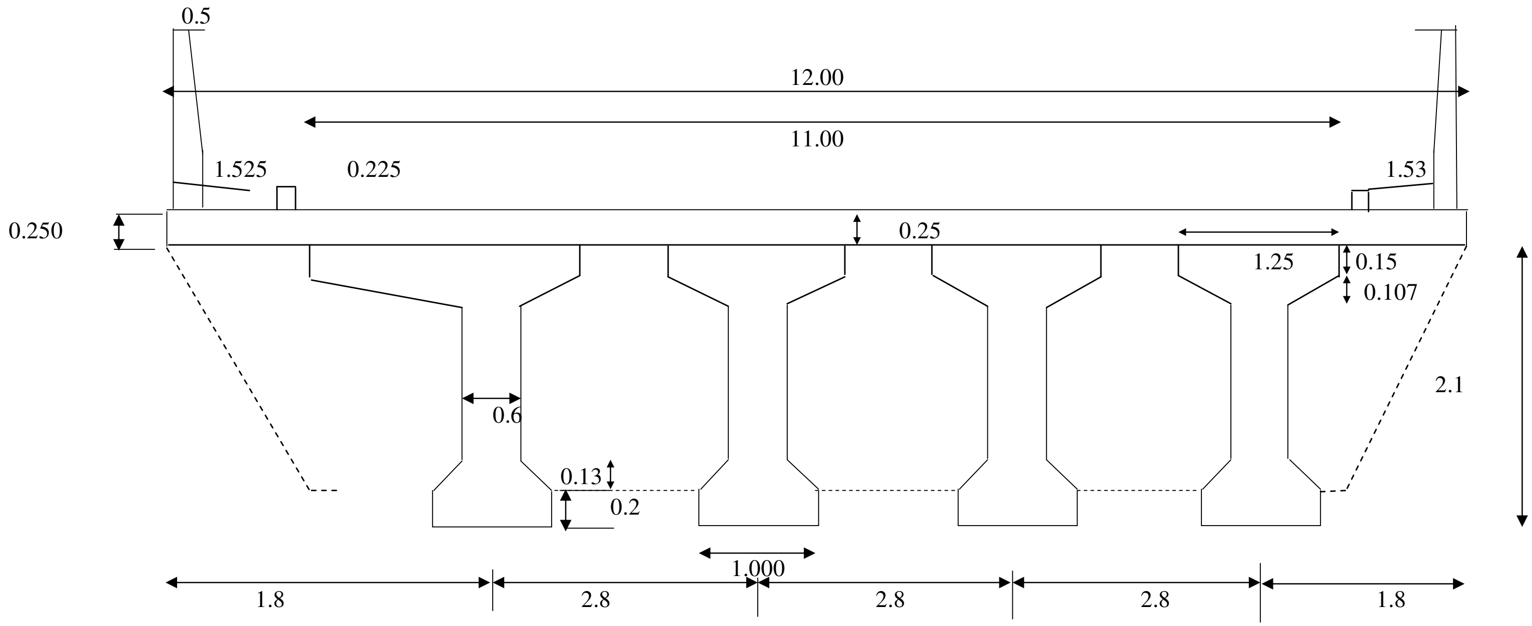
No. of column = 3
Width = 0.700





Depth of SuperStructure			=	2.35	
Number of Longitudinal Girders			=	4	
Mid section					
1 slab	=	12*0.25	=	3	m^2
2 Top Flange	=	4*[(0.15*1.25)+(0.5*(1.25+0.275))*0.16	=	1.238	m^2
3 Web	=	4*[(2.1-(0.15+0.16)-(0.2+0.24))*0.275]	=	1.485	m^2
4 Bottom Bulb	=	4*[(1*0.2)+(0.5*(1+0.275))*0.24	=	1.412	m^2
Total			=	**4.135**	m^2

End Section

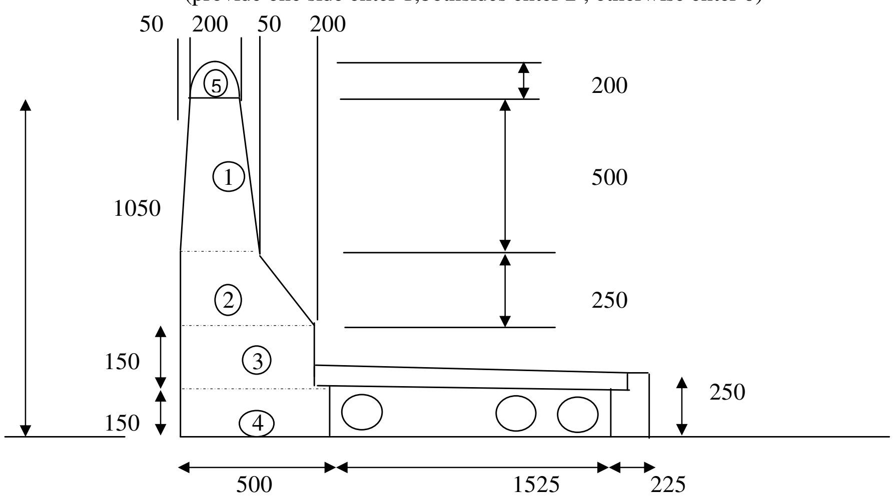


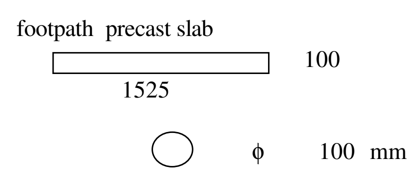
Depth of SuperStructure		=	2.35
Number of Longitudinal Girders		=	4

End section

1	slab	= 12*0.25	=	3	m^2
2	Top Flange	= 4*[(0.15*1.25)+(0.5*(1.25+0.6))*0.107	=	1.1459	m^2
3	Web	= 4*[(2.1-(0.15+0.107)-(0.2+0.132))*0.6]	=	3.6264	m^2
4	Bottom Bulb	= 4*[(1*0.2)+(0.5*(1+0.6))*0.132	=	1.2224	m^2
	Total		=	**5.99**	m^2
	DL per running meter at Intermediate section		=	10.338	t/m
	DL per running meter at End section		=	14.987	t/m

Intermediate Cross Girder				
Thickness of Cross Girder		=	0.3	
Number of intermediate cross girders		=	3	
Cross sectional area in elevation		=	12.259	m^2
Dl of intermediate cross girder		=	**27.582**	t
End Cross Girder				
Thickness of end Cross Girder		=	0.45	
Number of intermediate cross girders		=	2	
Cross sectional area in elevation		=	18.028	m^2
Dl of intermediate cross girder		=	**40.562**	t
S.I.D.L				
I Wearing Coat @ 1.0725 t/m		=	1.0725	t/m
II Crash barrier @ 1.00 t/m				
(provide one side enter 1,bothsides enter 2 , otherwise enter 0)	2	=	2.00	t/m
III R.C.C Post & Railing @ 0.085 t/m				
(provide one side enter 1,bothsides enter 2 , otherwise enter 0)	0	=	0	t/m





Wt due to Crashbarrier

Component		Area	Wt
1		0.100	0.24
2		0.0875	0.21
3		0.075	0.18
4		0.075	0.18
5		0.002	0.1541
			0.96 t/m

say, 1.000

Footpath precast slab

0.1525 0.3813 t/m

missllaneous utilities 0.2 t/m

Footpath kerb 0.0563 0.1406 t/m

sandfilling 0.2052 0.3693 t/m

IV Foot path precast s @ 0.38125 t/m

(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.7625 t/m

V Footpath Kerb @ 0.140625 t/m

(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.2813 t/m

VI Missillaneous utilities @ 0.2 t/m

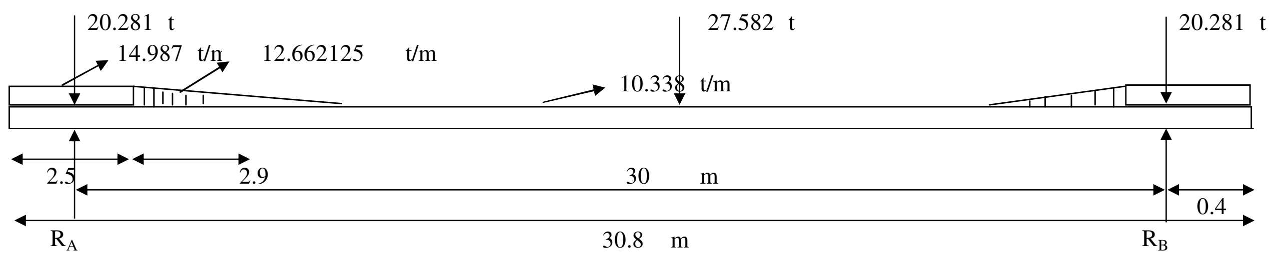
(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.4 t/m

VII Sandfilling in footpath @ 0.3693 t/m

(provide one side enter 1,bothsides enter 2 , otherwise enter 0) 2 = 0.7387 t/m

Total = 5.25 t/m

Total S.I.D.L Reaction = 169.21 t



Due to girders:

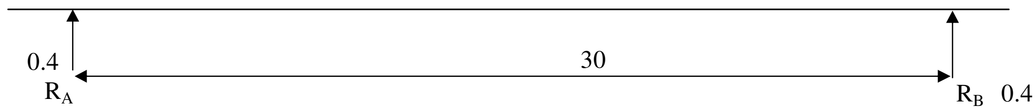
Reaction R_A = (10.3375*0.5*25.4)+(14.99*2.5)+(0.5*12.66*2.9)+20.28+13.79)

= 221.19 t

Due to SLAB = 120.75 t

Due to SIDL = **84.604** t

Total DL + SIDL = **853.08** t



Longitudinal Force :

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

For Class A Single lane : F_h = (0.2×55.4) = 11.080 t

For class 70R wheeled : F_h = (0.2×100) = 20.000 t

For class A 2 lane F_h = 0.2×55.4 = 11.08 t

Summary of Longitudinal Forces :

	Fixed Bearing	Free Bearing
1	$F_h - \mu (R_g+R_q)$	$\mu (R_g+R_q)$
2	$\frac{F_h}{2} + \mu (R_g+R_q)$	$\mu (R_g+R_q)$

1 $= 20.0 - 0.05 \times 457.29$ 0.05×523.79

$= -2.8645$ t $= 26.189$ t

2 $\frac{20.0}{2.0} + 0.05 \times 457.29$ 0.05×523.79

32.864 t $= 26.189$ t

Load Case	Longitudinal horizontal force (t)	Longitudinal Moment (t-m)
70R Wheeled	**32.86**	**0.000**
Span dislodged	**0.00**	**0.000**

Moment in Transverse Direction

Eccentricity of vertical load in transverse direction,

(a) Class A Single lane $= \frac{11.0}{2.00} - 0.40 - \left[\frac{1.80}{2}\right] = 4.2$

(b) Class A 2 lane $= \frac{11.0}{2.00} - 0.40 - \left[\frac{1.80 \times 2 + 1.7}{2}\right] = 2.45$

(c) Class 70R Wheeled

$= \frac{11.0}{2.00} - 1.63 - \left[\frac{1.93}{2}\right] = 2.91$

Footpath Live load :

The effective span of girders = 32.200 m > 30.00 m

Therefore, Intensity of footpath live load $= P = \left(P' - 260 + \frac{4800}{L}\right)\left(\frac{16.5 - W}{15}\right)$

Where, P' = 400 Kg/m^2

Therefore, $\left(400 - 260 + \frac{4800}{32.2}\right)\left(\frac{16.5 - 1.5}{15}\right)$

= 289.07 Kg/m^2 = 0.289 t/m^2

(a) When one side is loaded;

Load on each span = 0.289 x 1.5 x 32.200 = 13.962 t Say, 14.0 t

(b) When both sides are loaded;

Load on each span = 13.962 x 2 = 27.924 t Say, 28.0 t

Eccentricity of load in transverse direction:

(a) When one side is loaded $= 6 - \left(0.45 + \frac{1.5}{2}\right) = 4.80$ m

(b) When both sides loaded = Nil

LOADING:		R_A (t)	R_B(t)		M_T	R
Class A 1 Lane		13.62	41.78		175.48	41.78
Class A 2 Lane		27.24	83.56		**204.72**	**83.56**
70R Wheeled		16.75	83.25		242.26	83.25

SUMMARY OF LOAD ON PIER FROM SUPERSTRUCTURE

	P	M_L	M_T
1 D.L	341.9351813	0	0
2 S.I.D.L	98.60432467	0	0
3 L.L			
	83.25	0	242.26

COEFFICIENT OF ACTIVE EARTH PRESSURE

AS PER COULOMB'S THEORY, COEFFICIENT OF ACTIVE EARTH PRESSURE IS

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

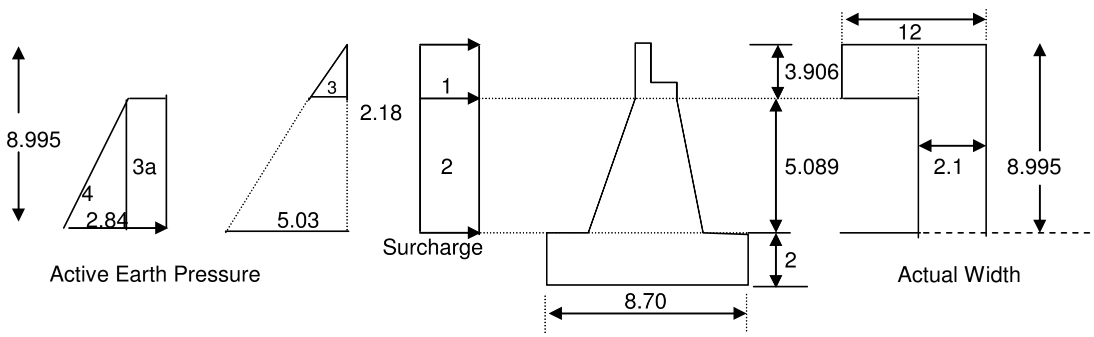
WHERE

ϕ = ANGLE OF INTERNAL FRICTION OF EARTH
α = ANGLE OF INCLINATION OF BACK OF WALL
δ = ANGLE OF INTERNAL FRICTION BETWEEN WALL & EARTH
ι = ANGLE OF INCLINATION OF BACKFILL

HERE	ϕ =	30 °	=	0.524	Radian
	α =	90 °	=	1.571	Radian
	δ =	20 °	=	0.349	Radian
	ι =	0 °	=	0	Radian
			K_a =	0.2973	

Therefore, Horizontal coefficient of Active earth pressure **$K_aCOS\phi$** = **K_{ha} =** **0.2794**

CALCULATION OF ACTIVE EARTH PRESSURE



L.W.L. Condition

a) Service Condition

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Actual Width	Effective Width*	Force (t)	L.A. from bottom of pile cap	Moment (tm)
1	LL Surcharge (height=1.2)	1.0	3.906	0.67	12.000	12.000	31.429	9.042	284.18
2		1.0	5.089	0.67	2.100	2.100	7.166	4.545	32.56
3	Dry Earth	0.5	3.906	2.18	12.000	12.000	51.150	8.730	446.52
3a		1.0	5.089	2.84	2.100	4.200	60.778	4.545	276.20
4		0.5	5.089	2.84	2.100	4.200	30.389	4.137	125.73
TOTAL							**180.91**		**1165.19**

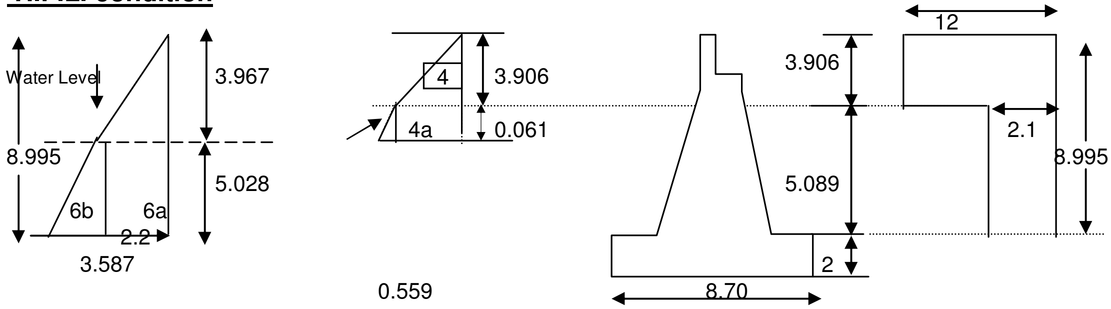
Design Force	180.91	t
Design Moment	1165.19	tm

b) Span Dislodge Condition

Net force =	180.91	-	38.59	=	**142.32**	t
Net moment =	1165.19	-	316.74	=	**848.45**	tm

* As per clause 710.4.3 of IRC : 78,

H.F.L. condition



Active Earth Pressure

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Actual Width	Effective Width	Force (t)	L.A. from bottom of pile cap	Moment (tm)
1+2	Surcharge	Same as in dry case					38.594		316.74
4	Dry Earth	0.5	3.906	2.183	12.000	7.800	33.248	8.730	290.235
4a		0.5	3.967	2.183	2.100	4.200	18.182	9.012	163.848
6a	Submg Earth	1.0	5.028	2.183	2.100	4.200	46.090	4.514	208.051
6b		0.5	5.028	1.405	2.10	4.200	14.832	4.112	60.987
TOTAL							**150.95**		**1039.86**

Design Force	150.95	t
Design Moment	1039.86	tm

b) Span Dislodge Condition

Net force =	150.95	-	38.59	=	**112.35**	t
Net moment=	1039.86	-	316.74	=	**723.12**	tm

Weight of Abutment

L.W.L. Condition

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	L.A.from pile cap B	Moment about B
1a	Pile cap	1.0	12.300	8.70	2.000	2.40	513.65	6.150	3158.94
2	Counterfort	0.5	1.250	2.10	5.289	2.40	16.66	7.567	126.06
3		1.0	1.750	2.10	5.289	2.40	46.65	6.275	292.72
4		0.5	1.250	2.10	5.289	2.40	16.66	4.983	83.02
5	Cap	1.0	1.750	12.00	1.000	2.40	50.40	6.275	316.26
6	Dirt Wall	1.0	0.300	12.00	2.406	2.40	20.79	7.000	145.51
7	Return Wall	1.0	3.984	0.90	0.500	2.40	4.30	9.142	39.34
8		0.5	3.984	0.90	2.656	2.40	11.43	8.478	96.89
TOTAL							**680.54**		**4258.7**

Weight of Earth and LL surcharge

L.W.L. Condition

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	L.A.from pile cap B	Moment about B
Front Earth below cap		1	7.15	8.7	5.289	2.000	658.0045	3.575	2352.366
9	VOLUME DEDUCTED	-0.5	5.400	8.70	3.600	2.0	-169.13	1.800	-304.43
4		-0.5	5.400	2.10	5.289	2.0	-59.98	3.600	-215.92
3		-1.0	1.750	2.10	5.289	2.0	-38.87	6.275	-243.94
Backfill Earth		1.0	5.150	8.70	8.995	2.0	806.04	9.725	7838.76
2	VOLUME DEDUCTED	-0.5	1.250	2.10	5.289	2.0	-13.88	7.567	-105.05
7		-1.0	3.984	0.90	0.500	2.0	-3.59	9.142	-32.78
8		-0.5	3.984	0.90	2.656	2.0	-9.52	8.478	-80.74
TOTAL							**1169.07**		**9208.3**

LLSURCHARGE

Element No	Component	Area Factor	Length	Width	Height	Density	Weight	L.A.from pile cap B	Moment about B
DRY EARTH		1.00	5.150	8.70	1.20	2.00	**107.53**	9.725	**1045.75**

BOUYANCY FACTOR = 1

BOUYANT FORCE (H.F.L. Case 12.30 X 8.70 **X** 7.028 = 752.0663

MOMENT DUE TO BOUYANT FORCE @, TOE = 4625.208

SUMMARY OF FORCES AND MOMENTS:

LOAD CASE	**Case. L.W.L.** Service Cond.	Span dislodged	**Case. H.F.L.** Service Cond.	Span dislodged
CLASS-70R	83.25		83.25	
Foot Path L.L.	0.00		0.00	
D.L. of superstructure	341.94		341.94	
S.I.D.L.	83.25		83.25	
Total Vertical Load from supr str (a)	508.44	0.00	508.44	0.00
L.L. Surcharge	107.53		107.53	
Vertical load from substructure (b)	1957.14	1849.61	1205.08	1097.54
Total Vertical Load V = (a) + (b)	**2465.58**	**1849.61**	**1713.51**	**1097.54**
Horizontal force at bearing	32.86	0.00	32.86	0.00
Horiz force due to active Earth Pressure	180.91	142.32	150.95	112.35
Total Horizontal Force H =	**213.78**	**142.32**	**183.81**	**112.35**
Moment, due to Horiz. force at bearing	292.13	0.00	292.13	0.00
`eccen' due to C.G.of pile & CLof bearing	0.000	0.000	0.000	0.000
M_L @ C.G. of pile group due to (a)	0.00	0.00	0.00	0.00
`eccen' due to C.G.of pile & CG of (b)	1.265	1.131	2.055	1.906
M_L @ C.G. of pile group due to (b)	2476.33	2091.91	2476.33	2091.91
Moment due to active earth pressure	1165.19	848.45	1039.86	723.12
Net Longitudinal moment	-1019.006	-1243.454	-1144.3	-1368.783
Transverse moment	242.258	0.000	242.3	0.000

Seismic Calculation

Seismic forces in longitudinal & transverse direction shall be generated only on superstructure and do not govern for Zone II & III,the design hence ignored.

CALCULATION OF LOAD ON PILES

Total no. of Piles (**N**) 12

P	2465.58	Z_L (outer)	36.00	Z_L (inner)	108.00
M_L	-1019.006				
M_T	242.258	Z_T (outer)	28.80	Z_T (inner)	0.00

L.W.L. Case

SERVICE CONDITION					SPAN DISLODGE CONDITION				
P/N	**M_L/Z_L (outer)**	**M_L/Z_L (inner)**	**M_T/Z_T (outer)**	**M_T/Z_T (inner)**	**P/N**	**M_L/Z_L (outer)**	**M_L/Z_L (inner)**	**M_T/Z_T (outer)**	**M_T/Z_T (inner)**
205.46	-28.31	-9.44	8.41	0.00	154.13	-34.54	-11.51	0	0

TRAFFIC DIRECTION

PILE NO.	A	B	C	D	A	B	C	D
1	**225.36**	206.49	187.62	168.75	**188.67**	165.65	142.62	119.59
2	233.77	214.90	196.03	177.16	188.67	165.65	142.62	119.59
3	242.18	223.31	204.44	185.57	188.67	165.65	142.62	119.59

H.F.L. Case

SERVICE CONDITION					SPAN DISLODGE CONDITION				
P/N	**M_L/Z_L (outer)**	**M_L/Z_L (inner)**	**M_T/Z_T (outer)**	**M_T/Z_T (inner)**	**P/N**	**M_L/Z_L (outer)**	**M_L/Z_L (inner)**	**M_T/Z_T (outer)**	**M_T/Z_T (inner)**
142.79	-31.79	-10.60	8.41	0.00	91.46	-38.02	-12.67	0.00	0.00

TRAFFIC DIRECTION

PILE NO.	A	B	C	D	A	B	C	D
1	**166.17**	144.98	123.79	102.59	**129.48**	104.14	78.79	53.44
2	174.58	153.39	132.20	111.01	129.48	104.14	78.79	53.44
3	182.99	161.80	140.61	119.42	129.48	104.14	78.79	53.44

DESIGN OF PILE CAP

NORMAL CASE (L.W.L.)

SERVICE CONDITION				
P/N	**M_L/Z_L (outer)**	**M_L/Z_L (inner)**	**M_T/Z_T (outer)**	**M_T/Z_T (inner)**
205.46	-28.31	-9.44	8.41	0.00

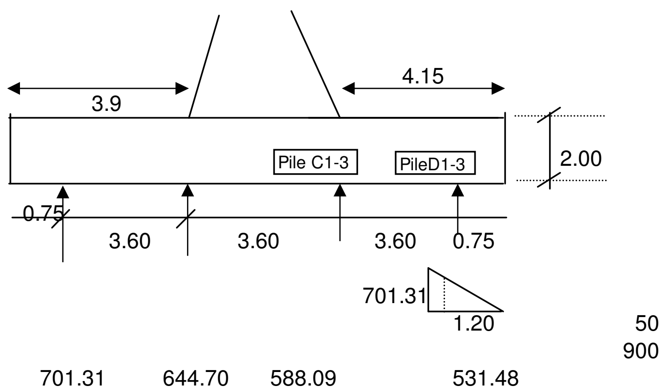
LONGITUDINAL MOMENT

PILE NO.	**A**	**B**	**C**	**D**	Total
1	**225.36**	206.49	187.62	168.75	788.21
2	233.77	214.90	196.03	177.16	821.86
3	242.18	223.31	204.44	185.57	855.51
Total	701.31	644.70	588.09	531.48	

TRANSVERSE MOMENT

TRAFFIC DIRECTION

DESIGNING THE SECTION PARALLEL TO THE TRAFFIC DIRECTION



CALCULATION OF BENDING MOMENT

		0 m from the face of the coloum				
Loadings		**Element**	**Area fact.**	**force/m**	**L.A.**	**Moment**
Downward Loads	SELF WEIGHT OF CONCRETE	1a	1.0	173.304	2.075	359.606
	SELF WEIGHT OF EARTH	4	1.0	381.9	2.075	792.481
	SELF WEIGHT OF EARTH	9	-1.0	-229.869	2.767	-635.970
	SELF WEIGHT OF EARTH	4	1.0	0.000	2.075	0.000
Upward Load of Piles	Pile no. D1,D2,& D3	PileD1-3		-531.477	3.400	-1807.023
	Pile no. C1,C2,& C3	PileC1-3		0.000	-0.200	0.000
TOTAL				-206.123		-1290.905

Bending Moment at face of stem		1290.905	tm			
				M-35		
Effective depth required		0.878	m			
				Q	=	192.29 t/m^2
Effective depth provided		1.910	m			
Area of Reinforcement required		43.4	cm^2/m		=	37769.02 mm^2
Minimum area of steel		24.0	cm^2/m			
Provide	32 ϕ	42	nos			
	16 ϕ	41	nos			
Area of steel provided					=	42021.94 mm^2
					=	49.148 cm^2/m
Provide minimum reinforcement at top face					=	10440 mm^2
	16 ϕ	54	nos		=	10857 mm^2

Check for Shear

1.910 m from the face of the coloum

Loadings		Element	Area fact.	force/m	L.A.	Moment
Downward Loads	SELF WEIGHT OF CONCRETE	1a	1.0	93.542	1.120	104.767
	SELF WEIGHT OF EARTH	4	1.0	206.1	1.120	230.881
	SELF WEIGHT OF EARTH blank	9	-1.0	-99.259	1.120	-111.170
	SELF WEIGHT OF SOIL triang	4	1.0	0.000	1.120	0.000
Upward Load of piles	Pile no. D1,D2,D3	Pile1		-531.477	1.490	-791.901
	Pile no. C1,C2,C3	Pile2		0.000	-2.110	0.000
TOTAL				-331.050		-567.422

Effective depth (d') at distence d 1.910 m

Shear force at critical section 331.05 t

Bending Moment at critical section 567.42 tm

tan β = 0.00

Net shear force S **331.05** t

Hence, Shear stress 19.92 t/m^2

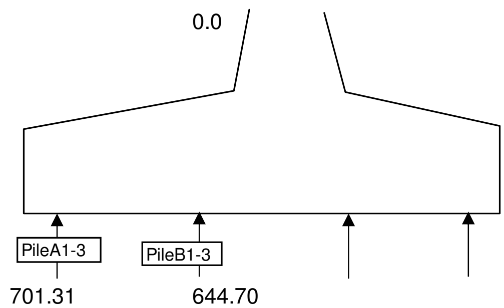
% of reinforcement 0.257

Permissible shear stress 23.68 t/m^2 **Hence O.K., No need of shear reinforcement**

Providing 2 legged 12 ϕ stirrups @, 250 c/c

say **250** c/c

DESIGN OF PILE CAP ON HEEL SIDE



BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

Loadings		Element	Area fact.	force	L.A.	moment
Downward Loads.	SELF WEIGHT OF CONCRETE	1a	1.0	162.864	1.950	317.585
	SELF WEIGHT OF EARTH	2a	1.0	610.401	1.950	1190.281
	SELF WT OF EARTH tring		0.5	0.000	1.950	0.000
	Surcharge			81.432	1.950	158.792
Upward Load of piles	Pile no. B1,B2,B3	Pile B1-3	0.0	0.000	-0.450	0.000
	Pile no. A1,A2,A3	Pile A1-3	0.0	-701.312	3.150	-2209.132
TOTAL				153.385		-542.473

Bending Moment at face of stem **542.473** tm

Effective depth required 0.569 m
Effective depth provided at face of stem 1.910 m

Area of Reinforcement required 18.24 cm^2/m

Hence, Provide 20 ϕ , @ 125 C/C
0 ϕ , @ 125 C/C
Area of steel provided 25.13 cm^2/m % OF STEEL EXTRA 37.77

Check for Shear

(Critical section at face of stem)

Shear force at face of stem 153.39 t

tan β 0.000
Bending moment at face of stem 542.473 tm
Net shear force S-M*tanβ /d 153.39 t

Hence, Shear stress at face of stem 9.23 t/m^2

Shear force at distence 1.910

% of reinforcement 0.13

Permissible shear stress 20.40 t/m^2 **Hence O.K.**

DESIGNING THE SECTION PERPENDICULAR TO THE TRAFFIC DIRECTION

Maximum design moment	256.7
Width taken for design purpose	4.250
Depth required (effective)	0.560
Depth provided (effective)	1.910
Area of steel required per meter of slab cm^2	17.668
Minimum steel required cm^2	24.06
Maximum shear force	0.00
Maximum shear stress	0.00
% of reinforcement	0.126
Permissible shear stress	20.400

Hence O.K., No need of shear reinforcement

Provide 20 ϕ , @ 125 C/C

Area of steel provided 25.13 cm^2

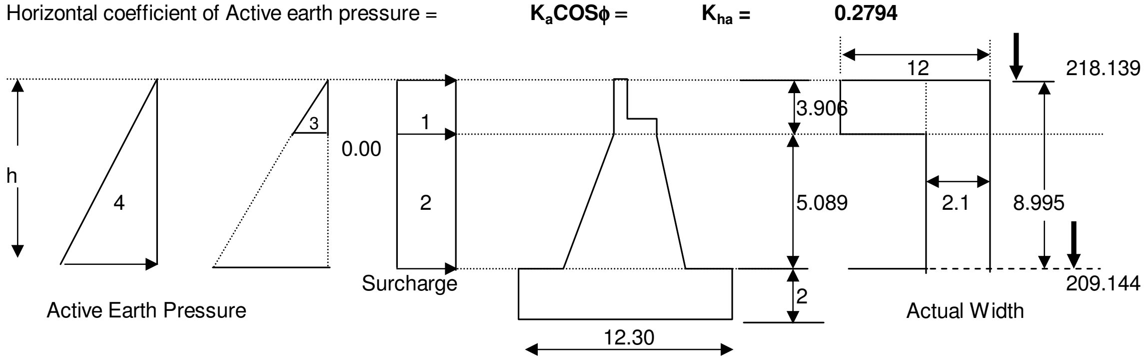
DESIGN OF COLUMN

Case : L.W.L. service condition.

(c) CHECKING THE INTERMEDIATE SECTION, AT R.L.= 209.14 m

CALCULATION OF EARTH PRESSURE

Horizontal coefficient of Active earth pressure = **$K_aCOS\phi$ =** **K_{ha} =** **0.2794**

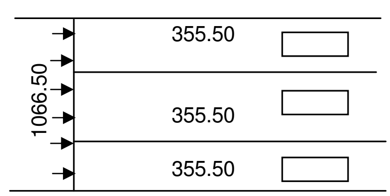


R.L. of Section = **209.14 m**

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Actual Width	Effective Width	Force (t)	L.A. (m)	Moment (tm)
1	Surcharge	1.00	3.906	0.67	12.00	12.00	31.43	7.04	221.3
2		1.00	5.089	0.67	2.10	2.10	7.17	6.45	46.2
3	Dry Earth	0.50	3.91	2.18	12.00	7.80	33.25	6.73	223.7
4		0.50	9.00	5.03	2.10	4.20	94.94	3.78	358.7
Total							166.78		**850.0**

Horizontal force at bearing due to superstructure =	32.86	6.59	216.5

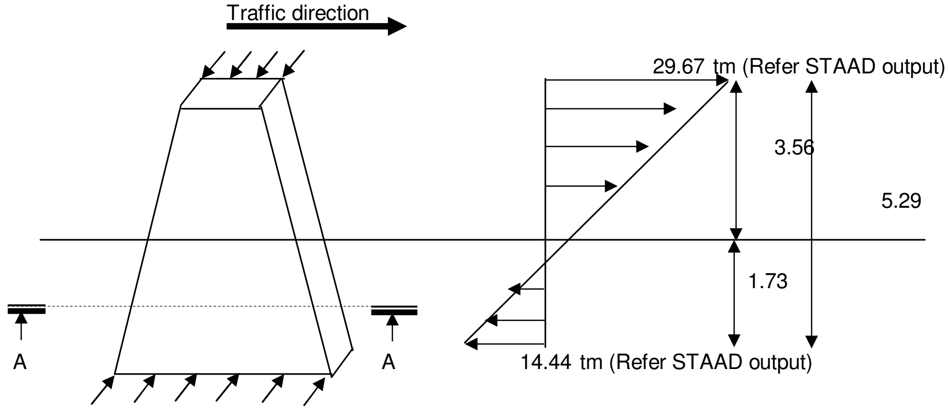
Total longitudinal force & moment on all counterfort 199.65 1066.5



Longitudinal force on each column 66.55 t

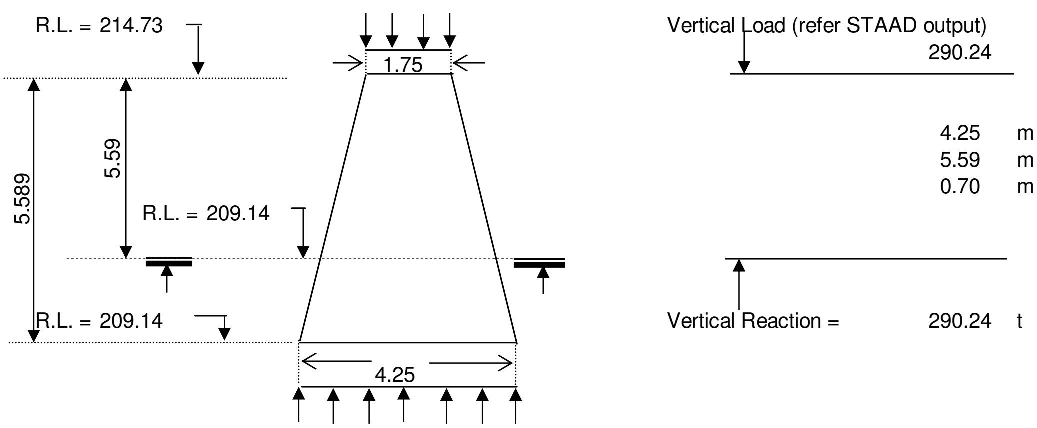
Longitudinal moment on each column 355.50 tm

TRANSVERSE MOMENT



Transverse moment at the section = 16.94 tm

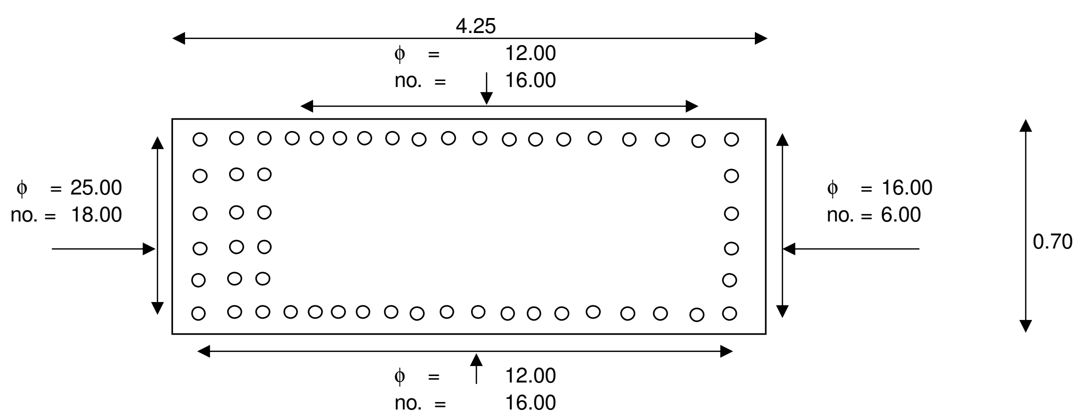
CALCULATION OF VERTICAL LOAD



The design has been done on **BIAX1** software

Input data of forces & moment required to run the **BIAX1**

Cross-section		4.25	x	0.70 m^2
Vertical load	=			290.24 t
Transverse moment	=			16.94 tm
Longitudinal moment	=			355.50 tm



0.297 %

Section A-A

CHECK FOR SHEAR

Checking the section at R.L. =	209.14	m				
Shear Force =	66.55	t				
Area of section	4.25	x	0.70	m^2		
Shear stress	22.37	t/m^2				
% of tensile reinforcement	0.30	%				
Vertical load	290.24	t				

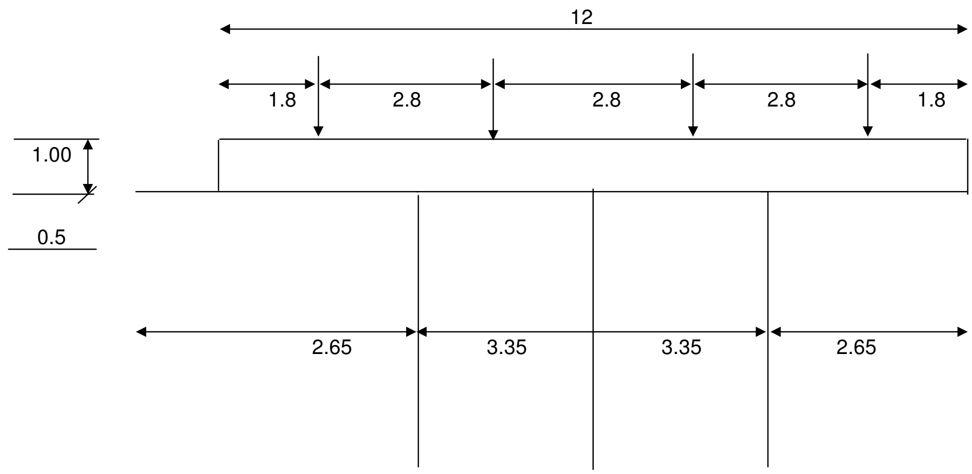
Muliplication factor as per IRC: 21 clause no. 304.7.1.3.3. = 1.14

Permissible shear stress 28.39 t/m^2

Design shear force (22.37 - 28.39) x (4.25 x 0.70) = 17.92 t

Area of reinforcement required for shear $\frac{17.92 \times 10000.00}{20400.00 \times (4.25 - 0.15)}$ = 2.14 cm^2/m

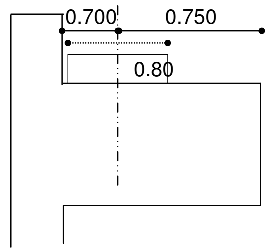
DESIGN OF ABUTMENT CAP



Reaction due to (DL+SIDL) = 441 t
Reaction due to live load = 83 t
Assuming impact factor as 0.12
Additional load due to impact 10.0
Total reaction = 440.5395059 + 83.25 + 10.0 = 533.8 t
Transverse moment due to live load = 242.3 t-m
Total no. of bearings = 4
c/c distance between bearings = 2.8 m
Section modulus = $1.4^2 \times 2 + 4.2^2 \times 2$ = 39.2 m^2
Load due to transverse moment in outer bearing = $\frac{242.3 \times 4.2}{39.2}$ = 25.96 t
Load due to transverse moment in inner bearing = $\frac{242.3 \times 1.4}{39.2}$ = 8.652 t

	MAX LOAD	MIN LOAD
Total load on outer bearing	$= \frac{533.8}{4} + 25.96 =$ 159 t &	$\frac{533.8}{4} - 25.96 =$ 107 t
Total load on third outer bearing	$= \frac{533.8}{4} + 8.652 =$ 142 t &	$\frac{533.8}{4} - 8.652 =$ 125 t

Size of pedestal = 0.8 x 0.8 mm
Width of cap = 1.75 m
Thickness of cap = 1.00 m
Thickness of dirt wall = 0.30 m
Distance between face of dirt wall & c.l. bearing = 0.700 m
Distance between edge of cap & c.l. bearing
= 1.75 - 0.700 - 0.30 = 0.750 m



From STAAD analysis

Maximum negative B.M.	=	135.00	t-m	(Refer STAAD output)
Maximum positive B.M.	=	34.90	t-m	"
Maximum S.F. at d from face	=	159.00	t	"

Effective depth required = $\sqrt{\dfrac{135.00}{192.3 \times 1.75}}$ = 0.633 m

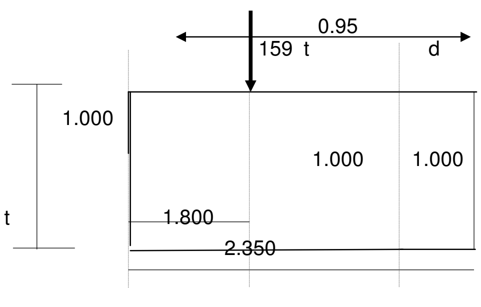
Effective depth provided = 1.00 - 0.05 = 0.95 m

Area of steel (at top) = $\dfrac{135.0 \times 1E+04}{0.877 \times 0.95 \times 20400}$ = 79.41 cm^2 98.17

Area of steel (at bottom) = $\dfrac{34.9 \times 1E+04}{0.877 \times 0.95 \times 20400}$ = 20.53 cm^2

CHECK FOR SHEAR

Checking the section at a distence `d' from support towards inner side



Shear stress = $\dfrac{159}{1.75 \times 0.95}$ = 95.64 t/m^2

$\dfrac{Ast}{b \times d} \times 100$ = $\dfrac{79.41 \times 1E-04}{1.75 \times 0.95} \times 100$ = 0.478 %

Permissible shear stress = 0.30 MPa = 30.89 t/m^2

Design shear = 95.63909774 - 30.89 x 1.75 x 0.95 = 107.6 t

$\dfrac{Asv}{sv}$ = $\dfrac{107.6 \times 1E+04}{20400 \times 0.95}$ = 55.54 cm^2/m

Provide 6 Legged 12 ϕ `@ 122.2 c/c

say 120 c/c

Checking the section under the load

Shear stress = $\dfrac{74.3}{1.75 \times 0.950}$ = 44.69 t/m^2

$\dfrac{Ast}{b \times d} \times 100$ = $\dfrac{79.41 \times 1E-04}{1.75 \times 0.95} \times 100$ = 0.478 %

Permissible shear stress = 0.30 MPa = 30.89 t/m^2

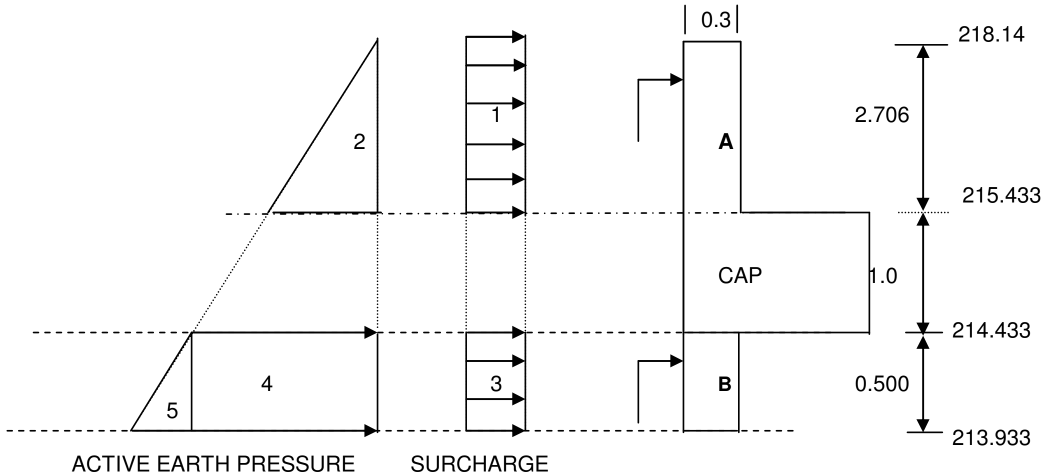
Design shear = 44.7 - 30.89 x 1.75 x 0.95 = 22.94 t

$\dfrac{Asv}{sv}$ = $\dfrac{22.94 \times 1E+04}{20400 \times 0.95}$ = 11.84 cm^2/m

Provide 6 Legged 10 ϕ `@ 398 c/c

say 200 c/c

DESIGN OF DIRT WALL



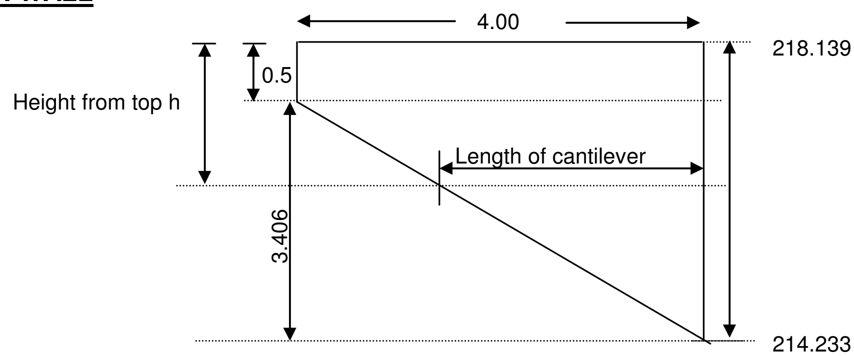
	ELEMENT NO.	AREA FACTOR	HEIGHT	PRESSURE	WIDTH	FORCE	L.A.	MOMENT
L.L. SURCHARGE	1	1.0	2.71	0.67	12.00	21.77	1.353	29.46
ACTIVE EARTH PRE	2	0.5	2.71	1.51	12.00	24.55	1.137	27.90
TOTAL						**46.32**		**57.36**
L.L. SURCHARGE	3	1.0	0.50	0.67	12.00	4.02	0.250	1.01
ACTIVE EARTH PRE	4	1.0	0.50	1.51	12.00	9.07	0.250	2.27
ACTIVE EARTH PRE	5	0.5	0.50	0.28	12.00	0.84	0.333	0.28
TOTAL						13.93		3.55

	For dirt wall A		For dirt wall B	
BENDING MOMENT AT THE EDGE OF CAP	57.36		3.6	
EFFECTIVE DEPTH REQUIRED	0.16		0.04	
EFFECTIVE DEPTH PROVIDED	0.25		0.25	
AREA OF STEEL REQUIRED PER METER WIDTH	10.68	cm^2/m	0.66	cm^2/m

CHECK FOR SHEAR

SHEAR FORCE AT THE EDGE OF CAP	46.32		13.9	
SHEAR STRESS	15.44	t/m^2	4.6	t/m^2
% OF STEEL	0.43		0.03	
PERMISSIBLE SHEAR STRESS	29.2	t/m^2	20.4	t/m^2

DESIGN OF RETURN WALL



Horizontal coefficient of Active earth pressure = **$K_a COS\phi$ =** **K_{ha} =** **0.2794**

CALCULATION OF BENDING MOMENT

Height from top h (m)	0.500	1.000	2.250	3.000
Surcharge pressure.	0.67	0.67	0.67	0.67
Active Earth pressure ($K_a \gamma h$)	0.28	0.56	1.26	1.68
Total	0.95	1.23	1.93	2.35
Length of cantilever (m)	4.000	3.413	1.945	1.064
Cantilever Moment	7.60	7.16	3.65	1.33
Required eff. Depth (m)	0.199	0.193	0.138	0.083
Eff.depth at the section (m)	0.350	0.350	0.350	0.350
Ast.reqd. (mm^2)	12.13	11.43	5.82	2.12
Ast. provided (mm^2)	17.95	17.95	15.71	15.71
Dia of bar (mm)	20	20	20	20
Specing of bar (mm)	175	175	200	200

Check for Shear

shear stress at the section	10.86	11.99	10.71	7.13
% of steel	0.35	0.33	0.17	0.06
Permissible shear stress	26.60	28.59	26.26	24.73
Hence	O.K.	O.K.	O.K.	O.K.

```
   Abutment SHAFT .case 1

   Depth of Section             =     4.250 m
   Width of Section             =      .700 m

along width-compression face- no of bar:   6   tension face- no of bar:  18
Dia (mm)                                  16                             25
Cover (cm)                               7.50                            7.5
along depth-compression face- no of bar:  16   tension face- no of bar:  16
Dia (mm)                                  12                             12
Cover  (cm)                              7.50                            7.5

   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                   =   289.910 T
   Mxx                          =   338.500 Tm
   Myy                          =    17.050 Tm


   Intercept of Neutral axis : X axis : =   1.812 m
                             : y axis : =   3.805 m

             Concrete Stress Governs Design



   Stress in Concrete due to Loads =   33.15 Kg/cm^2
   Stress in Steel    due to Loads =  146.55 Kg/cm^2
   Percentage of Steel             =     .46 %
```

```
INPUT FILE: spill rea.STD
     1. STAAD PLANE
     2. START JOB INFORMATION
     3. ENGINEER DATE 02-NOV-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0 0 0; 2 1.8 0 0; 3 2.65 0 0; 4 4.6 0 0; 5 6 0 0; 6 7.4 0 0;7 9.35 0 0
     9. 8 10.2 0 0; 9 12 0 0; 10 2.65 -5.064 0; 11 6 -5.064 0; 12 9.35 -5.064 0
    10. 13 1.65 -5.064 0;14 10.95 -5.064 0
    11. MEMBER INCIDENCES
    12. 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 3 10; 10 5 11
    13. 11 7 12;12 13 10;13 10 11;14 11 12;15 12 14
    14. DEFINE MATERIAL START
    15. ISOTROPIC MATERIAL1
    16. E 2.95E+006
    17. POISSON 0.15
    18. DENSITY 2.4
    19. END DEFINE MATERIAL
    20. MEMBER PROPERTY INDIAN
    21. 1 TO 8 PRIS YD 1 ZD 1.75
    22. 9 TO 11 PRIS YD 0.7  ZD 3
    23. 12 TO 15 PRIS YD 2 ZD 12.3
    24. CONSTANTS
    25. MATERIAL MATERIAL1 MEMB 1 TO 15
    26. SUPPORTS
    27. 10 TO 12 FIXED BUT FX MZ
    28. LOAD 1 SELFWEIGHT
    29. SELFWEIGHT Y -1
    30. LOAD 2
    31. JOINT LOAD
    32. 2 8 FY -159
    33. 4 6 FY -142
    34. LOAD 3
    35. MEMBER LOAD
    36. 1 TO 8 UMOM GX 5.875
    37. PERFORM ANALYSIS
    38. PRINT MEMBER FORCES
        MEMBER   FORCES

   MEMBER END FORCES    STRUCTURE TYPE = PLANE
   -----------------
   ALL UNITS ARE -- MTON METE
```

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
1	1	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	7.56	0.00	0.00	0.00	-6.80
	2	1	0.00	0.00	0.00	0.00	0.00	0.00
		2	0.00	0.00	0.00	0.00	0.00	0.00
	3	1	0.00	0.00	0.00	-5.29	0.00	0.00
		2	0.00	0.00	0.00	-5.29	0.00	0.00
2	1	2	0.00	-7.56	0.00	0.00	0.00	6.80
		3	0.00	11.13	0.00	0.00	0.00	-14.75

	2	2	0.00	-159.00	0.00	0.00	0.00	0.00
		3	0.00	159.00	0.00	0.00	0.00	-135.15
	3	2	0.00	0.00	0.00	-2.50	0.00	0.00
		3	0.00	0.00	0.00	-2.50	0.00	0.00
3	1	3	-1.08	9.66	0.00	0.00	0.00	10.94
		4	1.08	-1.47	0.00	0.00	0.00	-0.08
	2	3	-8.79	75.21	0.00	0.00	0.00	104.90
		4	8.79	-75.21	0.00	0.00	0.00	41.76
	3	3	0.00	0.00	0.00	-5.73	0.00	0.00
		4	0.00	0.00	0.00	-5.73	0.00	0.00
4	1	4	-1.08	1.47	0.00	0.00	0.00	0.08
		5	1.08	4.41	0.00	0.00	0.00	-2.14
	2	4	-8.79	-66.79	0.00	0.00	0.00	-41.76
		5	8.79	66.79	0.00	0.00	0.00	-51.75
	3	4	0.00	0.00	0.00	-4.11	0.00	0.00
		5	0.00	0.00	0.00	-4.11	0.00	0.00
5	1	5	-1.13	4.38	0.00	0.00	0.00	2.03
		6	1.13	1.50	0.00	0.00	0.00	-0.02
	2	5	-8.79	66.79	0.00	0.00	0.00	51.75
		6	8.79	-66.79	0.00	0.00	0.00	41.76
	3	5	0.00	0.00	0.00	-4.11	0.00	0.00
		6	0.00	0.00	0.00	-4.11	0.00	0.00
6	1	6	-1.13	-1.50	0.00	0.00	0.00	0.02
		7	1.13	9.69	0.00	0.00	0.00	-10.94
	2	6	-8.79	-75.21	0.00	0.00	0.00	-41.76
		7	8.79	75.21	0.00	0.00	0.00	-104.90
	3	6	0.00	0.00	0.00	-5.73	0.00	0.00
		7	0.00	0.00	0.00	-5.73	0.00	0.00
7	1	7	0.00	11.13	0.00	0.00	0.00	14.75
		8	0.00	-7.56	0.00	0.00	0.00	-6.80
	2	7	0.00	159.00	0.00	0.00	0.00	135.15
		8	0.00	-159.00	0.00	0.00	0.00	0.00
	3	7	0.00	0.00	0.00	-2.50	0.00	0.00
		8	0.00	0.00	0.00	-2.50	0.00	0.00
8	1	8	0.00	7.56	0.00	0.00	0.00	6.80
		9	0.00	0.00	0.00	0.00	0.00	0.00
	2	8	0.00	0.00	0.00	0.00	0.00	0.00
		9	0.00	0.00	0.00	0.00	0.00	0.00
	3	8	0.00	0.00	0.00	-5.29	0.00	0.00
		9	0.00	0.00	0.00	-5.29	0.00	0.00
9	1	3	20.79	1.08	0.00	0.00	0.00	3.81
		10	-46.31	-1.08	0.00	0.00	0.00	1.64
	2	3	234.21	8.79	0.00	0.00	0.00	30.25
		10	-234.21	-8.79	0.00	0.00	0.00	14.25
	3	3	0.00	0.00	0.00	0.00	0.00	0.00
		10	0.00	0.00	0.00	0.00	0.00	0.00
10	1	5	8.78	0.05	0.00	0.00	0.00	0.11
		11	-34.31	-0.05	0.00	0.00	0.00	0.15
	2	5	133.59	0.00	0.00	0.00	0.00	0.00
		11	-133.59	0.00	0.00	0.00	0.00	0.00
	3	5	0.00	0.00	0.00	0.00	0.00	0.00
		11	0.00	0.00	0.00	0.00	0.00	0.00

11	1	7	20.82	-1.13	0.00	0.00	0.00	-3.81
		12	-46.35	1.13	0.00	0.00	0.00	-1.91
	2	7	234.21	-8.79	0.00	0.00	0.00	-30.25
		12	-234.21	8.79	0.00	0.00	0.00	-14.25
	3	7	0.00	0.00	0.00	0.00	0.00	0.00
		12	0.00	0.00	0.00	0.00	0.00	0.00
12	1	13	0.00	0.00	0.00	0.00	0.00	0.00
		10	0.00	59.04	0.00	0.00	0.00	-29.52
	2	13	0.00	0.00	0.00	0.00	0.00	0.00
		10	0.00	0.00	0.00	0.00	0.00	0.00
	3	13	0.00	0.00	0.00	0.00	0.00	0.00
		10	0.00	0.00	0.00	0.00	0.00	0.00
13	1	10	1.08	90.43	0.00	0.00	0.00	27.88
		11	-1.08	107.35	0.00	0.00	0.00	-56.22
	2	10	8.79	-5.14	0.00	0.00	0.00	-14.25
		11	-8.79	5.14	0.00	0.00	0.00	-2.97
	3	10	0.00	0.00	0.00	0.00	0.00	0.00
		11	0.00	0.00	0.00	0.00	0.00	0.00
14	1	11	1.13	93.64	0.00	0.00	0.00	56.07
		12	-1.13	104.15	0.00	0.00	0.00	-73.67
	2	11	8.79	5.14	0.00	0.00	0.00	2.97
		12	-8.79	-5.14	0.00	0.00	0.00	14.25
	3	11	0.00	0.00	0.00	0.00	0.00	0.00
		12	0.00	0.00	0.00	0.00	0.00	0.00
15	1	12	0.00	94.46	0.00	0.00	0.00	75.57
		14	0.00	0.00	0.00	0.00	0.00	0.00
	2	12	0.00	0.00	0.00	0.00	0.00	0.00
		14	0.00	0.00	0.00	0.00	0.00	0.00
	3	12	0.00	0.00	0.00	0.00	0.00	0.00
		14	0.00	0.00	0.00	0.00	0.00	0.00

39. PRINT SUPPORT REACTIONS

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
10	1	0.00	195.79	0.00	0.00	0.00	0.00
	2	0.00	229.07	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
11	1	0.00	235.30	0.00	0.00	0.00	0.00
	2	0.00	143.87	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00
12	1	0.00	244.96	0.00	0.00	0.00	0.00
	2	0.00	229.06	0.00	0.00	0.00	0.00
	3	0.00	0.00	0.00	0.00	0.00	0.00

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

40. FINISH

```
INPUT FILE: CLA.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. JOINT COORDINATES
     6. 1          .00          .000          .000
     7. 2          0.4        .000          .000
     8. 3        30.4         .000          .000
     9. 4        30.8          .000          .000
    11. MEMBER INCIDENCES
    12. 1         1         2
    13. 2         2         3
    14. 3         3         4
    16. MEMBER PROPERTY CANADIAN
    17. 1 TO 3 PRI YD 2. ZD 1.
    18. CONSTANT
    19. E CONCRETE ALL
    20. DENSITY CONCRETE ALL
    21. POISSON CONCRETE ALL
    22. SUPPORT
    23. 2 3   PINNED
    24. DEFINE MOVING LOAD
    25. TYPE 2 LOAD 2.7 2.7  2*11.4 4*6.8 DIS 3*3 4.3 1.2 3.2 1.1
    26. LOAD GENERATION 180
    27. TYPE 2 -57.6 0. 0. XINC .5
    28. TYPE 2 -18.8 0. 0. XINC .5
    29. PERFORM ANALYSIS
    30. LOAD LIST 140
    31. PRINT SUPPORT REACTION
SUPPORT  REACTION

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

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

      2  140      0.00     13.62      0.00      0.00      0.00      0.00
      3  140      0.00     41.78      0.00      0.00      0.00      0.00

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

    32. FINISH


             *********** END OF THE STAAD.Pro RUN ***********
```

ADDITIONAL SHEETS (SEISMIC CHECK)

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 = $2.00\sqrt{\dfrac{D}{1000xF}}$

Where

D is appropriate dead load of superstructure and Live load

DL		341.94	t
SIDL		84.60	t
LL		83.56	t
DL+SIDL+50% LL	=	4683.20	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	29580.40	GPa	
		3017200.7	t/m^2	
	I	2.32	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	=	6.49	m
Distance of CG of super structure from deck slab top			1.40	m
Distance of CG of super structure from free end of cantilever				
	x	=	1.01	m
	F	=	997.07	kN
Corresponding time period	T	=	0.1371	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

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

Where

D is appropriate dead load of superstructure and Live load

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

DL+SIDL = 4265.40 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	29580.40	GPa	
		3017201	t/m^2	
	I	4.73	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	=	6.49	m
Top of abutment from free end of cantilever	x	=	1.01	m
	F	=	2034.84	kn
Corresponding time period	T	=	0.09	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	2.706	2.4	1.753	9.642	16.907
2	Abut cap	1	12.0	1.75	1	2.4	3.780	7.789	29.442
3	Abut shaft	1	2.1	3	3.6445	2.4	4.133	4.875	20.146
							9.666		**66.495**

Lever arm for the loads from super structure

Formation level	218.139	m
Wearing coat	56	mm
Top of deck slab	218.083	m
Distance of CG of super structure below deck slab top	1.400	m
Pile cap bottom	207.144	m
Lever arm for DL	9.539	m

Distance of CG of SIDL above deck slab top	0.261	m
Level arm for SIDL	11.200	m
Distance of CG of live load above formation level	1.2	
Lever arm for live load	12.195	

Load from Superstruture

Longitudinal direction

	Weight	Force	L.A	moment
DL	341.94	25.65	9.539	244.64
SIDL	84.60	6.35	11.200	71.06
	Total	**31.99**		**315.70**

Transverse direction

	Weight	Force	L.A	moment
DL	341.94	25.645	9.54	244.64
SIDL	84.60	6.345	11.20	71.06
LL 50%	41.78	3.134	12.20	38.21
Total		**35.124**		**353.92**

Summary

seismic longitudinal case

	P	M_L	M_T
Total load on piles for normal case	2465.58	-1019.01	242.26
Additional horizantal force and moment due to seismic	-	-382.197	-
(Live load not to be considered for longitudinal moment)			
Reduction in longitudinal moment due to braking force	-	292.13	-
Reduction in transverse moment due to live load			-242.26
	2465.58	**-1109.070**	**0.00**

Seismic Transverse case

		P	M_L	M_T
Total load on piles for normal case		2465.58	-1019.01	242.26
Additional horizantal force and moment due to seismic		-	-	420.41
(Only 50%% live load is to be considered for transverse moment)				
Reduction in longitudinal moment due to braking force		-	146.066 -	
Reduction in transverse moment due to live load	-	-		-121.13
		2465.58	**-872.94**	**541.54**

SEISMIC ANALYSIS

Total no. of Piles (**N**) 12

P	2465.58	Z_L (outer)	36.00	Z_L (inner)	108.00
M_L	-1109.070				
M_T	0.000	Z_T (outer)	28.80	Z_T (inner)	0.00

Seismic Longitudinal					Seismic Transverse				
P/N	M_L/Z_L (outer)	M_L/Z_L (inner)	M_T/Z_T (outer)	M_T/Z_T (inner)	P/N	M_L/Z_L (outer)	M_L/Z_L (inner)	M_T/Z_T (outer)	M_T/Z_T (inner)
205.46	-30.81	-10.27	0.00	0.00	205.46	-24.25	-8.08	18.80	0

TRAFFIC DIRECTION

PILE NO.	A	B	C	D	A	B	C	D
1	**236.27**	215.73	195.20	174.66	**210.91**	194.74	178.58	162.41
2	236.27	215.73	195.20	174.66	229.71	213.55	197.38	181.22
3	236.27	215.73	195.20	174.66	248.52	232.35	216.19	200.02

Permissible capacity of pile in normal case = 275 ton

Permissible capacity of pile in seismic case = 275 x 1.25 = 343.75 ton > 248 ton

Horizontal load per pile for Normal case

Horizantal force due to weight of earth	=	0.00	t	
Horizontal force due to braking	=	32.86	t	
Horizantal seismic force due to abutment avove G.L	=	0.00	t	
	=	**32.86**	t	
Number of Piles	=	12.00		
	=	2.74	t	per pile
Horizantal force in Normal condition due to earth	=	180.91	t	
	=	15.08	t	per pile
Therefore Total Horizantal force per pile	=	**17.81**	t	per pile
Maximum Load on the Pile	=	**242.18**		
Minimum Load on the Pile	=	**168.75**		

Determination of Lateral 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 m

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

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

K_1 = 0.775

K_2 = 0

T = 330.78

R = 0

L_e = Embeded Length of pile

= 207.19 - 199.5 = 769.4 cm

L_1 = Exposed Length = 207.09 - 206.99 = 10.4 cm

$\dfrac{L_1}{T}$ = 0.0314

$\dfrac{L_f}{T}$ = 2.15

L_f = 711.19 cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12\ E\ I} = \frac{17.815 \times 711.19}{12 \times 301533 \times 10178760}$$

$$= 0.174 \text{ cm}$$

$$= 1.74 \text{ mm}$$

In seismic case as permissble increase in deflection is 25%, hence

Permissible deflection = 5 x 1.25 = 6.25 mm > 1.7399 mm

Maximum Moment

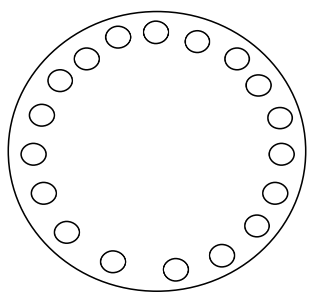
$$M_F = Q \times \frac{\left(L_1 + L_f\right)}{2} = 63.348 \text{ t-m}$$

$$M = m \times M_F$$

$$= 0.83 \times 63.348 = 52.58 \text{ t-m}$$

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm^2



Provide 10 mm Rings at 190 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)	49.14	74.21
AS=AREA OF STEEL (in sq. Cm)	60.319	60.319
P=VERTICAL LOAD (in tonnes)	242.18	168.75
BM=BENDING MOMENT (in t-m)	52.58	52.58
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	0.002	0.002
B=BD*PI/180	0.858	1.295
B1=COS(B)	0.654	0.272
A=ALPHA ANGLE=ACOS (RO*B1/RI)	0.726	1.255
RECOSB=RICOSA		
B2=SIN(B)^3	0.433	0.891
B3=SIN(4*B)	-0.285	-0.892
B4=SIN(2*B)	0.990	0.524
A=COS(A)	0.748	0.311
A2=SIN(2*A)	0.993	0.591
A3=SIN(A)	0.664	0.950
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.371	0.284
NUM2=2*(RO^3)/(1+B1)	0.261	0.340
NUM3=(RI^3) *T/(RO+RI*A1)	0.000	0.000
NUM4=(M-1) *PI+A-A2/2	28.50	29.23
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.104	0.106
DENM1=2*(RO^2)/(1+B1)	0.435	0.566
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	1.053	0.584
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.001	0.001
DENM4=(M-1) *PI*A1-A3+A*A1	21.02	8.23
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.480	0.341
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.218**	**0.312**
ACTUAL ECCENTRICITY EC = M/P	**0.217**	**0.312**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	0.393	0.163
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.993	0.763
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**505**	**494**
TS=TENS.STRESS IN STEEL (T/SQM)	**674**	**2343**
PERMISSIBLE COMP.STRESS IN Concrete 50% increase (in t/sq.m)	1190	1190
PERMISSIBLE TENSILE STRESS IN Steel 50% increase(in t/sq.m)	20400	20400

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

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

Horizontal seismic force due superstructure (DL+SIDL)	=	31.99	t	
(Live load not to be considered for longitudinal direction)				
Horizantal seismic force due to abutment avove G.L	=	9.67	t	
	=	**41.66**	t	
Number of Piles	=	12.00		
	=	3.47	t	per pile
Horizantal force in Normal condition due to earth	=	180.91	t	
	=	15.08	t	per pile
Therefore Total Horizantal force per pile	=	**18.55**	t	per pile
Maximum Load on the Pile	=	**236.27**		
Minimum Load on the Pile	=	**174.66**		

Determination of Lateral 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

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

K_1 = 0.775

K_2 = 0

T = 330.78

R = 0

L_e = Embeded Length of pile

= 207.19 - 199.5 = 769.4 cm

L_1 = Exposed Length = 207.09 - 206.99 = 10.4 cm

$\frac{L_1}{T}$ = 0.0314

$\frac{L_f}{T}$ = 2.15

L_f = 711.19 cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12\ E\ I} = \frac{18.547 \times 711.19}{12 \times 301533 \times 10178760}$$

= 0.181 cm

= 1.81 mm

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

Permissible deflection = 5 x 1.33 = 6.65 mm > 1.8114 mm

Maximum Moment

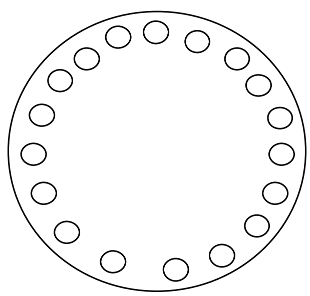
$$M_F = Q \times \frac{\left(L_1 + L_f\right)}{2} = 65.953 \text{ t-m}$$

$$M = m \times M_F$$

$$= 0.83 \times 65.953 = 54.74 \text{ t-m}$$

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm^2



Provide 10 mm Rings at 190 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)	54.11	74.43
AS=AREA OF STEEL (in sq. Cm)	60.319	60.319
P=VERTICAL LOAD (in tonnes)	236.27	174.66
BM=BENDING MOMENT (in t-m)	54.74	54.74
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	0.002	0.002
B=BD*PI/180	0.944	1.299
B1=COS(B)	0.586	0.268
A=ALPHA ANGLE=ACOS (RO*B1/RI)	0.837	1.259
RECOSB=RICOSA		
B2=SIN(B)^3	0.532	0.894
B3=SIN(4*B)	-0.594	-0.885
B4=SIN(2*B)	0.950	0.517
A=COS(A)	0.670	0.307
A2=SIN(2*A)	0.995	0.584
A3=SIN(A)	0.742	0.952
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.360	0.283
NUM2=2*(RO^3)/(1+B1)	0.272	0.341
NUM3=(RI^3) *T/(RO+RI*A1)	0.000	0.000
NUM4=(M-1) *PI+A-A2/2	28.61	29.24
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.106	0.106
DENM1=2*(RO^2)/(1+B1)	0.454	0.568
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	0.961	0.580
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.001	0.001
DENM4=(M-1) *PI*A1-A3+A*A1	18.76	8.11
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.456	0.340
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.233**	**0.313**
ACTUAL ECCENTRICITY EC = M/P	**0.232**	**0.313**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	0.352	0.161
NAD=DEPTH OF N.A FROM TOP=RO+NAC	0.952	0.761
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**518**	**514**
TS=TENS.STRESS IN STEEL (T/SQM)	**943**	**2456**
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.

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

Total horizantal force due to deck movement	=	0.000	t	
Horizontal force due to earth	=	0.000	t	
Horizantal seismic force due superstructure (DL+SIDL)	=	31.990	t	
Horizantal seismic force due superstructure (LL)	=	3.134	t	
Horizantal force due to abutment avove G.L	=	9.666	t	
Horizantal force due to braking	=	16.432		
	=	**44.79**	t	
Number of Piles	=	12.00		
	=	3.733	t	per pile
Horizantal force in Normal condition due to earth	=	180.91	t	
	=	16.445	t	per pile
Therefore Total Horizantal force per pile	=	**16.86**	t	per pile
Maximum Load on the Pile	=	**248.52**	t	
Minimum Load on the Pile	=	**162.41**	t	

Determination of Lateral 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

$\frac{L_f}{T}$ = 2.15

L_f = 711.19 cm

Pile Head deflection

$$y = \frac{Q\left(L_1 + L_f\right)^3}{12\ E\ I} = \frac{16.864 \ \text{x}\ 711.19}{12\ \text{x}\ 301533\ \text{x}\ 10178760}$$

= 0.165 cm

= 1.65 mm

in seismic transverse case permissible increase in deflection = 1.25 x 5 = 6.65 mm > 1.65 mn

Maximum Moment

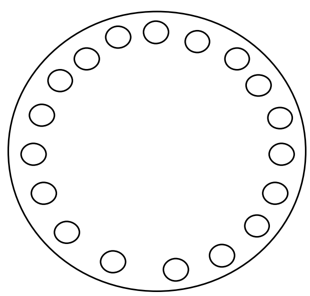
$$M_F = Q \ \text{x} \ \frac{\left(L_1 + L_f\right)}{2} = 59.966 \ \text{t-m}$$

$$M = m \ \text{x} \ M_F$$

$$= 0.83 \ \text{x} \ 59.966 = 49.77 \ \text{t-m}$$

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm^2



Provide 10 mm Rings at 190 mm c/c

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)	42.28	73.10
AS=AREA OF STEEL (in sq. Cm)	60.319	60.319
P=VERTICAL LOAD (in tonnes)	248.52	162.41
BM=BENDING MOMENT (in t-m)	49.77	49.77
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	0.002	0.002
B=BD*PI/180	0.738	1.276
B1=COS(B)	0.740	0.291
A=ALPHA ANGLE=ACOS (RO*B1/RI)	0.563	1.232
RECOSB=RICOSA		
B2=SIN(B)^3	0.305	0.876
B3=SIN(4*B)	0.189	-0.925
B4=SIN(2*B)	0.996	0.556
A=COS(A)	0.846	0.332
A2=SIN(2*A)	0.903	0.627
A3=SIN(A)	0.534	0.943
NUM1=(PI-B)/8+B3/32+B1*B2/3	0.381	0.289
NUM2=2*(RO^3)/(1+B1)	0.248	0.335
NUM3=(RI^3) *T/(RO+RI*A1)	0.000	0.000
NUM4=(M-1) *PI+A-A2/2	28.39	29.19
NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4	0.102	0.107
DENM1=2*(RO^2)/(1+B1)	0.414	0.558
DENM2=B2/3+(PI-B)*B1/2+B1*B4/4	1.175	0.604
DENM3=2*(RI^2) * T/ (RO+RI*A1)	0.001	0.001
DENM4=(M-1) *PI*A1-A3+A*A1	23.85	8.86
DENM=DENOMINATOR=DENM1*DENM2+DENM3*DENM4	0.509	0.348
CHECK FOR ECCENTRICITIES		
CALCULATED ECCENTRICITY EC = NUM/DENM	**0.200**	**0.307**
ACTUAL ECCENTRICITY EC = M/P	**0.200**	**0.306**
CHECK FOR STRESSES		
NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B)	0.444	0.174
NAD=DEPTH OF N.A FROM TOP=RO+NAC	1.044	0.774
DE=EFFECTIVE DEPTH=D-C	1.125	1.125
CC=COMP. STRESS IN CONC. IN T/SQM=P/DENM	**488**	**466**
TS=TENS.STRESS IN STEEL (T/SQM)	**379**	**2111**
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 SUPERSTRUCTURE

Bridge Data

Dimensional Data

1	Total span c/c of Expansion Joint		32.2	m
2	Length of gap slab at both ends		0.7	m
3	Distance of centre of bearing from end of girder		0.4	m
4	span c/c of bearing I.e. effective span		30	m
5	skew angle of the bridge		0	
6	width of clear carriageway		7.5	m
7	Extra widening in carriageway, if any		0	
8	Width of crash barrier footpath side		0.5	m
9	Width of foothpath		1.525	m
10	Width of railing curb		0.225	m
11	Width of median side curb		0.5	m
12	Half width of median including curb		2.25	m
13	Total width of bridge		12	m
14	Spacing of main Girders c/c		2.8	m
15	Number of main girders		4	
16	Spacing of cross girders c/c		7.500	m
17	Thickness of deck slab		0.25	m
18	Thicknes of wearing coat		0.056	m
19	Thickness of cantilever slab at fixed end		0.25	m
20	Thickness of cantilever slab at free end		0.25	m
21	Depth of precast girder including flange		2.1	m
22	Web thickness of precast girder at mid section		0.275	m
23	web thickness of precast girder at support section		0.6	m
24	Length of side flaring section in main girder		2.5	m
25	Length of widened section in main girder from end		2.9	m
26	Distance of end of main girder from centre of bearing		0.4	m
27	Length of widened section in main girder from bearing		2.5	m
28	Width of flange of precast girder at top		1.25	m
29	Width of flange of precast girder at bottom		1	m
30	Thickness of flange of precast girder at top		0.15	m
31	Size of top haunch	width	0.4875	m
		Depth	0.16	m
32	Size of bottom haunch	Width	0.3625	m
		Depth	0.24	m
	Size of bottom bulb	Width	1	m
		Depth	0.3	m
33	Depth of Intermediate cross girder excluding deck slab		1.8	m
34	Depth of end cross girder excuding deck slab		1.8	m
35	Web thickness of intermediate cross girder		0.3	m
36	Web thickness of end cross girder		0.45	m

Properties

Grade of concrete for superstructure	M	40	N/mm2
Grade of reinforcement for superstructure	FY	415	N/mm2
Clear cover to reinforcement for slabs		0.04	m
Clear cover to reinforcement for beams		0.05	m
Unit weight of RCC (as per IRC:6-2000,Cl 205)		2.4	t/m3
unit weight of prestressed concrete(as per IRC:6-2000,Cl 205)		2.5	t/m3
Unit weight of wearing coat		2.2	t/m3
Weight of crash barrier		1	t/m
Weight of railing,if any	no		
Weight of utilities under foothpath slab		0.1	t/m
Allowable stress in concrete in bending compression		13.33	N/mm2
allowable stress in steel in bending compression		200	N/mm2
Modular ratio		10	
Poissions ratio for concrete		0.15	
Modulus of elasticity		32500	N/mm2
Coefficient of thermal expansion		1.17E-05	per deg.C

General Features

Girder Length

Span c/c of exp. Joint	32.200	m
Gap slab	0.700	m
Overall length of girder	30.800	m
Distance of centre of bearing from end of girder	0.400	m
Effective length of girder from c/c of bearing	30.000	m

Sequence of deck construction

1 Cast PSC beams at site
2 Stress Stage -I tendons in the beam
3 Cast the in-situ portion of cross girders and deck slab at top
4 Stress Stage -II tendons in the beam
5 Cast the gap slab,Crash barrier,Kerb etc.
6 Lay the wearing coat

Design Sections

For flexural and shear stress
1 mid span sections
2 3/8 th section
3 1/4 th sections
4 1/8th span section
5 effective depth d

Concrete Strength

For analysis it has been assumed that the precast beams as well as the deck slab of the top shall have a minimum of M-40 grade concrete. However the concrete shall be design mix and have a minimum 28 days characterstics strength of 40Mpa on 150 mm cubes for all elements of super structure.

Codes of References

(I) IRC: 5-1998
(II) IRC:6-2000
(iii) IRC:18-2000
(iv) IRC:21-2000
(v) IRC:22-1986

Reinforcement

Reinforcing steel shall be of HYSD bars (grade designation S:415) confirming to IS:1786

Prestressing steel and Accessories

Cable consisting of 12 No. of 12.7 mm dia 7 ply class 2 strand as per IS:6006-1983 shall be used for main stressing.

Sheathing

Sheathing shall be of "Drassbatch" type 75 mm ID manufactured from minimum o.3 mm thick bright metal strip. It shall be as per IRC;18 -2000, Appendix 1.

SECTION PROPERTIES

GIRDER (0uter)
AT MID SPAN

Self			Composite	
Depth (mm)	**2100**		Depth (mm)	2350
Area (mm^2)	1106250	Deck slab	Area (mm^2)	1906250
I_z (mm^4)	6.2283E+11	**250**	I_z (mm^4)	1.31354E+12
Y_b(mm)	1008.95	**+** mm	Y_b(mm)	1519.3
Y_t(mm)	1091.05	thick and	Y_t(mm)	830.7
Z_b(mm^3)	6.173E+08	**3200**	Z_b(mm^3)	8.646E+08
Z_t(mm^3)	5.709E+08	mm wide.	Z_t(mm^3)	1.581E+09
Concrete M- 40			Concrete M- 40	

I_y

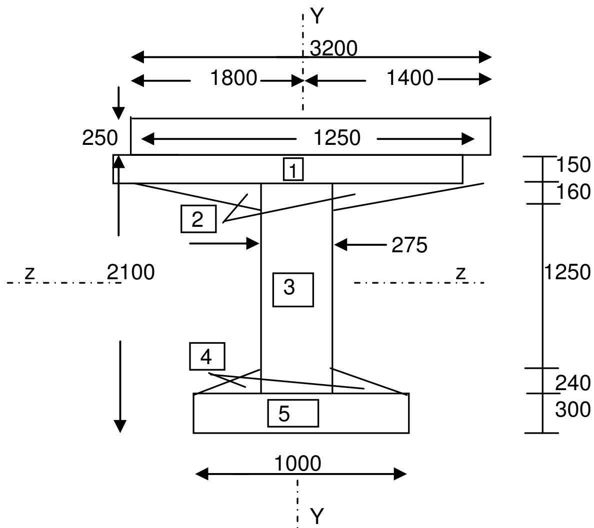
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	1.030E+09	7.019E+09
3	2.860E+09	0
4	6.351E+08	5.805E+09
5	2.500E+10	0
	5.394E+10	1.282E+10

Iy(self) = 6.676E+10 mm^4

zL = 1716.1

Iy(slab) = 6.827E+11

Iy(comp) = 7.68E+11



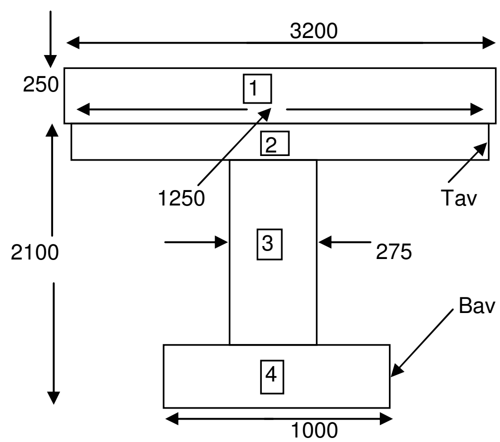
I_x

Tav = 230.0 mm
Bav = 420.0 mm

Portion no	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
1	3200	250	8.33E+09
2	1250	230	4.41E+09
3	275	1450	8.73E+09
4	1000	420	1.89E+10

I_x (mm^4) = 4.037E+10

for deck slab Ix = b x d^3 / 6



AT SUPPORT

Iz

Self			Composite	
Depth (mm)	2100		Depth (mm)	2350
Area (mm^2)	1538649.43	Deck slab	Area (mm^2)	2338649
I_z (mm^4)	6.932E+11	250	I_z (mm^4)	1.42534E+12
Y_b(mm)	1048.92	+ mm	Y_b(mm)	1451.2
Y_t(mm)	1051.08	thick and	Y_t(mm)	898.8
Z_b(mm^3)	6.608E+08	3200	Z_b(mm^3)	9.822E+08
Z_t(mm^3)	6.595E+08	mm	Z_t(mm^3)	1.586E+09
W (kN/m)	38.5	wide.		
Concrete M- 40			Concrete M- 40	

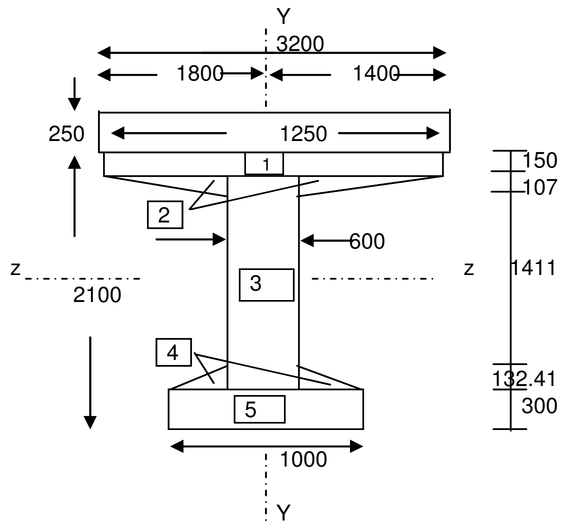
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	2.034E+08	5.780E+09
3	2.970E+10	0
4	5.885E+07	3.560E+09
5	2.500E+10	0
	7.938E+10	9.340E+09

Iy(self) = 8.872E+10 mm^4

zL = 1731.6

Iy(slab) = 6.827E+11

Iy(comp) = 7.9244E+11



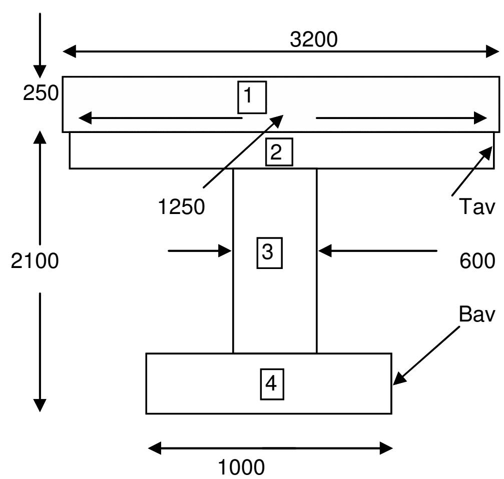
Ix

Tav = 203.3 mm
Bav = 366.2 mm

Portion no	b	d	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
	mm	mm	
1	3200	250	8.33E+09
2	1250	203	3.07E+09
3	600	1530	8.60E+10
4	1000	366.206897	1.30E+10

Ix (mm^4) = 1.10E+11

for deck slab Ix = b x d^3 / 6

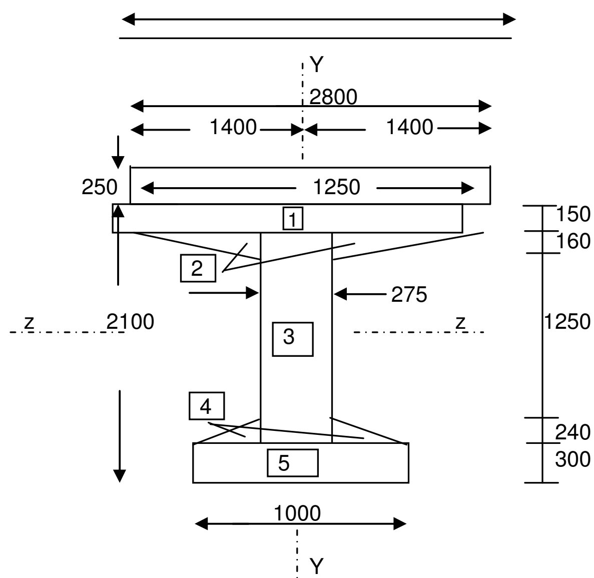


SECTION PROPERTIES

GIRDER (INNER)
AT MID SPAN

Self			Composite	
Depth (mm)	**2100**		Depth (mm)	2350
Area (mm^2)	1106250	Deck slab	Area (mm^2)	1806250
I_z (mm^4)	6.23E+11	**250**	I_z (mm^4)	1.26046E+12
Y_b(mm)	1008.95	**+** mm	Y_b(mm)	1480.2
Y_t(mm)	1091.05	thick and	Y_t(mm)	869.8
Z_b(mm^3)	6.173E+08	**2800**	Z_b(mm^3)	8.515E+08
Z_t(mm^3)	5.709E+08	mm wide.	Z_t(mm^3)	1.449E+09
Concrete M- 40			Concrete M- 40	

I_y



Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	1.030E+09	7.019E+09
3	2.860E+09	0
4	6.351E+08	5.805E+09
5	2.500E+10	0
	5.394E+10	1.282E+10

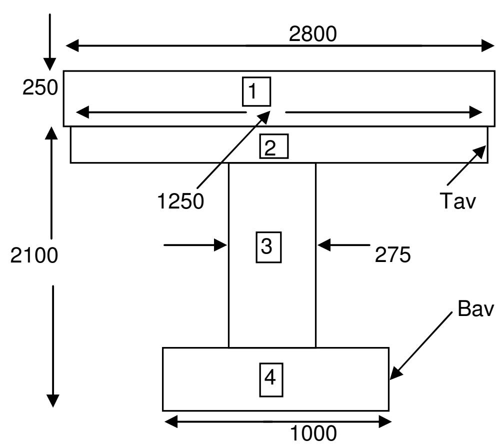
Iy(self) = 6.676E+10 mm^4

zL = 1400.0

Iy(slab) = 4.573E+11

Iy(comp) = 5.241E+11

I_x



Tav = 230.0 mm
Bav = 420.0 mm

Portion no	b	d	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
	mm	mm	
1	2800	250	7.29E+09
2	1250	230	4.41E+09
3	275	1450	8.73E+09
4	1000	420	1.89E+10

I_x (mm^4) = 3.93E+10

for deck slab Ix = b x d^3 / 6

AT SUPPORT

Iz

Self			Composite	
Depth (mm)	2100		Depth (mm)	2350
Area (mm^2)	1538649.43	Deck slab	Area (mm^2)	2238649
I_z (mm^4)	6.93E+11	250	I_z (mm^4)	1.36228E+12
Y_b(mm)	1048.92	+ mm	Y_b(mm)	1416.7
Y_t(mm)	1051.08	thick and	Y_t(mm)	933.3
Z_b(mm^3)	6.608E+08	2800	Z_b(mm^3)	9.616E+08
Z_t(mm^3)	6.595E+08	mm	Z_t(mm^3)	1.460E+09
W (kN/m)	38.5	wide.		
Concrete M- 40			Concrete M- 40	

I_y

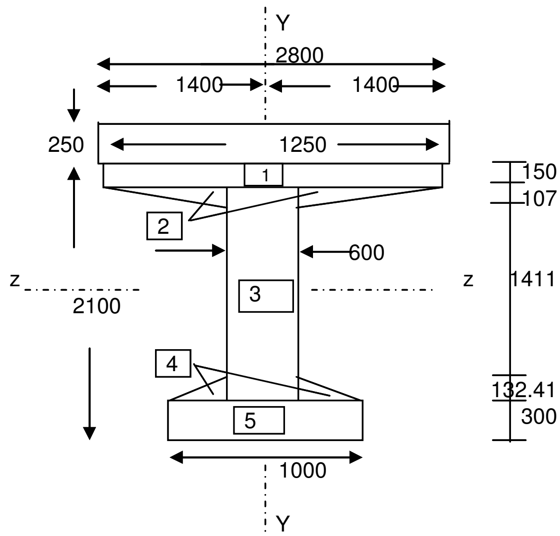
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	2.034E+08	5.780E+09
3	2.970E+10	0
4	5.885E+07	3.560E+09
5	2.500E+10	0
	7.938E+10	9.340E+09

Iy(self) = 8.872E+10 mm^4

zL = 1400.0

Iy(slab) = 4.573E+11

Iy(comp) = 5.4605E+11



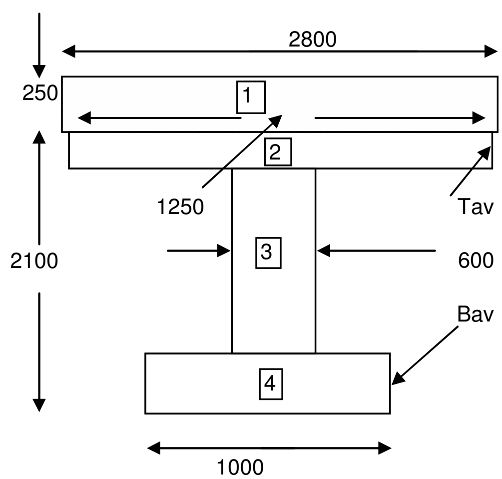
Ix

Tav = 203.3 mm
Bav = 366.2 mm

ortion nc	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
1	2800	250	7.29E+09
2	1250	203	3.07E+09
3	600	1530	8.60E+10
4	1000	366.206897	1.30E+10

Ix (mm^4) = 1.09E+11

for deck slab Ix = b x d^3 / 6



Cross member At End Supp.

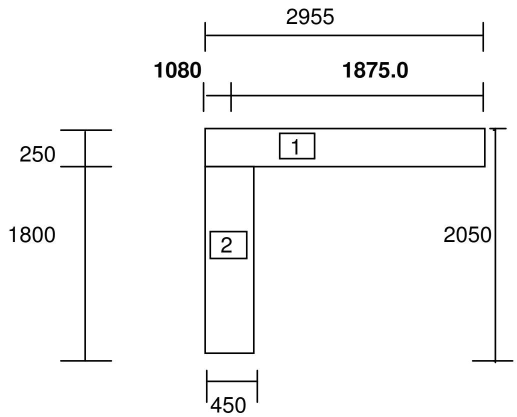
I_z

Area (Ax) = 2955 x 250
+ 1800 x 450

= 1548750 mm^2

yb = 1389 mm

yt = 661 mm



Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	3.848E+09	2.123E+11
2	2.187E+11	1.9363E+11
	2.225E+11	4.0593E+11

Iz = 6.285E+11 mm^4

I_y

y_l = 1270 mm

y_r = 1685 mm

Element	Iself = $bd^3/12$ mm^4	A_y^2 mm^4
1	5.376E+11	3.19E+10
2	1.367E+10	2.91E+10
	5.512E+11	6.10E+10

Iy = 6.123E+11 mm^4

Ix

Element	b	d	$\frac{3.b^3.d^3}{10*(b^2+d^2)}$
	mm	mm	
1	2955	250	7.695E+09
2	450	1800	4.631E+10

Ix = 5.401E+10 mm^4

Cross member At Intermediate Supp.

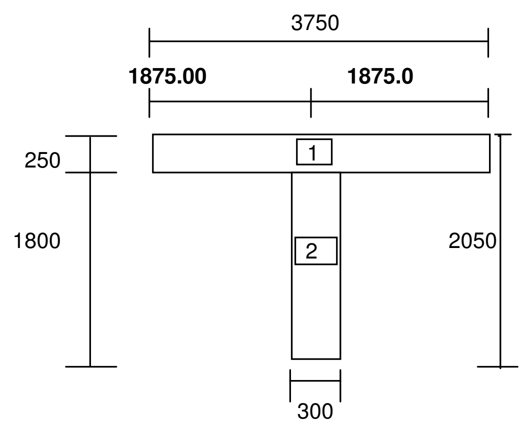
I_z

Area (Ax) = 3750 x 250
+ 1800 x 300

= 1477500 mm^2

yb = 1550 mm

yt = 500 mm



Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	4.883E+09	1.3157E+11
2	1.458E+11	2.2842E+11
	1.507E+11	3.5999E+11

Iz = 5.107E+11 mm^4

I_y

y_l = 1875 mm

y_r = 1875 mm

Element	Iself = $bd^3/12$ mm^4	A_y^2 mm^4
1	1.099E+12	0.00E+00
2	4.050E+09	0.00E+00
	1.103E+12	0.00E+00

Iy = 1.103E+12 mm^4

Ix

Element	b	d	$\frac{3.b^3.d^3}{10*(b^2+d^2)}$
	mm	mm	
1	3750	250	9.766E+09
2	300	1800	1.419E+10

Ix = 2.395E+10 mm^4

GIRDER (inner)

AT 1/8 TH SPAN

Self			**Composite**	
Depth (mm)	**2100**		Depth (mm)	2350
Area (mm^2)	1315912.36	Deck slab	Area (mm^2)	2015912
I_z (mm^4)	6.559E+11	**250**	I_z (mm^4)	1.30906E+12
Y_b(mm)	1032.73	**+** mm	Y_b(mm)	1446.7
Y_t(mm)	1067.27	thick and	Y_t(mm)	903.3
Z_b(mm^3)	6.351E+08	**2800**	Z_b(mm^3)	9.048E+08
Z_t(mm^3)	6.145E+08	mm wide.	Z_t(mm^3)	1.449E+09
Concrete M- 40			Concrete M- 40	

I_y

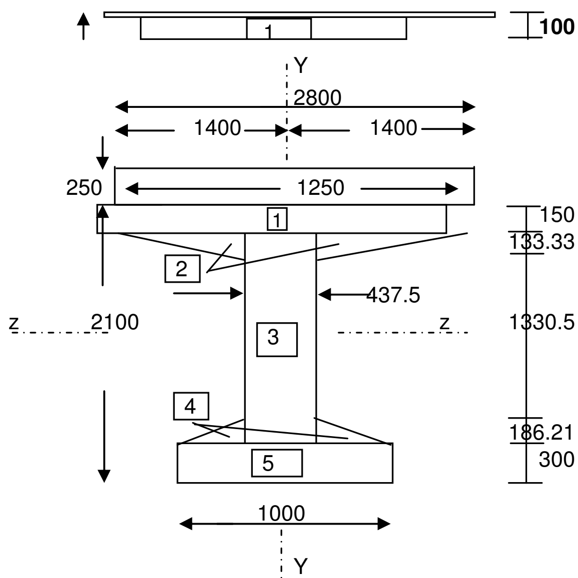
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	4.966E+08	6.794E+09
3	1.151E+10	0
4	2.301E+08	5.114E+09
5	2.500E+10	0
	6.166E+10	1.191E+10

Iy(self) = 7.356E+10 mm^4

zL = 1400.0

Iy(slab) = 4.573E+11

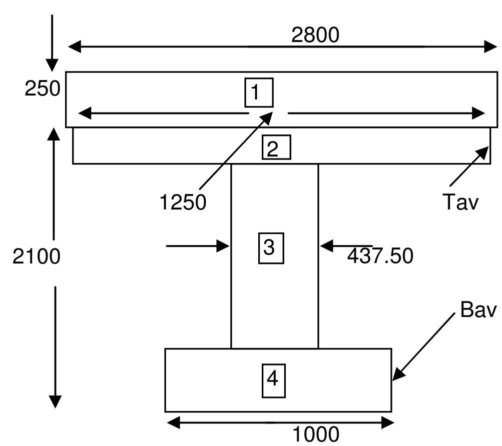
Iy(comp) = 5.309E+11



I_x

Tav = 216.7 mm
Bav = 393.1 mm

Portion no	b	d	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
	mm	mm	
1	2800	250	7.29E+09
2	1250	217	3.70E+09
3	438	1490	3.45E+10
4	1000	393.103448	1.58E+10
	Girder only		5.395E+10
	I_x (mm^4)	=	6.12E+10



for deck slab Ix = b x d^3 / 6

GIRDER (Outer)

AT 1/8 TH SPAN

Self				**Composite**	
Depth (mm)	**2100**			Depth (mm)	2350
Area (mm^2)	1315912.36		Deck slab	Area (mm^2)	2115912
I_z (mm^4)	6.56E+11		**250**	I_z (mm^4)	1.36729E+12
Y_b(mm)	1032.73	**+**	mm	Y_b(mm)	1483.5
Y_t(mm)	1067.27		thick and	Y_t(mm)	866.5
Z_b(mm^3)	6.351E+08		**3200**	Z_b(mm^3)	9.217E+08
Z_t(mm^3)	6.145E+08		mm wide.	Z_t(mm^3)	1.578E+09
Concrete M- 40				Concrete M- 40	

I_y

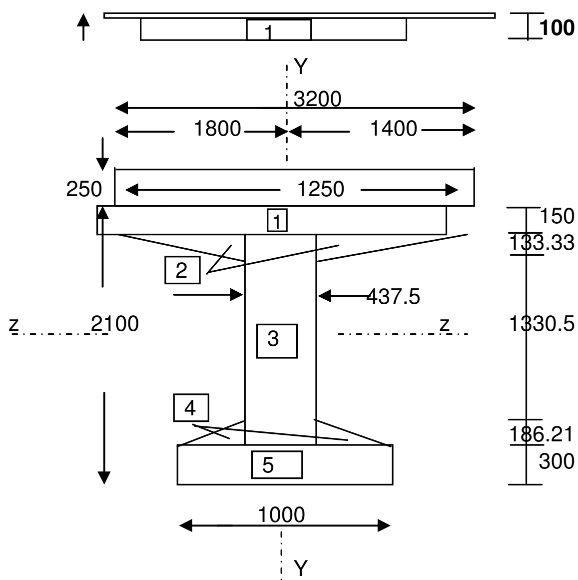
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	4.966E+08	6.794E+09
3	1.151E+10	0
4	2.301E+08	5.114E+09
5	2.500E+10	0
	6.166E+10	1.191E+10

Iy(self) = 7.356E+10 mm^4

zL = 1724.4

Iy(slab) = 6.827E+11

Iy(comp) = 7.7613E+11



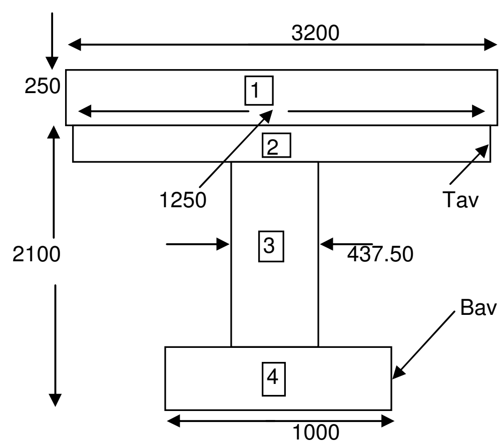
I_x

Tav = 216.7 mm
Bav = 393.1 mm

Portion no	b mm	d mm	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
1	3200	250	8.33E+09
2	1250	217	3.70E+09
3	438	1490	3.45E+10
4	1000	393.103448	1.58E+10

I_x (mm^4) = 6.23E+10

for deck slab Ix = b x d^3 / 6



GIRDER (Inner)

at effective depth d

Self			**Composite**	
Depth (mm)	**2100**		Depth (mm)	2350
Area (mm^2)	1538649.43	Deck slab	Area (mm^2)	2238649
I_z (mm^4)	6.932E+11	**250**	I_z (mm^4)	1.36228E+12
Y_b(mm)	1048.92	**+** mm	Y_b(mm)	1416.7
Y_t(mm)	1051.08	thick and	Y_t(mm)	933.3
Z_b(mm^3)	6.608E+08	**2800**	Z_b(mm^3)	9.616E+08
Z_t(mm^3)	6.595E+08	mm wide.	Z_t(mm^3)	1.460E+09
Concrete M- 40			Concrete M- 40	

I_y

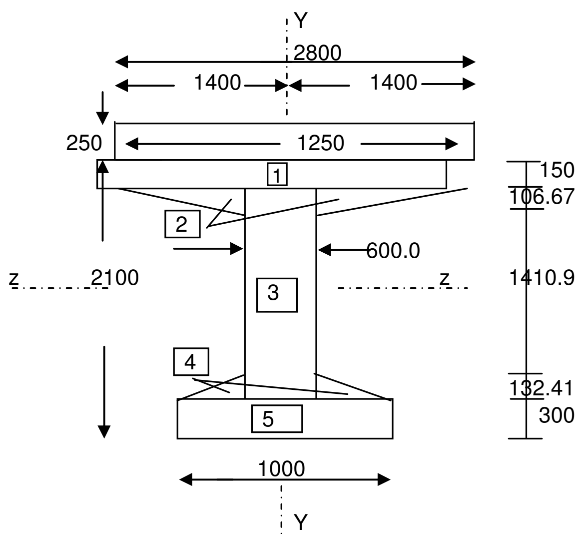
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	2.034E+08	5.780E+09
3	2.970E+10	0
4	5.885E+07	3.560E+09
5	2.500E+10	0
	7.938E+10	9.340E+09

Iy(self) = 8.872E+10 mm^4

zL = 1400.0

Iy(slab) = 4.573E+11

Iy(comp) = 5.4605E+11

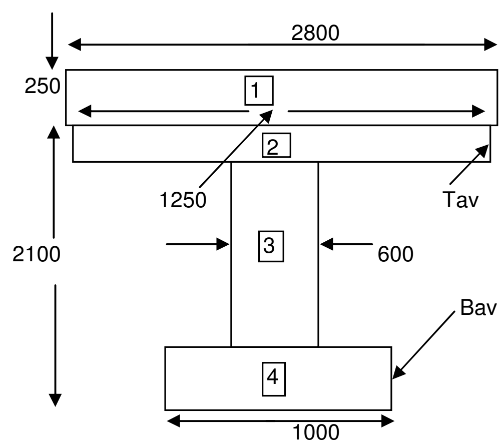


I_x

Tav = 203.3 mm

Bav = 366.2 mm

Portion no	b	d	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
	mm	mm	
1	2800	250	7.29E+09
2	1250	203	3.07E+09
3	600	1530	8.60E+10
4	1000	366.206897	1.30E+10
Girder only			1.020E+11
I_x (mm^4)		=	1.09E+11



for deck slab Ix = b x d^3 / 6

GIRDER (Outer)

AT effective depth d

Self			Composite	
Depth (mm)	**2100**		Depth (mm)	2350
Area (mm^2)	1538649.43	Deck slab	Area (mm^2)	2338649
I_z (mm^4)	6.93E+11	**250**	I_z (mm^4)	1.42534E+12
Y_b(mm)	1048.92	**+** mm	Y_b(mm)	1451.2
Y_t(mm)	1051.08	thick and	Y_t(mm)	898.8
Z_b(mm^3)	6.608E+08	**3200**	Z_b(mm^3)	9.822E+08
Z_t(mm^3)	6.595E+08	mm wide.	Z_t(mm^3)	1.586E+09
Concrete M- 40			Concrete M- 40	

I_y

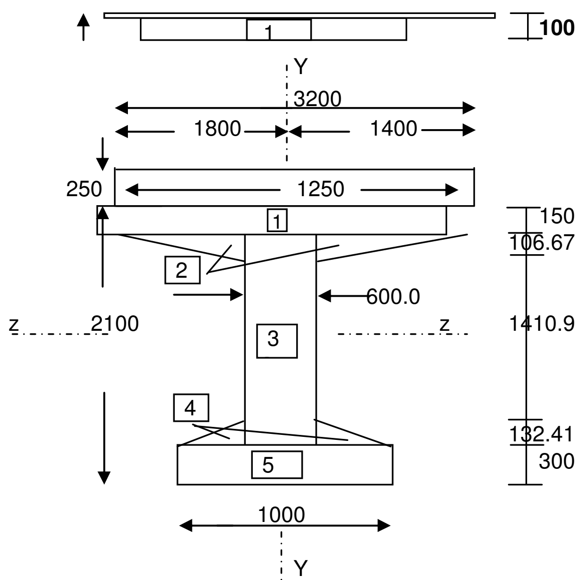
Element	Iself = $bd^3/12$ mm^4	A_z^2 mm^4
1	2.441E+10	0
2	2.034E+08	5.780E+09
3	2.970E+10	0
4	5.885E+07	3.560E+09
5	2.500E+10	0
	7.938E+10	9.340E+09

Iy(self) = 8.872E+10 mm^4

zL = 1731.6

Iy(slab) = 6.827E+11

Iy(comp) = 7.9244E+11

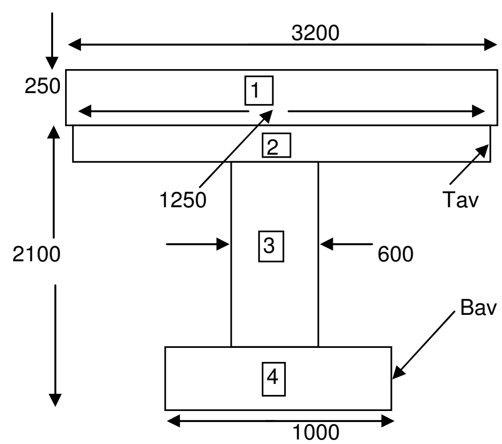


I_x

Tav = 203.3 mm
Bav = 366.2 mm

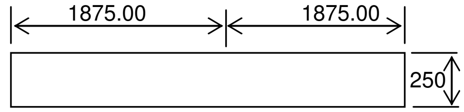
Portion no	b	d	$\frac{3.b^3.d^3}{10(b^2+d^2)}$
	mm	mm	
1	3200	250	8.33E+09
2	1250	203	3.07E+09
3	600	1530	8.60E+10
4	1000	366.206897	1.30E+10

I_x (mm^4) = 1.10E+11



for deck slab Ix = b x d^3 / 6

<u>FOR MEMBERS 107 TO 112 116 to 122 126 TO 132 136 to141</u>



Area Ax =

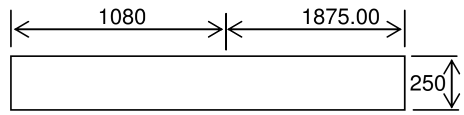
$$3.75 \times 0.25 = 0.9375 \text{ m}^2$$

$$Ix = \frac{3.75 \times 0.25^3}{6} = 0.009766 \text{ m}^4$$

$$Iy = \frac{0.25 \times 3.75^3}{12} = 1.098633 \text{ m}^4$$

$$Iz = \frac{3.75 \times 0.25^3}{12} = 0.004883 \text{ m}^4$$

<u>FOR MEMBERS 102 106 142 146</u>



Area Ax =

$$2.955 \times 0.25 = 0.73875 \text{ m}^2$$

$$Ix = \frac{2.955 \times 0.25^3}{6} = 0.007695 \text{ m}^4$$

$$Iy = \frac{0.25 \times 2.955^3}{12} = 0.537565 \text{ m}^4$$

$$Iz = \frac{2.955 \times 0.25^3}{12} = 0.003848 \text{ m}^4$$

Summary of member Properties

Member No.		Due to Composite AX	IX	IZ
1 to 16,81 to 96		0.001	0.0001	0.0001
22 to 27 70 to 75		1.906	0.0404	1.3135
38 to 43 54 to 59		1.806	0.0393	1.2605
17 31 65 79				
18 32 66 80		2.339	0.1104	1.4253
33 34 47 to 50 63 64		2.239	0.1093	1.3623
19 TO 21 28 TO 30 67TO 69 76 TO 78		2.122	0.0754	1.3694
35 TO 37 44 TO 46 51 TO 53 60 TO 62		2.022	0.0743	1.3114
103 to 105 143 to 145		1.549	0.0540	0.6285
123 to 125 113 to 115 133 to 135		1.478	0.0240	0.5107
107 to 112 116 to 122 126 to 132 136 to141		0.938	0.0098	0.0049
97 to 101 147 to 151		0.001	0.0001	0.0001
102 106 142 146		0.739	0.0077	0.0038

SUMMARY OF BENDING MOMENT AT MID SPAN

Loads	G1	G2	G3	G4
Self Weight of girder	316.27	316.27	316.27	316.27
Weight of deck slab	223.83	223.83	223.83	223.83
Weight of the shuttering	55.96	55.96	55.96	55.96
Weight of Diaphragm	44.63	44.63	44.63	44.63
SIDL	163.3	164.96	164.96	163.3
LIVE LOAD (Max)	238.92	177.7	146.88	123.46
Total	**1042.91**	**983.35**	**952.53**	**927.45**

Hence Girder G-1 shall be designed and rest of the girders will be kept same as that of G-1

SUMMARY OF BENDING MOMENT & SHEAR FORCES AT DIFFERENT SECTIONS OF G-1

Section	DL OF GIRDER		DL OF DECK SLAB		SHUTTERING		WT. OF DIAPHRAGM		SIDL		LIVE LOAD			
	B.M. (t.m.)	SF (t)	BM (t.m.)	SF (t)	BM (t.m.)	SF (t)	BM (t.m.)	SF (t)	BM (t.m.)	SF (t)	Max B.M. & Corres. SF		Max. Shear & Corres. BM	
											BM(tm)	SF(t)	BM(tm)	SF(t)
1 - 1	316.27	0.00	223.83	0	55.96	0	44.63	1.49	163.3	2	238.92	7.65	198.15	12.22
2 - 2	296.83	10.37	209.77	7.5	52.44	1.88	39.05	1.49	164.75	8.6	207.59	8.94	195.41	13.7
3 - 3	238.50	20.74	167.58	15	41.9	3.75	33.47	4.46	126.54	10.3	175.11	21.94	170.34	22.64
4 - 4	141.29	31.11	97.27	22.5	24.32	5.62	16.74	4.46	81.97	20.35	89.10	23.89	88.84	23.95
5 - 5	92.26	36.06	62.24	25.42	15.56	6.36	10.21	4.46	51.3	21.61	54.36	23.76	54.05	24.46

CALCULATION FOR FRICTION AND SLIP LOSSES

The cable is assumed to have a straight portion near ends followed by a parabolic profile. The parabolic profile is followed by plan bending. Cable profile for plan bending is assumed to be made up of two parabola. Near midspan it has a small horizontal.

No of Prestressing Cables	=	7	
Type of cable	=	12 -T-13	
Type of Sheathing	=	Bright Metal	
X-sectional area of each cable, A_s	=	11.844	cm^2
Modulus of elasticity of prestressing steel, E_s	=	1.988E+07	t/m^2
	=	(1.95 Mpa x 105)	
Span length bearing to bearing	=	30.000	m
Wobble coefficient k	=	0.0046	
Friction coefficient μ	=	0.2500	
Expected slip, s	=	6.0	mm
Half slip area (= 0.5 x A_s x E_s x s)	=	70.64	tm
Grip length inside the jack	=	0.350	m
Extra length of cable required for jack attachment	=	0.750	m
UTS of Cable	=	224.71	t
Jacking End Force	=	70.0	% of UTS
Dia of Duct	=	75	mm

CABLE PROFILE

CABLE	a = Horr. Length of inclined Portion in m.	b = Horr. Length of parabolic Portion in m.	c = Horr. Length of plan bend. Portion in m.	d = Length of strait Portion in m.	H = Y co-ordinate At jacking end	h3 = Y co-ordinate At mid span	m = plan bending in m.	Jacking force (t)	Stressing stage
1	1.000	14.250	0.000	0.000	0.300	0.115	0.000	157.30	1
2	1.000	14.250	0.000	0.000	0.300	0.115	0.000	157.30	1
3	1.000	8.250	0.000	6.000	0.600	0.115	0.000	157.30	1
4	1.000	8.250	0.000	6.000	0.900	0.265	0.000	157.30	1
5	1.000	14.250	0.000	0.000	1.200	0.415	0.000	157.30	2
6	1.000	14.250	0.000	0.000	1.500	0.565	0.000	157.30	2
7	1.000	14.250	0.000	0.000	1.800	0.715	0.000	157.30	1

CALCULATION FOR LENGTHS AND CUMULATIVE ANGLES

CABLE	A to B	A to C	A to D	A to E	A to F	Length* of cable	∂_c	∂_d	$\partial_e=\partial_f$
	(m)	(m)	(m)	(m)	(m)		(rad)	(rad)	(rad)
1	1.000	15.251	15.251	15.251	15.251	32.003	0.0228	0.0228	0.0228
2	1.000	15.251	15.251	15.251	15.251	32.003	0.0228	0.0228	0.0228
3	1.004	9.267	9.267	9.267	15.267	32.034	0.0944	0.0944	0.0944
4	1.008	9.279	9.279	9.279	15.279	32.058	0.1233	0.1233	0.1233
5	1.005	15.277	15.277	15.277	15.277	32.054	0.0963	0.0963	0.0963
6	1.007	15.288	15.288	15.288	15.288	32.076	0.1146	0.1146	0.1146
7	1.009	15.301	15.301	15.301	15.301	32.102	0.1328	0.1328	0.1328

* Total length of cable includes 750mm extra gripping length required for jack attachment.

∂ is total angular change w.r.t. node A

FORCES IN CABLES AFTER FRICTION ONLY

CABLE	k	F_A (t)	F_B (t)	F_C (t)	F_D (t)	F_E (t)	F_F (t)	Average Force (t)
1	7.99E-04	157.30	156.57	145.81	145.81	145.81	145.81	151.57
2	7.99E-04	157.30	156.57	145.81	145.81	145.81	145.81	151.57
3	5.74E-03	157.30	156.57	147.22	147.22	147.22	143.21	149.60
4	7.51E-03	157.30	156.57	146.15	146.15	146.15	142.17	148.90
5	3.39E-03	157.30	156.57	143.13	143.13	143.13	143.13	150.32
6	4.04E-03	157.30	156.57	142.47	142.47	142.47	142.47	150.01
7	4.69E-03	157.30	156.57	141.82	141.82	141.82	141.82	149.70

k is coefficient of parabolic equation $y = k\,x^2$ (in vertical plane)

SLIP DISTANCE CALCULATION

CABLE	AB	BC	CD	DE	EF	F2	F1	d	F_p (t)
1	-70.28	17.22	17.22	17.22	17.22	156.57	145.81	14.25	147.00
2	-70.28	17.22	17.22	17.22	17.22	156.57	145.81	14.25	147.00
3	-70.27	-22.24	-22.24	-22.24	26.92	147.22	143.21	6.00	145.16
4	-70.27	-16.68	-16.68	-16.68	32.17	146.15	142.17	6.00	144.56
5	-70.27	39.12	39.12	39.12	39.12	156.57	143.13	14.27	145.98
6	-70.27	44.57	44.57	44.57	44.57	156.57	142.47	14.28	145.74
7	-70.27	50.00	50.00	50.00	50.00	156.57	141.82	14.29	145.52

FORCES IN CABLES AFTER SLIP

Cable	Slip distance	F_A (t)	F_B (t)	F_C (t)	F_D (t)	F_E (t)	F_F (t)	Avg. Force (t)	Elongation (mm)
1	13.676	136.70	137.42	145.81	145.81	145.81	145.81	141.32	100.51
2	13.676	136.70	137.42	145.81	145.81	145.81	145.81	141.32	100.51
3	12.348	133.02	133.75	143.10	143.10	143.10	143.21	139.95	99.34
4	11.680	131.82	132.55	142.97	142.97	142.97	142.17	139.28	98.96
5	12.259	134.65	135.38	143.13	143.13	143.13	143.13	138.98	99.87
6	11.976	134.19	134.92	142.47	142.47	142.47	142.47	138.42	99.74
7	11.714	133.75	134.48	141.82	141.82	141.82	141.82	137.88	99.62

FORCES IN CABLES AT SECTION 1-1 (MIDSPAN) AFTER SLIP

Cable no	Stage	Length of cable from midspan	∂ (rad)	φ (rad)	Co-ord from bottom	P (t)	$P\cos\partial$ (t)	$P\sin\partial$ (t)
1	1	0.00	0.00	0.00	0.115	145.81	145.81	0.00
2	1	0.00	0.00	0.00	0.115	145.81	145.81	0.00
3	1	0.00	0.00	0.00	0.115	143.21	143.21	0.00
4	1	0.00	0.00	0.00	0.265	142.17	142.17	0.00
5	2	0.00	0.00	0.00	0.415	143.13	143.13	0.00
6	2	0.00	0.00	0.00	0.565	142.47	142.47	0.00
7	1	0.00	0.00	0.00	0.715	141.82	141.82	0.00

∂ is angle of cable in vertical plane w.r.t. soffit slab

φ is angle of cable in horizontal plane

Stage 1	Total horizontal force	=	718.8	t
	ecc wrt bottom	=	0.263	m
	Total vertical force	=	0.000	t
Stage 2	Total horizontal force	=	285.6	t
	ecc wrt bottom	=	0.490	m
	Total vertical force	=	0.00	t

FORCES IN CABLES AT SECTION 2-2 (THREE EIGHTH SECTION) AFTER SLIP

3.750 m FROM MIDSPAN

Cable no	Stage	Length from midspan	∂ (rad)	φ (rad)	Co-ord from bottom	P (t)	Pcos∂ (t)	Psin∂ (t)
1	1	3.750	0.006	0.000	0.126	145.43	145.43	0.87
2	1	3.750	0.006	0.000	0.126	145.43	145.43	0.87
3	1	3.750	0.000	0.000	0.115	144.62	144.62	0.00
4	1	3.750	0.000	0.000	0.265	144.47	144.47	0.00
5	2	3.750	0.025	0.000	0.463	145.40	145.35	3.70
6	2	3.751	0.030	0.000	0.622	145.43	145.37	4.40
7	1	3.751	0.035	0.000	0.781	145.48	145.39	5.11

Stage 1	Total horizontal force	=	725.3 t
	ecc wrt bottom	=	0.283 m
	Total vertical force	=	6.854 t
Stage 2	Total horizontal force	=	290.72 t
	ecc wrt bottom	=	0.542 m
	Total vertical force	=	8.100 t
Stage 3	Total horizontal force =		0.000 t
	ecc wrt bottom =		0.000 m
	Total vertical force =	=	0.000 t

FORCES IN CABLES AT SECTION 3-3 (ONE FOURTH SECTION) AFTER SLIP

7.500 m FROM MIDSPAN

Cable	Stage	Length from midspan	∂ (rad)	φ (rad)	Co-ord from bottom	P (t)	Pcos∂ (t)	Psin∂ (t)
1	1	7.500	0.012	0.000	0.160	142.62	142.61	1.71
2	1	7.500	0.012	0.000	0.160	142.62	142.61	1.71
3	1	7.500	0.017	0.000	0.128	141.44	141.42	2.43
4	1	7.500	0.023	0.000	0.282	141.12	141.09	3.18
5	2	7.503	0.051	0.000	0.606	141.89	141.71	7.21
6	2	7.505	0.061	0.000	0.792	141.76	141.50	8.58
7	1	7.506	0.070	0.000	0.979	141.64	141.29	9.95

Stage 1	Total horizontal force	=	709.0 t
	ecc wrt bottom	=	0.341 m
	Total vertical force	=	18.977 t
Stage 2	Total horizontal force	=	283.2 t
	ecc wrt bottom	=	0.699 m
	Total vertical force	=	15.79 t

FORCES IN CABLES AT SECTION 4-4 (ONE EIGHTH SECTION) AFTER SLIP

11.250 m FROM MIDSPAN

Cable	Stage	Length from midspan	∂ (rad)	φ (rad)	Co-ord from bottom	P	Pcos∂ (t)	Psin∂ (t)
1	1	11.251	0.0180	0.000	0.216	139.75	139.729	2.512
2	1	11.251	0.0180	0.000	0.216	139.75	139.729	2.512
3	1	11.253	0.0602	0.000	0.273	137.21	136.965	8.258
4	1	11.255	0.0788	0.000	0.472	136.41	135.985	10.744
5	2	11.261	0.0763	0.000	0.844	138.30	137.902	10.539
6	2	11.265	0.0909	0.000	1.076	137.99	137.419	12.519
7	1	11.271	0.1054	0.000	1.308	137.69	136.923	14.489

Stage 1	Total horizontal force	=	689.3 t
	ecc wrt bottom	=	0.495 m
	Total vertical force	=	38.515 t
Stage 2	Total horizontal force	=	275.32 t
	ecc wrt bottom	=	0.960 m
	Total vertical force	=	23.058 t

FORCES IN CABLES AT SECTION 5-5 (Effective depth d) AFTER SLIP

12.713 m FROM MIDSPAN

Cable no	Stage	Length from midspan	∂ (rad)	φ (rad)	Co-ord from bottom	P (t)	Pcos∂ (t)	Psin∂ (t)
1	1	12.713	0.020	0.000	0.244	138.62	138.59	2.82
2	1	12.713	0.020	0.000	0.244	138.62	138.59	2.82
3	1	12.719	0.077	0.000	0.373	135.53	135.13	10.43
4	1	12.724	0.101	0.000	0.603	134.53	133.84	13.54
5	2	12.728	0.086	0.000	0.963	136.88	136.37	11.78
6	2	12.735	0.103	0.000	1.218	136.49	135.77	13.99
7	1	12.743	0.119	0.000	1.472	136.12	135.15	16.18

Stage 1	Total horiontal force =	681.3 t	TRUE
	ecc wrt bottom =	0.584 m	
	Total vertical force =	45.772 t	0.00E+00
Stage 2	Total horiontal force =	272.1 t	
	ecc wrt bottom =	1.090 m	
	Total vertical force =	25.77 t	

STRESS CHECK AT VARIOUS SECTIONS

Type of cable	=	**12-T-13**			
Area of each cable	=	1184.4 sq.mm			
UTS of one cable	=	224.7 t			
Jacking stress	=	**70.0%** of UTS			
Jacking force	=	157.29 t			
Modulus of Elasticity of	=				
HTS, Es	=	1.95E+05 MPa	=	1.99E+07 t/m^2	
Grade of concrete	=	**M - 40**			
f_{ck}	=	40 N/mm^2			
Modulus of Elasticity of Concrete, Ec	=	$5700x(f_{ck})^{1/2}$			
	=	3.22E+04 MPa	=	3.29E+06 t/m^2	(At 4th Day)
		3.60E+04 MPa	=	3.68E+06 t/m^2	(At 28th Day)

The girders shall be cast on the ground near bridge site. The stressing shall be done when cube strength of concrete reached 32 MPa. Thereafter the girders will be launched / shifted to the bridge location and placed on permanent bearings and deck slab & diaphragm shall be casted.

f_{cj} at release = 32 MPa

Maturity of concrete at the time of release as % of f_{ck} = 80.00 %

E_c = $5700x(f_{cj})^{1/2}$ = 3.22E+04 MPa = 3.29E+06 t/m2

Creep strain (per 10MPa) as per Table 2 of IRC:18 - 2000 From 14th day (80% maturity) to 28th day (100% maturity)	
Creep strain for 80% maturity	0.000510
Creep strain for 100% maturity	0.000400
Differential Creep strain between 14th and 28th day =	0.000110
Residual Creep strain 28th day onward =	0.000400

Residual shrinkage strain as per Table 3 of IRC:18 - 2000	
Residual Shrinkage strain (εs) at 14th day =	0.00025
Residual Shrinkage strain (ε_s) at 28th day =	0.00019
Therefore, Shrinkage strain (ε_s)between 14th & 28th day	0.00006
Residual Shrinkage strain 28th day onward	0.00019

Loss due to Relaxation for low Relaxation steel as per Table 4A of IRC:18 - 2000		
at average initial force of	0.5 f_p is	0 MPa
at average initial force of	0.7 f_p is	97 MPa
at average initial force of	0.8 f_p is	200 MPa

STRESS CHECK AT MID SPAN (SEC 1-1)

At first stage 4 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb	=	0.263 m
Eccentricity of cables from c.g., e	=	0.746 m
Sectional Modulus at C.G. of cables, Zc (for precast girder only)	=	0.835 m^3
B.M. due to S.Wt. of the girder M_d	=	316.27 tm
B.M.due to Cast in Situ slab M_c	=	324.42 tm
Prestressing force (after friction and slip losses) , P	=	718.82 t
Moment due to P, M_p = P x e	=	536.17 tm

Stresses due to self weight of the girder

$\sigma_t = \frac{316.27}{0.571} = 554.02$ t/m^2

$\sigma_c = \frac{316.27}{0.835} = -378.77$ t/m^2

$\sigma_b = \frac{316.27}{0.617} = -512.34$ t/m^2

Stresses due to prestress at transfer stage

$\sigma_t = \frac{718.82}{1.106} - \frac{536.17}{0.571} = -289.5$ t/m^2

$\sigma_c = \frac{718.82}{1.106} + \frac{536.17}{0.835} = 1291.90$ t/m^2

$\sigma_b = \frac{718.82}{1.106} + \frac{536.17}{0.617} = 1518.3$ t/m^2

Stresses due to initial prestress & self weight of the girder

σ_t = 554.0 + -289.5 = 264.6 t/m^2

σ_c = -378.8 + 1291.9 = 913.1 t/m^2

σ_b = -512.3 + 1518.3 = 1006.0 t/m^2

<u>Losses due to Elastic Shortening</u>

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss = 2759 t/m^2

(i.e. Elastic Shortening loss) = **16.34 t =** 2.08 % of Jacking Force

Prestressing force after initial losses P_i = 702.48 t

Moment due to P_i = M_{pi} = P_i x e = 523.98 tm

<u>Loss due to Shrinkage of Concrete</u> = ε_s x E_s

= 1192.7 t/m^2

= **7.06 t =** 0.90 % of Jacking Force

Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.685

Loss due to friction & slip = 786.4653 - 718.8 = 67.6 t

% friction and slip loss $\frac{67.6}{786.465} \times 100 = 6.021$ %

Ratio of average force to UTS = 0.685 - 0.060 = 0.625

Corresponding Relaxation Loss = 6195.8 t/m^2

= **36.69 t** 4.67 % of Jacking Force

Loss due to Relaxation & Shrinkage = 43.75 t

Moment due to Relaxation & Shrinkage loss = 32.637 tm

Stresses due to Prestress after initial losses

$\sigma_t = \frac{702.48}{1.106} - \frac{523.98}{0.571} = -282.88$ t/m^2

$\sigma_c = \frac{702.48}{1.106} + \frac{523.98}{0.835} = 1262.5$ t/m^2

$\sigma_b = \frac{702.48}{1.106} + \frac{523.98}{0.617} = 1483.8$ t/m^2

Stresses due to prestress after initial losses & self weight of the girder

$\sigma_t = 554.0 + -282.9 = 271.1$ t/m^2

$\sigma_c = -378.8 + 1262.5 = 883.8$ t/m^2

$\sigma_b = -512.3 + 1483.8 = 971.5$ t/m^2

Stresses after Shrinkage and Relaxation losses

$\sigma_t = 271.15 - \frac{43.75}{1.106} + \frac{32.637}{0.571} = 288.77$ t/m^2

$\sigma_c = 883.8 - \frac{43.75}{1.106} - \frac{32.637}{0.835} = 805.13$ t/m^2

$\sigma_b = 971.49 - \frac{43.75}{1.106} - \frac{32.637}{0.617} = 879.07$ t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 844.44 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 1810.2 t/m^2

= **10.72 t =** 1.363 % of Jacking Force

Total Initial & Intermediate losses = **70.82 t =** 9.00 % of Jacking Force

(i.e. losses upto 28th day)

Force after intermediate losses, P_{int} = 648.0 t

Moment due to P_{int} = $M_{pint} = P_{int} \times e$ = 483.35 tm

Effect of prestress after Intermediate loss

$\sigma_t = \frac{648.0}{1.106} - \frac{483.35}{0.571} = -260.9$ t/m^2

$\sigma_c = \frac{648.0}{1.106} + \frac{483.35}{0.835} = 1164.6$ t/m^2

$\sigma_b = \frac{648.0}{1.106} + \frac{483.35}{0.617} = 1368.8$ t/m^2

Stresses Due to cast-in-situ slab

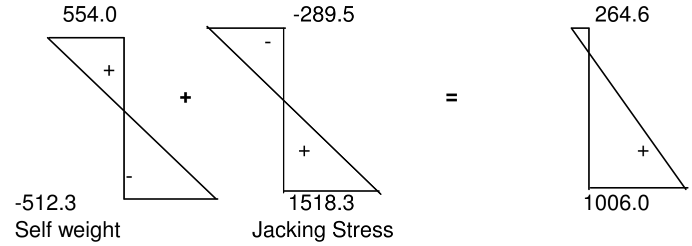
$\sigma_t = \frac{324.4}{0.571} = 568.30$ t/m^2

$\sigma_c = \frac{324.4}{0.835} = -388.53$ t/m^2

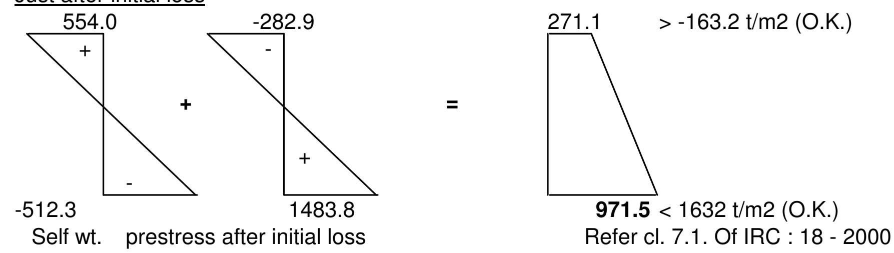
$\sigma_b = \frac{324.4}{0.617} = -525.54$ t/m^2

RESULTANT STRESSES

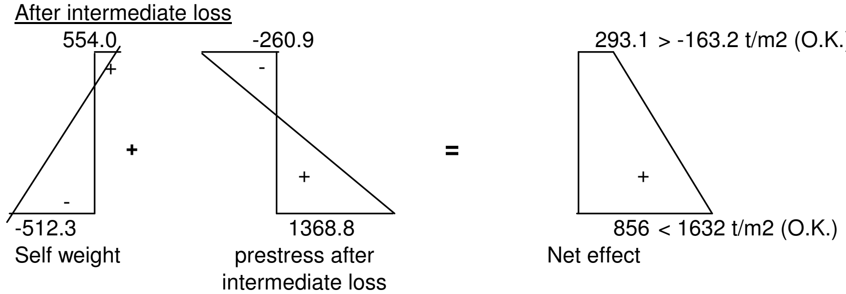
a) Transfer stage



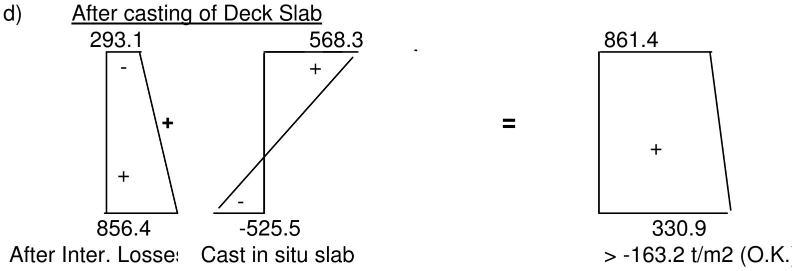
b) Just after initial loss



c) After intermediate loss



d) After casting of Deck Slab



STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section	=	1.5812	m^3
Section Modulus at junction of deck slab & girder Z_i	=	2.2620	m^3
Section Modulus at C.G. of cables Zc	=	1.1067	m^3
Section Modulus at bottom of composite section	=	0.8646	m^3
Composite area of deck slab + outer girder	=	1.9063	m^2
first stage cables			
Prestressing force (after 1st stage) , P_1	=	648.00	t
ecc. of cables from bottom, eb (Ist stage)	=	0.263	m
Eccentricity of cables from c.g., e (Ist stage)	=	0.746	m
second stage cables			
Prestressing force (after friction and slip losses) , P	=	285.61	t
ecc. of cables from bottom, eb (2nd stage)	=	0.490	m
Eccentricity of cables from c.g., e (2nd stage)	=	1.029	m
Net eccentricity of cables of 1st and 2nd stages, eb'	=	0.332	m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, e'	=	1.187	m (from c.g.)

B.M. due to S.Wt. of the girder M_d	=	316.27 tm
B.M.due to Cast in Situ slab M_c (without shuttering)	=	268.46 tm
B.M. due to SIDL M_s	=	163.30 tm
B.M. due to Live loads M_L	=	238.92 tm
B.M. due to FPLL	=	0.00 tm
Prestressing force (after 1st stage) , P_1	=	648.00 t
Moment due to P_{int} = $M_{pint} = P_1 \times e$	=	483.35 tm
for second stage prestressing		
Prestressing force (after friction and slip losses) , P	=	285.61 t
Moment due to P, $M_p = P \times e$	=	294.03 tm

Stresses due to self weight of the girder + cast in situ slab

$\sigma_j = \dfrac{584.73}{0.571} = 1024.3$ t/m^2

$\sigma_c = \dfrac{584.73}{0.835} = -700.3$ t/m^2

$\sigma_b = \dfrac{584.73}{0.617} = -947.2$ t/m^2

Stresses due to prestress (second stage cables)

$\sigma_t = \dfrac{285.61}{1.906} - \dfrac{294.03}{1.581} = -36.1$ t/m^2

$\sigma_i = \dfrac{285.61}{1.906} - \dfrac{294.03}{2.262} = 19.8$ t/m^2

$\sigma_c = \dfrac{285.61}{1.906} + \dfrac{294.03}{1.107} = 415.5$ t/m^2

$\sigma_b = \dfrac{285.61}{1.906} + \dfrac{294.03}{0.865} = 489.9$ t/m^2

Total Stresses due to prestress

σ_t =	0.0	+	-36.1	=	-36.1	t/m^2
σ_j =	-260.9	+	19.8	=	-241.1	t/m^3
σ_c =	1164.6	+	415.5	=	1580.1	t/m^2
σ_b =	1368.8	+	489.9	=	1858.7	t/m^2

Stresses due to prestress & self weight of the girder+deck slab

σ_t =	0.0	+	-36.1	=	-36.1	t/m^2
σ_j =	1024.3	+	-241.1	=	783.2	t/m^3
σ_c =	-700.3	+	1580.1	=	879.8	t/m^2
σ_b =	-947.2	+	1858.7	=	911.4	t/m^2

Losses due to Elastic Shortening

Loss due to Elastic Shortening = $0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss = 2378 t/m^2

(i.e. Elastic Shortening loss) = **19.72 t =** 1.79 % of Jacking Force

For first stage cables

Prestressing force after initial losses P_{ii} = 642.37 t

Moment due to $P_i = M_{pii} = P_{ii} \times e$ = 479.15 tm

For second stage cables

Prestressing force after initial losses P_i = 279.98 t

Moment due to $P_i = M_{pi} = P_i \times e$ = 288.23 tm

Loss due to Shrinkage of Concrete = $\varepsilon_s \times E_s$

= 3776.8 t/m^2

= **31.31 t =** 2.84 % of Jacking Force

Loss due to Shrinkage = 36.95 t

Moment due to Shrinkage loss = 43.850 tm

<u>Loss due to relaxation of steel</u>

For second stage cables:

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.694

Loss due to friction and slip = 314.5861 - 285.6 = 29.0 t

% friction and slip loss $\frac{29.0}{314.58612}$ x 100 = 6.45 %

Ratio of average force to UTS = 0.694 - 0.064479 = 0.629

Corresponding Relaxation Loss = 6394.2 t/m^2
= **15.15 t** 4.81 % of Jacking Force

Loss due to Relaxation & Shrinkage = 20.78 t
Moment due to Relaxation & Shrinkage loss = 24.663 tm

Shrinkage losses (for first stage cables)

$\sigma_t = \frac{-36.95}{1.906} + \frac{43.850}{1.581} = 8.35$ t/m^2

$\sigma_i = \frac{-36.95}{1.906} + \frac{43.850}{2.262} = 0.00$ t/m^2

$\sigma_c = \frac{-36.95}{1.906} - \frac{43.850}{1.107} = -59.00$ t/m^2

$\sigma_b = \frac{-36.95}{1.906} - \frac{43.850}{0.865} = -70.10$ t/m^2

Shrinkage and Relaxation losses (for second stage cables)

$\sigma_t = \frac{-20.78}{1.906} + \frac{24.663}{1.581} = 4.70$ t/m^2

$\sigma_j = \frac{-20.78}{1.906} + \frac{24.663}{2.262} = 0.00$ t/m^2

$\sigma_c = \frac{-20.78}{1.906} - \frac{24.663}{1.107} = -33.19$ t/m^2

$\sigma_b = \frac{-20.78}{1.906} - \frac{24.663}{0.865} = -39.43$ t/m^2

Total Shrinkage and Relaxation losses

σ_t =	8.35	+	4.70	=	13.05	t/m^2
σ_j =	0.00	+	0.00	=	0.01	t/m^2
σ_c =	-59.00	+	-33.19	=	-92.19	t/m^2
σ_b =	-70.10	+	-39.43	=	-109.53	t/m^2

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab

σ_t =	-36.1	+	13.05	=	-23.07	t/m^2
σ_j =	783.2	+	0.0	=	783.20	t/m^2
σ_c =	879.8	+	-92.2	=	787.66	t/m^2
σ_b =	911.4	+	-109.5	=	801.92	t/m^2

<u>Loss due to Creep of concrete</u>

Airthmetic mean of initial and final stress after 28 days = 787.66 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 6139.9 t/m^2
= **50.91 t =** 4.623 % of Jacking Force

Loss due to Creep = 50.91 t
Moment due to Creep loss = 60.42 tm

Creep losses (for first stage cables)

$\sigma_t = \dfrac{-50.91}{1.906} + \dfrac{60.418}{1.581} = 11.51$ t/m^2

$\sigma_i = \dfrac{-50.91}{1.906} + \dfrac{60.418}{2.262} = 0.01$ t/m^2

$\sigma_c = \dfrac{-50.91}{1.906} - \dfrac{60.418}{1.107} = -81.30$ t/m^2

$\sigma_b = \dfrac{-50.91}{1.906} - \dfrac{60.418}{0.865} = -96.59$ t/m^2

Creep losses (for second stage cables)

$\sigma_t = \dfrac{-14.54}{1.906} + \dfrac{-17.262}{1.581} = -18.55$ t/m^2

$\sigma_i = \dfrac{-14.54}{1.906} + \dfrac{-17.262}{2.262} = -15.26$ t/m^2

$\sigma_c = \dfrac{-14.54}{1.906} - \dfrac{-17.262}{1.107} = 7.97$ t/m^2

$\sigma_b = \dfrac{-14.54}{1.906} - \dfrac{-17.262}{0.865} = 12.34$ t/m^2

Final Prestressing Force = 775.57 t

Total Creep losses

σ_t =	11.51	+	-18.55	=	-7.04	t/m^2
σ_i =	0.01	+	-15.26	=	-15.26	t/m^2
σ_c =	-81.30	+	7.97	=	-73.33	t/m^2
σ_b =	-96.59	+	12.34	=	-84.25	t/m^2

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

σ_t =	-23.07	+	-7.04	=	-30.11
σ_i =	783.20	+	-15.26	=	767.95
σ_c =	787.66	+	-73.33	=	714.33
σ_b =	801.92	+	-84.25	=	717.67

Stresses Due to SIDL

$\sigma_t = \dfrac{163.30}{1.581} = 103.3$ t/m^2

$\sigma_j = \dfrac{163.30}{2.262} = 72.2$ t/m^2

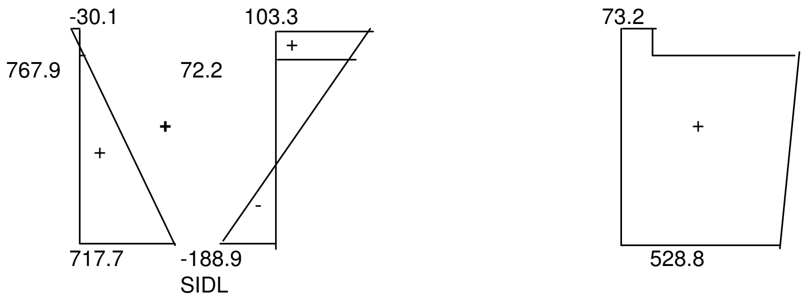
$\sigma_c = \dfrac{163.30}{1.107} = -147.6$ t/m^2

$\sigma_b = \dfrac{163.30}{0.865} = -188.9$ t/m^2

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

σ_t =	-30.1	+	103.27	=	73.2	t/m^2
σ_j =	767.9	+	72.19	=	840.1	t/m^2
σ_c =	714.3	+	-147.55	=	566.8	t/m^2
σ_b =	717.7	+	-188.88	=	528.8	t/m^2

After laying of SIDL



Stresses due to Live load

$\sigma_t = \dfrac{238.92}{1.581} = 151.10$ t/m^2

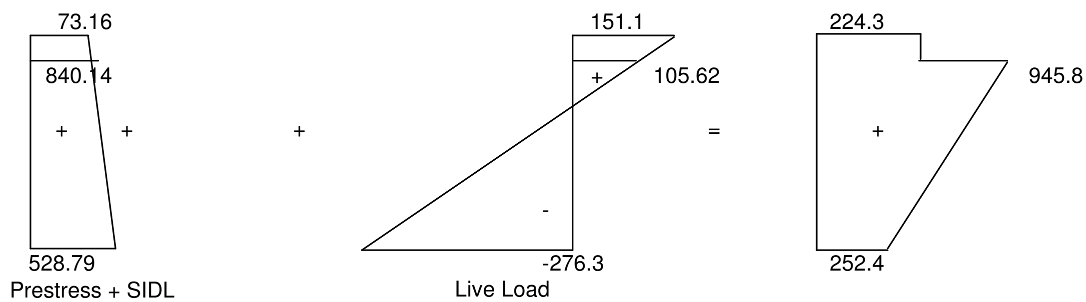
$\sigma j = \dfrac{238.92}{2.262} = 105.62$ t/m^2

$\sigma_c = \dfrac{238.92}{1.107} = -215.881$ t/m^2

$\sigma_b = \dfrac{238.92}{0.865} = -276.35$ t/m^2

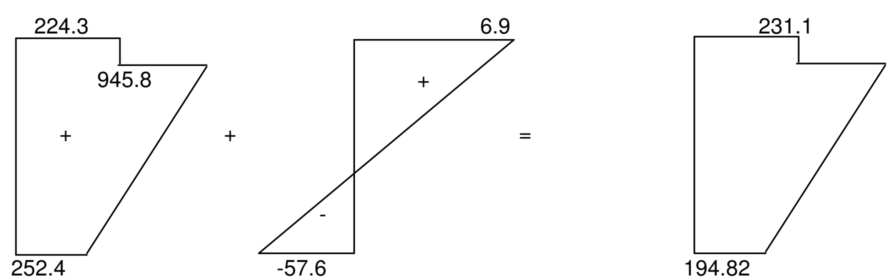
Stresses due to Prestress + DL + SIDL + FPLL + LL

σ_t =	73.16	+	151.1	=	224.3	t/m^2
σ_j =	840.14	+	105.6	=	945.8	t/m^2
σ_c =	566.78	+	-215.9	=	350.9	t/m^2
σ_b =	528.79	+	-276.3	=	252.4	t/m^2



Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

	=	151.84 t
20% extra time dependant losses	=	30.37 t
Stress due to 20% extra time dependant losses at top	=	**6.9 t/m2**
Stress due to 20% extra time dependant losses at bottom	=	**-57.6 t/m2**



STRESS CHECK AT 3-8 TH (SEC 2-2)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb	=	0.283	m
Eccentricity of cables from c.g., e	=	0.726	m
Sectional Modulus at C.G. of cables, Zc (for precast girder only)	=	0.858	m^3
B.M. due to S.Wt. of the girder M_d	=	296.83	tm
B.M.due to Cast in Situ slab M_c	=	301.26	tm
Prestressing force (after friction and slip losses) , P	=	725.33	t
Moment due to P, M_p = P x e	=	526.66	tm

Stresses due to self weight of the girder

$\sigma_t = \frac{296.83}{0.571} = 519.97$ t/m^2

$\sigma_c = \frac{296.83}{0.858} = -346.04$ t/m^2

$\sigma_b = \frac{296.83}{0.617} = -480.85$ t/m^2

Stresses due to prestress at transfer stage

$\sigma_t = \frac{725.33}{1.106} - \frac{526.66}{0.571} = -266.9$ t/m^2

$\sigma_c = \frac{725.33}{1.106} + \frac{526.66}{0.858} = 1269.64782$ t/m^2

$\sigma_b = \frac{725.33}{1.106} + \frac{526.66}{0.617} = 1508.8$ t/m^2

Stresses due to initial prestress & self weight of the girder

$\sigma_t = 520.0 + -266.9 = 253.1$ t/m^2

$\sigma_c = -346.0 + 1269.6 = 923.6$ t/m^2

$\sigma_b = -480.8 + 1508.8 = 1028.0$ t/m^2

<u>Losses due to Elastic Shortening</u>

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss = 2791 t/m^2

(i.e. Elastic Shortening loss) = **16.53 t =** 2.10 % of Jacking Force

Prestressing force after initial losses P_i = 708.81 t

Moment due to P_i = M_{pi} = P_i x e = 514.66 tm

<u>Loss due to Shrinkage of Concrete</u> = ε_s x E_s

= 1192.7 t/m^2

= **7.06 t =** 0.90 % of Jacking Force

Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.685

Loss due to friction & slip = 786.4653 - 725.3 = 61.1 t

% friction and slip loss $\frac{61.1}{786.46529}$ x 100 = 5.44 %

Ratio of average force to UTS = 0.685 - 0.05441 = 0.631

Corresponding Relaxation Loss = 6474.5 t/m^2

= **38.34 t** 4.88 % of Jacking Force

Loss due to Relaxation & Shrinkage = 45.41 t

Moment due to Relaxation & Shrinkage loss = 32.968 tm

Stresses due to Prestress after initial losses

$\sigma_t = \frac{708.81}{1.106} - \frac{514.66}{0.571} = -260.82$ t/m^2

$\sigma_c = \frac{708.81}{1.106} + \frac{514.66}{0.858} = 1240.7$ t/m^2

$\sigma_b = \frac{708.81}{1.106} + \frac{514.66}{0.617} = 1474.4$ t/m^2

Stresses due to prestress after initial losses & self weight of the girder

$\sigma_t = 520.0 + -260.8 = 259.1$ t/m^2

$\sigma_c = -346.0 + 1240.7 = 894.7$ t/m^2

$\sigma_b = -480.8 + 1474.4 = 993.6$ t/m^2

Stresses after Shrinkage and Relaxation losses

$\sigma_t = 259.15 - \frac{45.41}{1.106} + \frac{32.968}{0.571} = 275.85$ t/m^2

$\sigma_c = 894.7 - \frac{45.41}{1.106} - \frac{32.968}{0.858} = 815.19$ t/m^2

$\sigma_b = 993.60 - \frac{45.41}{1.106} - \frac{32.968}{0.617} = 899.15$ t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 854.93 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 1832.7 t/m^2

= **10.85 t =** 1.380 % of Jacking Force

Total Initial & Intermediate losses = **72.79 t =** 1.38 % of Jacking Force

(i.e. losses upto 28th day)

Force after intermediate losses, P_{int} = 652.5 t

Moment due to P_{int} = $M_{pint} = P_{int} \times e$ = 473.81 tm

Effect of prestress after Intermediate loss

$\sigma_t = \frac{652.5}{1.106} - \frac{473.81}{0.571} = -240.1$ t/m^2

$\sigma_c = \frac{652.5}{1.106} + \frac{473.81}{0.858} = 1142.2$ t/m^2

$\sigma_b = \frac{652.5}{1.106} + \frac{473.81}{0.617} = 1357.4$ t/m^2

Stresses Due to cast-in-situ slab

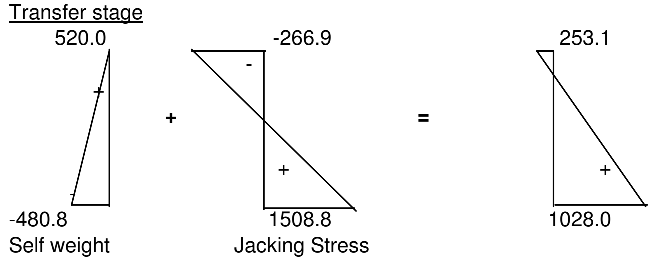
$\sigma_t = \frac{301.3}{0.571} = 527.73$ t/m^2

$\sigma_c = \frac{301.3}{0.858} = -351.21$ t/m^2

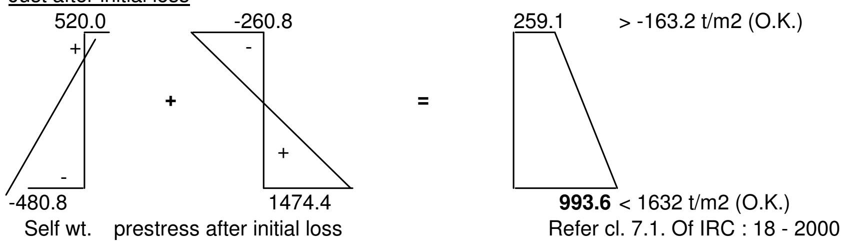
$\sigma_b = \frac{301.3}{0.617} = -488.02$ t/m^2

RESULTANT STRESSES

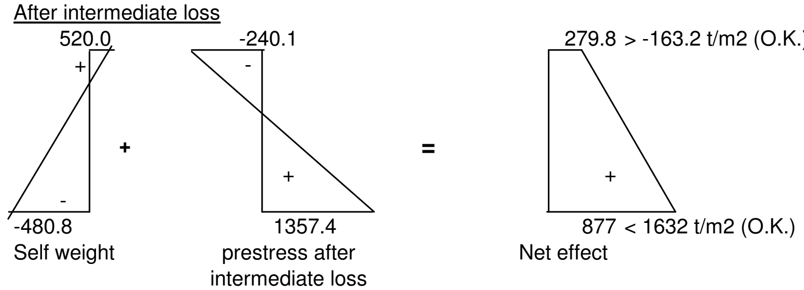
a) Transfer stage



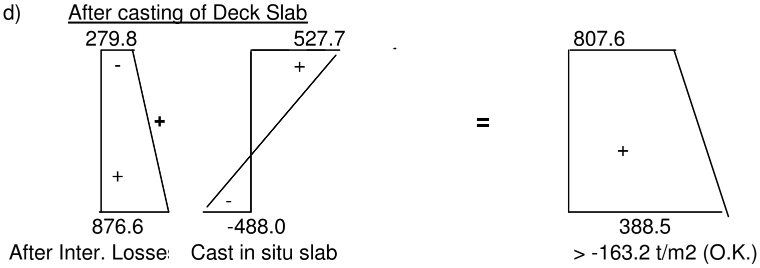
b) Just after initial loss



c) After intermediate loss



d) After casting of Deck Slab



STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section	=	1.58123097	m^3
Section Modulus at junction of deck slab & girder Z_j	=	2.2620	m^3
Section Modulus at C.G. of cables Zc	=	1.136	m^3
Section Modulus at bottom of composite section	=	0.8646	m^3
Composite area of deck slab + outer girder	=	1.9063	m^2
first stage cables			
Prestressing force (after 1st stage) , P_1	=	652.55	t
ecc. of cables from bottom, eb (Ist stage)	=	0.283	m
Eccentricity of cables from c.g., e (Ist stage)	=	0.726	m
second stage cables			
Prestressing force (after friction and slip losses) , P	=	290.72	t
ecc. of cables from bottom, eb (2nd stage)	=	0.542	m
Eccentricity of cables from c.g., e (2nd stage)	=	0.977	m
Net eccentricity of cables of 1st and 2nd stages, eb'	=	0.363	m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, e'	=	1.156	m (from c.g.)

B.M. due to S.Wt. of the girder M_d	=	296.83 tm
B.M.due to Cast in Situ slab M_c	=	245.30 tm
B.M. due to SIDL M_s	=	164.75 tm
B.M. due to Live loads M_L	=	207.59 tm
B.M. due to FPLL	=	0.00 tm

Prestressing force (after 1st stage) , P_1	=	652.55 t
Moment due to P_{int} = $M_{pint} = P_1 \times e$	=	473.81 tm
for second stage prestressing		
Prestressing force (after friction and slip losses) , P	=	290.72 t
Moment due to P, $M_p = P \times e$	=	284.05 tm

Stresses due to self weight of the girder + cast in situ slab

$\sigma_j = \frac{542.13}{0.571} = 949.7$ t/m^2

$\sigma_c = \frac{542.13}{0.858} = -632.0$ t/m^2

$\sigma_b = \frac{542.13}{0.617} = -878.2$ t/m^2

Stresses due to prestress (second stage cables)

$\sigma_t = \frac{290.72}{1.906} - \frac{284.05}{1.581} = -27.1$ t/m^2

$\sigma_j = \frac{290.72}{1.906} - \frac{284.05}{2.262} = 26.9$ t/m^2

$\sigma_c = \frac{290.72}{1.906} + \frac{284.05}{1.136} = 402.6$ t/m^2

$\sigma_b = \frac{290.72}{1.906} + \frac{284.05}{0.865} = 481.0$ t/m^2

Total Stresses due to prestress

σ_t =	0.0	+	-27.1	=	-27.1	t/m^2
σ_j =	-240.1	+	26.9	=	-213.2	t/m^3
σ_c =	1142.2	+	402.6	=	1544.8	t/m^2
σ_b =	1357.4	+	481.0	=	1838.5	t/m^2

Stresses due to prestress & self weight of the girder+deck slab

σ_t =	0.0	+	-27.1	=	-27.1	t/m^2
σ_j =	949.7	+	-213.2	=	736.5	t/m^3
σ_c =	-632.0	+	1544.8	=	912.8	t/m^2
σ_b =	-878.2	+	1838.5	=	960.3	t/m^2

Losses due to Elastic Shortening

Loss due to Elastic Shortening = $0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss = 2467 t/m^2

(i.e. Elastic Shortening loss) = **20.46 t =** 91.03 % of Jacking Force

For first stage cables

Prestressing force after initial losses P_{ii} = 646.70 t

Moment due to $P_i = M_{pii} = P_{ii} \times e$ = 469.57 tm

For second stage cables

Prestressing force after initial losses P_i = 284.87 t

Moment due to $P_i = M_{pi} = P_i \times e$ = 278.34 tm

Loss due to Shrinkage of Concrete = $\varepsilon_s \times E_s$

= 3776.8 t/m^2

= **31.31 t =** 2.84 % of Jacking Force

Loss due to Shrinkage = 37.16 t

Moment due to Shrinkage loss = 42.972 tm

Loss due to relaxation of steel

For second stage cables:

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.381

Loss due to friction and slip = 314.5861 - 290.7 = 23.9 t

% friction and slip loss $\frac{23.9}{314.58612}$ x 100 = 7.588 %

Ratio of average force to UTS = 0.381 - 0.075879 = 0.306

Corresponding Relaxation Loss = 0.0 t/m^2

= **0.00 t** 0.00 % of Jacking Force

Loss due to Relaxation & Shrinkage = 5.84 t

Moment due to Relaxation & Shrinkage loss = 6.759 tm

Shrinkage losses (for first stage cables)

$\sigma_t = \frac{-37.16}{1.906} + \frac{42.972}{1.581} = 7.68$ t/m^2

$\sigma_j = \frac{-37.16}{1.906} + \frac{42.972}{2.262} = -0.49$ t/m^2

$\sigma_c = \frac{-37.16}{1.906} - \frac{42.972}{1.136} = -57.33$ t/m^2

$\sigma_b = \frac{-37.16}{1.906} - \frac{42.972}{0.865} = -69.20$ t/m^2

Shrinkage and Relaxation losses (for second stage cables)

$\sigma_t = \frac{-5.84}{1.906} + \frac{6.759}{1.581} = 1.21$ t/m^2

$\sigma_j = \frac{-5.84}{1.906} + \frac{6.759}{2.262} = -0.08$ t/m^2

$\sigma_c = \frac{-5.84}{1.906} - \frac{6.759}{1.136} = -9.02$ t/m^2

$\sigma_b = \frac{-5.84}{1.906} - \frac{6.759}{0.865} = -10.88$ t/m^2

Total Shrinkage and relaxation losses

σ_t =	7.68	+	1.21	=	8.89	t/m^2
σ_j =	-0.49	+	-0.08	=	-0.57	t/m^2
σ_c =	-57.33	+	-9.02	=	-66.34	t/m^2
σ_b =	-69.20	+	-10.88	=	-80.08	t/m^2

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab

σ_t =	-27.1	+	8.9	=	-18.24	t/m^2
σ_j =	736.5	+	-0.6	=	735.91	t/m^2
σ_c =	912.8	+	-66.3	=	846.48	t/m^2
σ_b =	960.3	+	-80.1	=	880.17	t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress after 28 days = 846.48 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 6598.4 t/m^2

= **54.71 t =** 4.969 % of Jacking Force

Loss due to Creep = 54.71 t

Moment due to Creep loss = 63.27 tm

Creep losses (for first stage cables)

$\sigma_t = \frac{-54.71}{1.906} + \frac{63.268}{1.581} = 11.31$ t/m^2

$\sigma_j = \frac{-54.71}{1.906} + \frac{63.268}{2.262} = -0.73$ t/m^2

$\sigma_c = \frac{-54.71}{1.906} - \frac{63.268}{1.136} = -84.40$ t/m^2

$\sigma_b = \frac{-54.71}{1.906} - \frac{63.268}{0.865} = -101.88$ t/m^2

Creep losses (for second stage cables)

$\sigma_t = \frac{-15.63}{1.906} + \frac{-18.077}{1.581} = -19.63$ t/m^2

$\sigma_j = \frac{-15.63}{1.906} + \frac{-18.077}{2.262} = -16.19$ t/m^2

$\sigma_c = \frac{-15.63}{1.906} - \frac{-18.077}{1.136} = 7.72$ t/m^2

$\sigma_b = \frac{-15.63}{1.906} - \frac{-18.077}{0.865} = 12.71$ t/m^2

Final Prestressing force = 809.47 t

Total Creep losses

σ_t =	11.31	+	-19.63	=	-8.32	t/m^2
σ_j =	-0.73	+	-16.19	=	-16.92	t/m^2
σ_c =	-84.40	+	7.72	=	-76.69	t/m^2
σ_b =	-101.88	+	12.71	=	-89.17	t/m^2

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

σ_t =	-18.24	+	-8.32	=	-26.56
σ_j =	735.91	+	-16.92	=	718.99
σ_c =	846.48	+	-76.69	=	769.79
σ_b =	880.17	+	-89.17	=	791.00

Stresses Due to SIDL

$\sigma_t = \frac{164.75}{1.581} = 104.2$ t/m^2

$\sigma_j = \frac{164.75}{2.262} = 72.8$ t/m^2

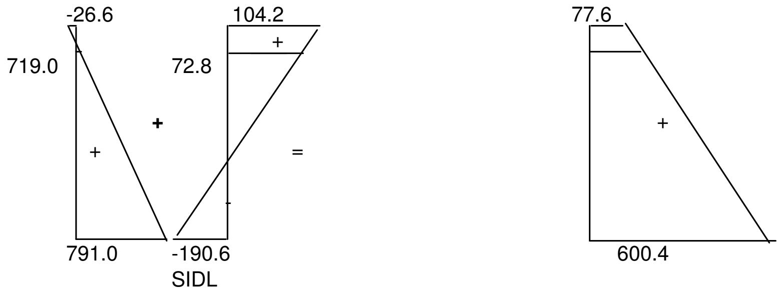
$\sigma_c = \frac{164.75}{1.136} = -145.1$ t/m^2

$\sigma_b = \frac{164.75}{0.865} = -190.6$ t/m^2

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

σ_t =	-26.6	+	104.19	=	77.6	t/m^2
σ_j =	719.0	+	72.83	=	791.8	t/m^2
σ_c =	769.8	+	-145.05	=	624.7	t/m^2
σ_b =	791.0	+	-190.56	=	600.4	t/m^2

After laying of SIDL



Stresses due to Live load

$\sigma_t = \frac{207.59}{1.581} = 131.28$ t/m^2

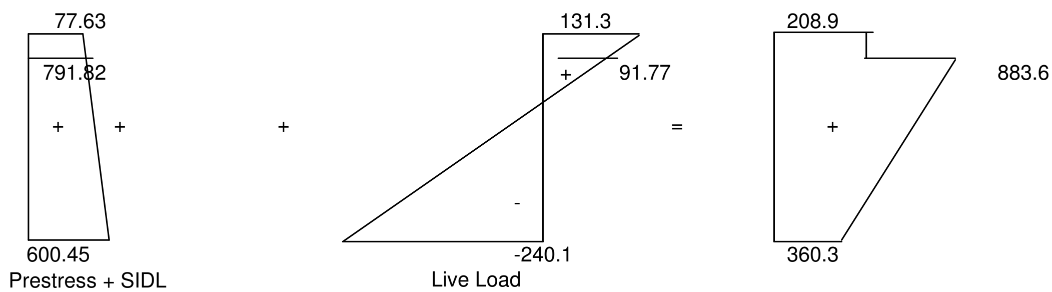
$\sigma j = \frac{207.59}{2.262} = 91.77$ t/m^2

$\sigma_c = \frac{207.59}{1.136} = -182.7717$ t/m^2

$\sigma_b = \frac{207.59}{0.865} = -240.11$ t/m^2

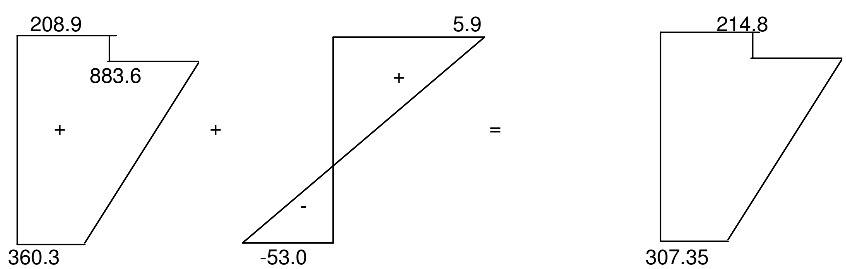
Stresses due to Prestress + DL + SIDL + FPLL + LL

σ_t =	77.63	+	131.3	=	208.9	t/m^2
σ_j =	791.82	+	91.8	=	883.6	t/m^2
σ_c =	624.74	+	-182.8	=	442.0	t/m^2
σ_b =	600.45	+	-240.1	=	360.3	t/m^2



Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

	=	142.28 t
20% extra time dependant losses	=	28.46 t
Stress due to 20% extra time dependant losses at top	=	**5.9 t/m2**
Stress due to 20% extra time dependant losses at bottom	=	**-53.0 t/m2**



STRESS CHECK AT 1-4 TH (SEC 3-3)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb	=	0.341	m
Eccentricity of cables from c.g., e	=	0.668	m
Sectional Modulus at C.G. of cables, Zc (for precast girder only)	=	0.932	m^3
B.M. due to S.Wt. of the girder M_d	=	238.50	tm
B.M.due to Cast in Situ slab M_c	=	242.95	tm
Prestressing force (after friction and slip losses) , P	=	709.01	t
Moment due to P, M_p = P x e	=	473.62	tm

Stresses due to self weight of the girder

$\sigma_t = \dfrac{238.50}{0.571} = 417.79$ t/m^2

$\sigma_c = \dfrac{238.50}{0.932} = -255.80$ t/m^2

$\sigma_b = \dfrac{238.50}{0.617} = -386.36$ t/m^2

Stresses due to prestress at transfer stage

$\sigma_t = \dfrac{709.01}{1.106} - \dfrac{473.62}{0.571} = -188.8$ t/m^2

$\sigma_c = \dfrac{709.01}{1.106} + \dfrac{473.62}{0.932} = 1148.88945$ t/m^2

$\sigma_b = \dfrac{709.01}{1.106} + \dfrac{473.62}{0.617} = 1408.2$ t/m^2

Stresses due to initial prestress & self weight of the girder

σ_t = 417.8 + -188.8 = 229.0 t/m^2

σ_c = -255.8 + 1148.9 = 893.1 t/m^2

σ_b = -386.4 + 1408.2 = 1021.8 t/m^2

<u>Losses due to Elastic Shortening</u>

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss = 2699 t/m^2

(i.e. Elastic Shortening loss) = **15.98 t =** 2.03 % of Jacking Force

Prestressing force after initial losses P_i = 693.03 t

Moment due to P_i = M_{pi} = P_i x e = 462.95 tm

<u>Loss due to Shrinkage of Concrete</u> = ε_s x E_s

= 1192.7 t/m^2

= **7.06 t =** 0.90 % of Jacking Force

Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.686

Loss due to friction & slip = 786.4653 - 709.0 = 77.5 t

% friction and slip loss $\frac{77.5}{786.46529}$ x 100 = 6.894 %

Ratio of average force to UTS = 0.686 - 0.068937 = 0.617

Corresponding Relaxation Loss = 5780.0 t/m^2
= **34.23 t** 4.35 % of Jacking Force

Loss due to Relaxation & Shrinkage = 41.29 t
Moment due to Relaxation & Shrinkage loss = 27.583 tm

Stresses due to Prestress after initial losses

$\sigma_t = \frac{693.03}{1.106} - \frac{462.95}{0.571} = -184.50$ t/m^2

$\sigma_c = \frac{693.03}{1.106} + \frac{462.95}{0.932} = 1123.0$ t/m^2

$\sigma_b = \frac{693.03}{1.106} + \frac{462.95}{0.617} = 1376.4$ t/m^2

Stresses due to prestress after initial losses & self weight of the girder

$\sigma_t = 417.8 + -184.5 = 233.3$ t/m^2

$\sigma_c = -255.8 + 1123.0 = 867.2$ t/m^2

$\sigma_b = -386.4 + 1376.4 = 990.1$ t/m^2

Stresses after Shrinkage and Relaxation losses

$\sigma_t = 233.29 - \frac{41.29}{1.106} + \frac{27.583}{0.571} = 244.29$ t/m^2

$\sigma_c = 867.2 - \frac{41.29}{1.106} - \frac{27.583}{0.932} = 800.28$ t/m^2

$\sigma_b = 990.06 - \frac{41.29}{1.106} - \frac{27.583}{0.617} = 908.05$ t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 833.74 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 1787.3 t/m^2
= **10.58 t =** 1.346 % of Jacking Force

Total Initial & Intermediate losses = **67.86 t =** 1.35 % of Jacking Force
(i.e. losses upto 28th day)

Force after intermediate losses, P_{int} = 641.2 t

Moment due to P_{int} = $M_{pint} = P_{int} \times e$ = 428.29 tm

Effect of prestress after Intermediate loss

$\sigma_t = \frac{641.2}{1.106} - \frac{428.29}{0.571} = -170.7$ t/m^2

$\sigma_c = \frac{641.2}{1.106} + \frac{428.29}{0.932} = 1038.9$ t/m^2

$\sigma_b = \frac{641.2}{1.106} + \frac{428.29}{0.617} = 1273.4$ t/m^2

Stresses Due to cast-in-situ slab

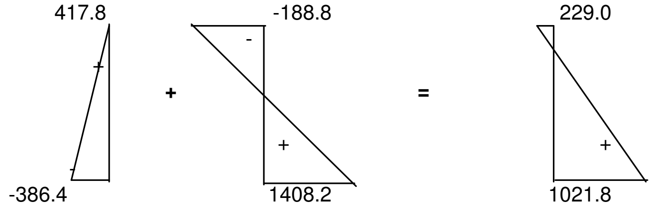
$\sigma_t = \frac{243.0}{0.571} = 425.59$ t/m^2

$\sigma_c = \frac{243.0}{0.932} = -260.57$ t/m^2

$\sigma_b = \frac{243.0}{0.617} = -393.56$ t/m^2

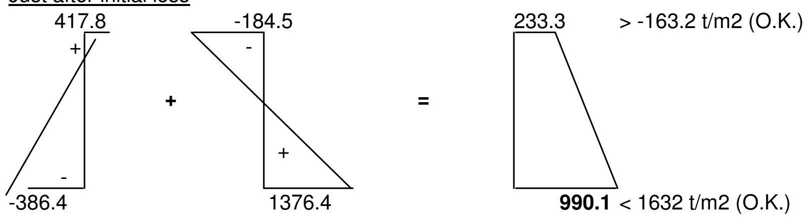
RESULTANT STRESSES

a) Transfer stage



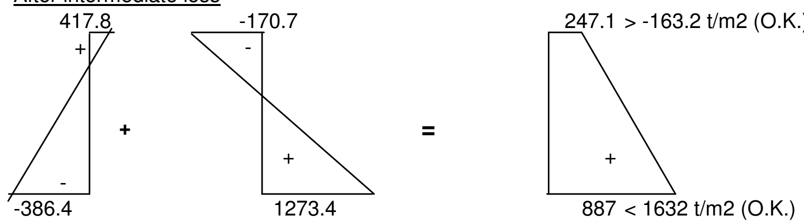
Self weight Jacking Stress

b) Just after initial loss



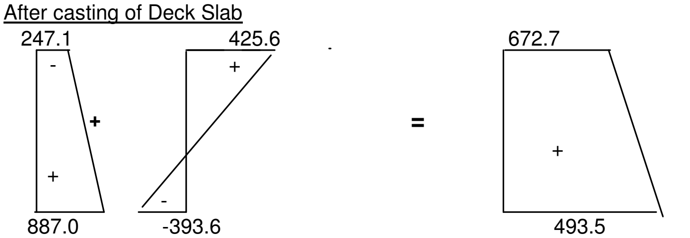
Self wt. prestress after initial loss Refer cl. 7.1. Of IRC : 18 - 2000

c) After intermediate loss



Self weight prestress after intermediate loss Net effect

d) After casting of Deck Slab



After Inter. Losse: Cast in situ slab > -163.2 t/m2 (O.K.)

STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section	=	1.58123097	m^3
Section Modulus at junction of deck slab & girder Z_j	=	2.2620	m^3
Section Modulus at C.G. of cables Zc	=	1.229	m^3
Section Modulus at bottom of composite section	=	0.8646	m^3
Composite area of deck slab + outer girder	=	1.9063	m^2
first stage cables			
Prestressing force (after 1st stage) , P_1	=	641.15	t
ecc. of cables from bottom, eb (Ist stage)	=	0.341	m
Eccentricity of cables from c.g., e (Ist stage)	=	0.668	m
second stage cables			
Prestressing force (after friction and slip losses) , P	=	283.21	t
ecc. of cables from bottom, eb (2nd stage)	=	0.699	m
Eccentricity of cables from c.g., e (2nd stage)	=	0.820	m

Net eccentricity of cables of 1st and 2nd stages, eb'	=	0.451 m	(from bottom)
Net eccentricity of cables of 1st and 2nd stages, e'	=	1.069 m	(from c.g.)
B.M. due to S.Wt. of the girder M_d	=	238.50 tm	
B.M.due to Cast in Situ slab M_c	=	186.99 tm	
B.M. due to SIDL M_s	=	126.54 tm	
B.M. due to Live loads M_L	=	175.11 tm	
B.M. due to FPLL	=	0.00 tm	
Prestressing force (after 1st stage) , P_1	=	641.15 t	
Moment due to P_{int} = $M_{pint} = P_1 \times e$	=	428.29 tm	
for second stage prestressing			
Prestressing force (after friction and slip losses) , P	=	283.21 t	
Moment due to P, $M_p = P \times e$	=	232.36 tm	

Stresses due to self weight of the girder + cast in situ slab

$\sigma_j = \dfrac{425.49}{0.571} = 745.3$ t/m^2

$\sigma_c = \dfrac{425.49}{0.932} = -456.3$ t/m^2

$\sigma_b = \dfrac{425.49}{0.617} = -689.3$ t/m^2

Stresses due to prestress (second stage cables)

$\sigma_t = \dfrac{283.21}{1.906} - \dfrac{232.36}{1.581} = 1.6$ t/m^2

$\sigma_j = \dfrac{283.21}{1.906} - \dfrac{232.36}{2.262} = 45.8$ t/m^2

$\sigma_c = \dfrac{283.21}{1.906} + \dfrac{232.36}{1.229} = 337.6$ t/m^2

$\sigma_b = \dfrac{283.21}{1.906} + \dfrac{232.36}{0.865} = 417.3$ t/m^2

Total Stresses due to prestress

σ_t =	0.0	+	1.6	=	1.6	t/m^2
σ_j =	-170.7	+	45.8	=	-124.8	t/m^3
σ_c =	1038.9	+	337.6	=	1376.5	t/m^2
σ_b =	1273.4	+	417.3	=	1690.7	t/m^2

Stresses due to prestress & self weight of the girder+deck slab

σ_t =	0.0	+	1.6	=	1.6	t/m^2
σ_j =	745.3	+	-124.8	=	620.5	t/m^3
σ_c =	-456.3	+	1376.5	=	920.2	t/m^2
σ_b =	-689.3	+	1690.7	=	1001.4	t/m^2

Losses due to Elastic Shortening

Loss due to Elastic Shortening = $0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss = 2487 t/m^2

(i.e. Elastic Shortening loss) = **20.62 t =** 1.87 % of Jacking Force

For first stage cables

Prestressing force after initial losses P_{ii} = 635.26 t

Moment due to $P_i = M_{pii} = P_{ii} \times e$ = 424.36 tm

For second stage cables

Prestressing force after initial losses P_i = 277.32 t

Moment due to $P_i = M_{pi} = P_i \times e$ = 227.53 tm

Loss due to Shrinkage of Concrete = $\varepsilon_s \times E_s$
= 3776.8 t/m^2
= **31.31 t =** 2.84 % of Jacking Force

Loss due to Shrinkage = 37.20 t
Moment due to Shrinkage loss = 39.760 tm

Loss due to relaxation of steel
For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.
Ratio of average force to UTS = 0.693
Loss due to friction and slip = 314.5861 - 283.2 = 31.4 t
% friction and slip loss $\frac{31.4}{314.58612} \times 100 = 6.98$ %
Ratio of average force to UTS = 0.693 - 0.069817 = 0.624

Corresponding Relaxation Loss = 6115.9 t/m^2
= **14.49 t** 4.61 % of Jacking Force

Loss due to Relaxation & Shrinkage = 20.38 t
Moment due to Relaxation & Shrinkage loss = 21.779 tm

Shrinkage losses (for first stage cables)

$\sigma_t = \frac{-37.20}{1.906} + \frac{39.760}{1.581} = 5.63$ t/m^2

$\sigma_j = \frac{-37.20}{1.906} + \frac{39.760}{2.262} = -1.94$ t/m^2

$\sigma_c = \frac{-37.20}{1.906} - \frac{39.760}{1.229} = -51.87$ t/m^2

$\sigma_b = \frac{-37.20}{1.906} - \frac{39.760}{0.865} = -65.50$ t/m^2

Shrinkage and Relaxation losses (for second stage cables)

$\sigma_t = \frac{-20.38}{1.906} + \frac{21.779}{1.581} = 3.08$ t/m^2

$\sigma_j = \frac{-20.38}{1.906} + \frac{21.779}{2.262} = -1.06$ t/m^2

$\sigma_c = \frac{-20.38}{1.906} - \frac{21.779}{1.229} = -28.41$ t/m^2

$\sigma_b = \frac{-20.38}{1.906} - \frac{21.779}{0.865} = -35.88$ t/m^2

Total Shrinkage and Relaxation losses

σ_t =	5.63	+	3.08	=	8.71	t/m^2
σ_j =	-1.94	+	-1.06	=	-3.00	t/m^2
σ_c =	-51.87	+	-28.41	=	-80.28	t/m^2
σ_b =	-65.50	+	-35.88	=	-101.39	t/m^2

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab

σ_t =	1.6	+	8.7	=	10.33	t/m^2
σ_j =	620.5	+	-3.0	=	617.50	t/m^2
σ_c =	920.2	+	-80.3	=	839.93	t/m^2
σ_b =	1001.4	+	-101.4	=	900.06	t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress after 28 days = 839.93 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 6547.4 t/m^2
= **54.28 t =** 4.930 % of Jacking Force

Loss due to Creep = 54.28 t
Moment due to Creep loss = 58.01 tm

Creep losses (for first stage cables)

$\sigma_t = \frac{-54.28}{1.906} + \frac{58.012}{1.581} = 8.21$ t/m^2

$\sigma_j = \frac{-54.28}{1.906} + \frac{58.012}{2.262} = -2.83$ t/m^2

$\sigma_c = \frac{-54.28}{1.906} - \frac{58.012}{1.229} = -75.67$ t/m^2

$\sigma_b = \frac{-54.28}{1.906} - \frac{58.012}{0.865} = -95.58$ t/m^2

Creep losses (for second stage cables)

$\sigma_t = \frac{-15.51}{1.906} + \frac{-16.575}{1.581} = -18.62$ t/m^2

$\sigma_j = \frac{-15.51}{1.906} + \frac{-16.575}{2.262} = -15.46$ t/m^2

$\sigma_c = \frac{-15.51}{1.906} - \frac{-16.575}{1.229} = 5.35$ t/m^2

$\sigma_b = \frac{-15.51}{1.906} - \frac{-16.575}{0.865} = 11.04$ t/m^2

Final Prestressing Force = 761.88 t

Total Creep losses

$\sigma_t = 8.21 + -18.62 = -10.41$ t/m^2
$\sigma_j = -2.83 + -15.46 = -18.29$ t/m^2
$\sigma_c = -75.67 + 5.35 = -70.33$ t/m^2
$\sigma_b = -95.58 + 11.04 = -84.54$ t/m^2

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

$\sigma_t = 10.33 + -10.41 = -0.08$
$\sigma_j = 617.50 + -18.29 = 599.21$
$\sigma_c = 839.93 + -70.33 = 769.60$
$\sigma_b = 900.06 + -84.54 = 815.52$

Stresses Due to SIDL

$\sigma_t = \frac{126.54}{1.581} = 80.0$ t/m^2

$\sigma_j = \frac{126.54}{2.262} = 55.9$ t/m^2

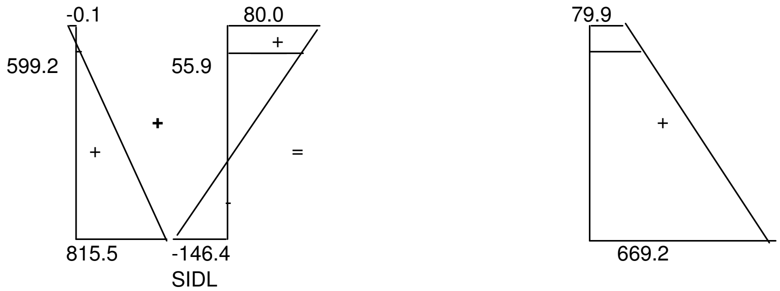
$\sigma_c = \frac{126.54}{1.229} = -103.0$ t/m^2

$\sigma_b = \frac{126.54}{0.865} = -146.4$ t/m^2

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

$\sigma_t = -0.1 + 80.03 = 79.9$ t/m^2
$\sigma_j = 599.2 + 55.94 = 655.2$ t/m^2
$\sigma_c = 769.6 + -102.95 = 666.6$ t/m^2
$\sigma_b = 815.5 + -146.36 = 669.2$ t/m^2

After laying of SIDL



Stresses due to Live load

$\sigma_t = \frac{175.11}{1.581} = 110.74$ t/m^2

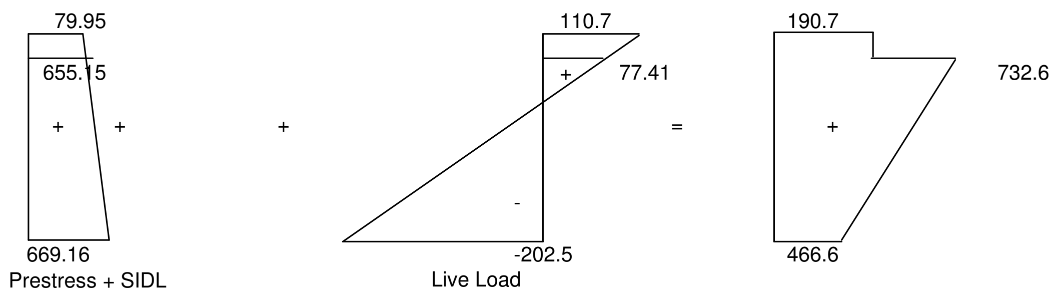
$\sigma_j = \frac{175.11}{2.262} = 77.41$ t/m^2

$\sigma_c = \frac{175.11}{1.229} = -142.4697$ t/m^2

$\sigma_b = \frac{175.11}{0.865} = -202.54$ t/m^2

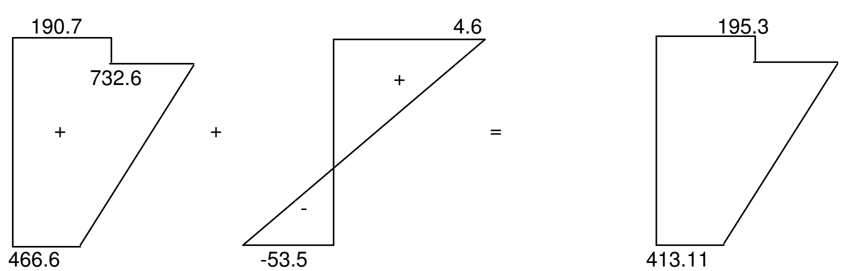
Stresses due to Prestress + DL + SIDL + FPLL + LL

σ_t =	79.95	+	110.7	=	190.7	t/m^2
σ_j =	655.15	+	77.4	=	732.6	t/m^2
σ_c =	666.65	+	-142.5	=	524.2	t/m^2
σ_b =	669.16	+	-202.5	=	466.6	t/m^2



Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

	=	151.96 t
20% extra time dependant losses	=	30.39 t
Stress due to 20% extra time dependant losses at top	=	**4.6 t/m2**
Stress due to 20% extra time dependant losses at bottom	=	**-53.5 t/m2**



STRESS CHECK AT 1-8 TH (SEC 4-4)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb	=	0.495	m
Eccentricity of cables from c.g., e	=	0.538	m
Sectional Modulus at C.G. of cables, Zc	=	1.219	m^3
(for precast girder only)			
B.M. due to S.Wt. of the girder M_d	=	141.29	tm
B.M.due to Cast in Situ slab M_c	=	138.33	tm
Prestressing force (after friction and slip losses) , P	=	689.33	t
Moment due to P, M_p = P x e	=	370.81	tm

Stresses due to self weight of the girder

$\sigma_t = \dfrac{141.29}{0.615} = 229.91$ t/m^2

$\sigma_c = \dfrac{141.29}{1.219} = -115.88$ t/m^2

$\sigma_b = \dfrac{141.29}{0.635} = -222.47$ t/m^2

Stresses due to prestress at transfer stage

$\sigma_t = \dfrac{689.33}{1.316} - \dfrac{370.81}{0.615} = -79.6$ t/m^2

$\sigma_c = \dfrac{689.33}{1.316} + \dfrac{370.81}{1.219} = 828.0$ t/m^2

$\sigma_b = \dfrac{689.33}{1.316} + \dfrac{370.81}{0.635} = 1107.7$ t/m^2

Stresses due to initial prestress & self weight of the girder

σ_t =	229.9	+	-79.6	=	150.3	t/m^2
σ_c =	-115.9	+	828.0	=	712.1	t/m^2
σ_b =	-222.5	+	1107.7	=	885.2	t/m^2

<u>Losses due to Elastic Shortening</u>

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss = 2152 t/m^2
(i.e. Elastic Shortening loss) = **12.74 t =** 1.62 % of Jacking Force

Prestressing force after initial losses P_i = 676.59 t
Moment due to P_i = M_{pi} = P_i x e = 363.96 tm

<u>Loss due to Shrinkage of Concrete</u> = ε_s x E_s
= 1192.7 t/m^2
= **7.06 t =** 0.90 % of Jacking Force

Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.689

Loss due to friction & slip = 786.4653 - 689.3 = 97.1 t

% friction and slip loss $\frac{97.1}{786.46529}$ x 100 = 8.645 %

Ratio of average force to UTS = 0.689 - 0.086454 = 0.602

Corresponding Relaxation Loss = 5056.0 t/m^2

= **29.94 t** 3.81 % of Jacking Force

Loss due to Relaxation & Shrinkage = 37.00 t

Moment due to Relaxation & Shrinkage loss = 19.906 tm

Stresses due to Prestress after initial losses

$\sigma_t = \frac{676.59}{1.316} - \frac{363.96}{0.615} = -78.09$ t/m^2

$\sigma_c = \frac{676.59}{1.316} + \frac{363.96}{1.219} = 812.7$ t/m^2

$\sigma_b = \frac{676.59}{1.316} + \frac{363.96}{0.635} = 1087.2$ t/m^2

Stresses due to prestress after initial losses & self weight of the girder

$\sigma_t = 229.9 + -78.1 = 151.8$ t/m^2

$\sigma_c = -115.9 + 812.7 = 696.8$ t/m^2

$\sigma_b = -222.5 + 1087.2 = 864.8$ t/m^2

Stresses after Shrinkage and Relaxation losses

$\sigma_t = 151.82 - \frac{37.00}{1.316} + \frac{19.906}{0.615} = 156.09$ t/m^2

$\sigma_c = 696.8 - \frac{37.00}{1.316} - \frac{19.906}{1.219} = 652.34$ t/m^2

$\sigma_b = 864.77 - \frac{37.00}{1.316} - \frac{19.906}{0.635} = 805.31$ t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 674.56 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 1446.0 t/m^2

= **8.56 t =** 1.089 % of Jacking Force

Total Initial & Intermediate losses = **58.31 t =** 1.09 % of Jacking Force

(i.e. losses upto 28th day)

Force after intermediate losses, P_{int} = 631.0 t

Moment due to P_{int} = $M_{pint} = P_{int} \times e$ = 339.45 tm

Effect of prestress after Intermediate loss

$\sigma_t = \frac{631.0}{1.316} - \frac{339.45}{0.615} = -72.8$ t/m^2

$\sigma_c = \frac{631.0}{1.316} + \frac{339.45}{1.219} = 757.9$ t/m^2

$\sigma_b = \frac{631.0}{1.316} + \frac{339.45}{0.635} = 1014.0$ t/m^2

Stresses Due to cast-in-situ slab

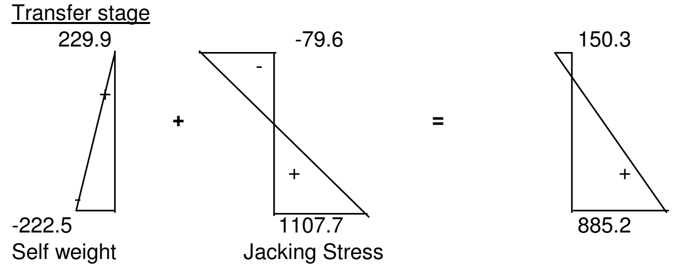
$\sigma_t = \frac{138.3}{0.615} = 225.10$ t/m^2

$\sigma_c = \frac{138.3}{1.219} = -113.45$ t/m^2

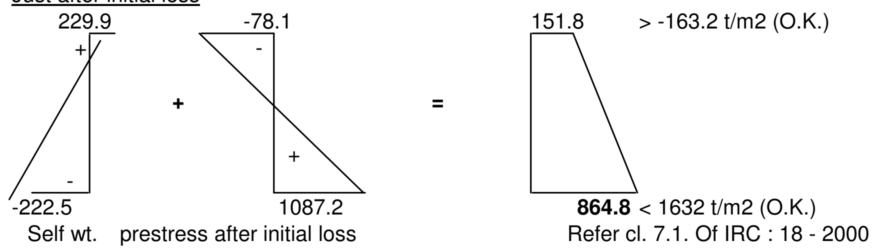
$\sigma_b = \frac{138.3}{0.635} = -217.81$ t/m^2

RESULTANT STRESSES

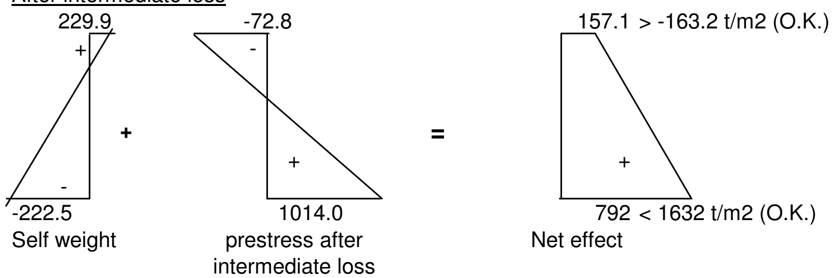
a) Transfer stage



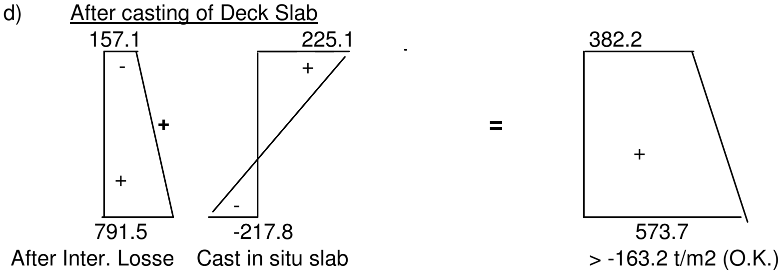
b) Just after initial loss



c) After intermediate loss



d) After casting of Deck Slab



STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section	=	1.44924207	m^3
Section Modulus at junction of deck slab & girder Z_j	=	2.0039	m^3
Section Modulus at C.G. of cables Zc	=	1.615	m^3
Section Modulus at bottom of composite section	=	0.9048	m^3
Composite area of deck slab + outer girder	=	2.0159	m^2
first stage cables			
Prestressing force (after 1st stage) , P_1	=	631.02	t
ecc. of cables from bottom, eb (Ist stage)	=	0.495	m
Eccentricity of cables from c.g., e (Ist stage)	=	0.514	m
second stage cables			
Prestressing force (after friction and slip losses) , P	=	275.32	t
ecc. of cables from bottom, eb (2nd stage)	=	0.960	m
Eccentricity of cables from c.g., e (2nd stage)	=	0.559	m

Net eccentricity of cables of 1st and 2nd stages, eb'	=	0.636	m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, e'	=	0.883	m (from c.g.)
B.M. due to S.Wt. of the girder M_d	=	141.29	tm
B.M.due to Cast in Situ slab M_c	=	82.37	tm
B.M. due to SIDL M_s	=	81.97	tm
B.M. due to Live loads M_L	=	89.10	tm
B.M. due to FPLL	=	0.00	tm
Prestressing force (after 1st stage) , P_1	=	631.02	t
Moment due to P_{int} = $M_{pint} = P_1 \times e$	=	324.45	tm
for second stage prestressing			
Prestressing force (after friction and slip losses) , P	=	275.32	t
Moment due to P, $M_p = P \times e$	=	154.03	tm

Stresses due to self weight of the girder + cast in situ slab

$\sigma_j = \frac{223.66}{0.615} = 364.0$ t/m^2

$\sigma_c = \frac{223.66}{1.219} = -183.4$ t/m^2

$\sigma_b = \frac{223.66}{0.635} = -352.2$ t/m^2

Stresses due to prestress (second stage cables)

$\sigma_t = \frac{275.32}{2.016} - \frac{154.03}{1.449} = 30.3$ t/m^2

$\sigma_j = \frac{275.32}{2.016} - \frac{154.03}{2.004} = 59.7$ t/m^2

$\sigma_c = \frac{275.32}{2.016} + \frac{154.03}{1.615} = 232.0$ t/m^2

$\sigma_b = \frac{275.32}{2.016} + \frac{154.03}{0.905} = 306.8$ t/m^2

Total Stresses due to prestress

σ_t =	0.0	+	30.3	=	30.3	t/m^2
σ_j =	-72.8	+	59.7	=	-13.1	t/m^3
σ_c =	757.9	+	232.0	=	989.9	t/m^2
σ_b =	1014.0	+	306.8	=	1320.8	t/m^2

Stresses due to prestress & self weight of the girder+deck slab

σ_t =	0.0	+	30.3	=	30.3	t/m^2
σ_j =	364.0	+	-13.1	=	350.8	t/m^3
σ_c =	-183.4	+	989.9	=	806.5	t/m^2
σ_b =	-352.2	+	1320.8	=	968.6	t/m^2

Losses due to Elastic Shortening

Loss due to Elastic Shortening = $0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss = 2180 t/m^2

(i.e. Elastic Shortening loss) = **18.07 t =** 1.64 % of Jacking Force

For first stage cables

Prestressing force after initial losses P_{ii} = 625.86 t

Moment due to P_i = $M_{pii} = P_{ii} \times e$ = 321.79 tm

For second stage cables

Prestressing force after initial losses P_i = 270.16 t

Moment due to P_i = $M_{pi} = P_i \times e$ = 151.14 tm

Loss due to Shrinkage of Concrete = ε_s x E_s
= 3776.8 t/m^2
= **31.31 t =** 2.84 % of Jacking Force

Loss due to Shrinkage = 36.48 t
Moment due to Shrinkage loss = 32.217 tm

Loss due to relaxation of steel
For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.
Ratio of average force to UTS = 0.694
Loss due to friction and slip = 314.5861 - 275.3 = 39.3 t
% friction and slip loss $\frac{39.3}{314.58612}$ x 100 = 8.74 %
Ratio of average force to UTS = 0.694 - 0.087369 = 0.607

Corresponding Relaxation Loss = 5287.7 t/m^2
= **12.53 t** 3.98 % of Jacking Force

Loss due to Relaxation & Shrinkage = 17.69 t
Moment due to Relaxation & Shrinkage loss = 15.623 tm

Shrinkage losses (for first stage cables)

$$\sigma_t = \frac{-36.48}{2.016} + \frac{32.217}{1.449} = 4.14 \quad t/m^2$$

$$\sigma_j = \frac{-36.48}{2.016} + \frac{32.217}{2.004} = -2.02 \quad t/m^2$$

$$\sigma_c = \frac{-36.48}{2.016} - \frac{32.217}{1.615} = -38.04 \quad t/m^2$$

$$\sigma_b = \frac{-36.48}{2.016} - \frac{32.217}{0.905} = -53.70 \quad t/m^2$$

Shrinkage and Relaxation losses (for second stage cables)

$$\sigma_t = \frac{-17.69}{2.016} + \frac{15.623}{1.449} = 2.01 \quad t/m^2$$

$$\sigma_j = \frac{-17.69}{2.016} + \frac{15.623}{2.004} = -0.98 \quad t/m^2$$

$$\sigma_c = \frac{-17.69}{2.016} - \frac{15.623}{1.615} = -18.45 \quad t/m^2$$

$$\sigma_b = \frac{-17.69}{2.016} - \frac{15.623}{0.905} = -26.04 \quad t/m^2$$

Total Shrinkage and Relaxation losses

σ_t =	4.14	+	2.01	=	6.14	t/m^2
σ_j =	-2.02	+	-0.98	=	-2.99	t/m^2
σ_c =	-38.04	+	-18.45	=	-56.49	t/m^2
σ_b =	-53.70	+	-26.04	=	-79.74	t/m^2

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab

σ_t =	30.3	+	6.1	=	36.43	t/m^2
σ_j =	350.8	+	-3.0	=	347.83	t/m^2
σ_c =	806.5	+	-56.5	=	749.96	t/m^2
σ_b =	968.6	+	-79.7	=	888.91	t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress after 28 days = 749.96 t/m^2

Creep loss = $(\sigma/10)$ x ε_c x E_s = 5846.1 t/m^2
= **48.47 t =** 4.402 % of Jacking Force

Loss due to Creep = 48.47 t
Moment due to Creep loss = 42.81 tm

Creep losses (for first stage cables)

$\sigma_t = \frac{-48.47}{2.016} + \frac{42.809}{1.449} = 5.50$ t/m^2

$\sigma_j = \frac{-48.47}{2.016} + \frac{42.809}{2.004} = -2.68$ t/m^2

$\sigma_c = \frac{-48.47}{2.016} - \frac{42.809}{1.615} = -50.55$ t/m^2

$\sigma_b = \frac{-48.47}{2.016} - \frac{42.809}{0.905} = -71.35$ t/m^2

Creep losses (for second stage cables)

$\sigma_t = \frac{-13.85}{2.016} + \frac{-12.231}{1.449} = -15.31$ t/m^2

$\sigma_j = \frac{-13.85}{2.016} + \frac{-12.231}{2.004} = -12.97$ t/m^2

$\sigma_c = \frac{-13.85}{2.016} - \frac{-12.231}{1.615} = 0.71$ t/m^2

$\sigma_b = \frac{-13.85}{2.016} - \frac{-12.231}{0.905} = 6.65$ t/m^2

Final Prestressing Loss = 759.26 t

Total Creep losses

$\sigma_t = 5.50 + -15.31 = -9.81$ t/m^2

$\sigma_j = -2.68 + -12.97 = -15.65$ t/m^2

$\sigma_c = -50.55 + 0.71 = -49.85$ t/m^2

$\sigma_b = -71.35 + 6.65 = -64.71$ t/m^2

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

$\sigma_t = 36.43 + -9.81 = 26.62$

$\sigma_j = 347.83 + -15.65 = 332.18$

$\sigma_c = 749.96 + -49.85 = 700.11$

$\sigma_b = 888.91 + -64.71 = 824.20$

Stresses Due to SIDL

$\sigma_t = \frac{81.97}{1.449} = 56.6$ t/m^2

$\sigma_j = \frac{81.97}{2.004} = 40.9$ t/m^2

$\sigma_c = \frac{81.97}{1.615} = -50.8$ t/m^2

$\sigma_b = \frac{81.97}{0.905} = -90.6$ t/m^2

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

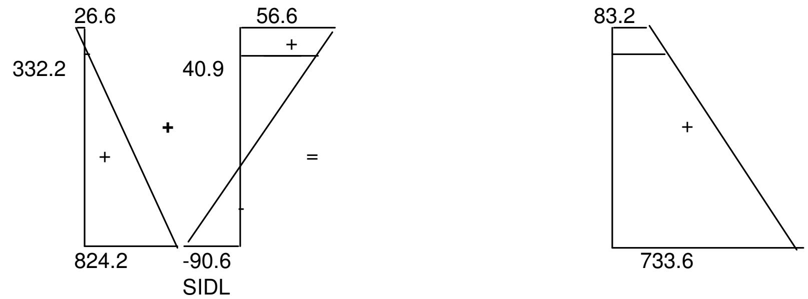
$\sigma_t = 26.6 + 56.56 = 83.2$ t/m^2

$\sigma_j = 332.2 + 40.91 = 373.1$ t/m^2

$\sigma_c = 700.1 + -50.76 = 649.4$ t/m^2

$\sigma_b = 824.2 + -90.59 = 733.6$ t/m^2

After laying of SIDL

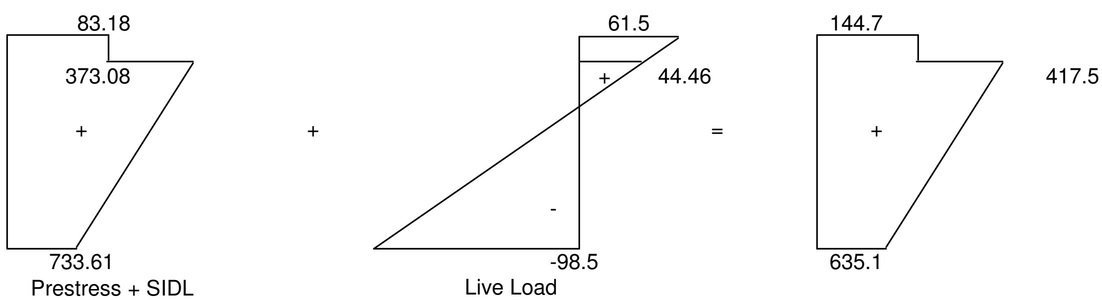


Stresses due to Live load

σ_t =	$\frac{89.10}{1.449}$	=	61.48	t/m^2
σj =	$\frac{89.10}{2.004}$	=	44.46	t/m^2
σ_c =	$\frac{89.10}{1.615}$	=	-55.1773	t/m^2
σ_b =	$\frac{89.10}{0.905}$	=	-98.47	t/m^2

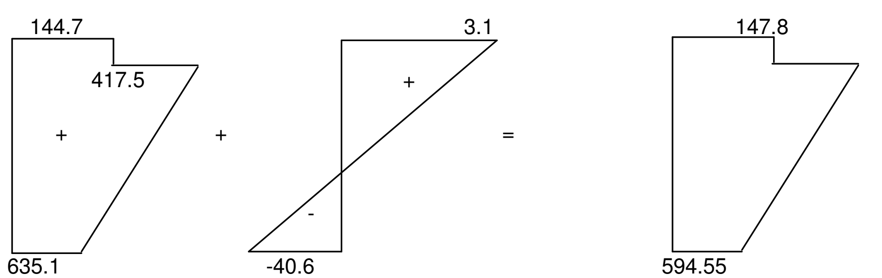
Stresses due to Prestress + DL + SIDL + FPLL + LL

σ_t =	83.18	+	61.5	=	144.7	t/m^2
σ_j =	373.08	+	44.5	=	417.5	t/m^2
σ_c =	649.35	+	-55.2	=	594.2	t/m^2
σ_b =	733.61	+	-98.5	=	635.1	t/m^2



Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

	=	137.87 t
20% extra time dependant losses	=	27.57 t
Stress due to 20% extra time dependant losses at top	=	**3.1 t/m2**
Stress due to 20% extra time dependant losses at bottom	=	**-40.6 t/m2**



STRESS CHECK AT EFF. DEPTH (SEC 5-5)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb	=	0.584	m
Eccentricity of cables from c.g., e	=	0.465	m
Sectional Modulus at C.G. of cables, Zc (for precast girder only)	=	1.491	m^3
B.M. due to S.Wt. of the girder M_d	=	92.26	tm
B.M.due to Cast in Situ slab M_c	=	88.01	tm
Prestressing force (after friction and slip losses) , P	=	681.31	t
Moment due to P, M_p = P x e	=	316.78	tm

Stresses due to self weight of the girder

$\sigma_t = \dfrac{92.26}{0.659} = 139.90$ t/m^2

$\sigma_c = \dfrac{92.26}{1.491} = -61.89$ t/m^2

$\sigma_b = \dfrac{92.26}{0.661} = -139.61$ t/m^2

Stresses due to prestress at transfer stage

$\sigma_t = \dfrac{681.31}{1.539} - \dfrac{316.78}{0.659} = -37.6$ t/m^2

$\sigma_c = \dfrac{681.31}{1.539} + \dfrac{316.78}{1.491} = 655.3$ t/m^2

$\sigma_b = \dfrac{681.31}{1.539} + \dfrac{316.78}{0.661} = 922.2$ t/m^2

Stresses due to initial prestress & self weight of the girder

$\sigma_t = 139.9 + -37.6 = 102.3$ t/m^2

$\sigma_c = -61.9 + 655.3 = 593.4$ t/m^2

$\sigma_b = -139.6 + 922.2 = 782.6$ t/m^2

<u>Losses due to Elastic Shortening</u>

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss = 1793 t/m^2

(i.e. Elastic Shortening loss) = **10.62 t =** 1.35 % of Jacking Force

Prestressing force after initial losses P_i = 670.69 t

Moment due to P_i = M_{pi} = P_i x e = 311.85 tm

<u>Loss due to Shrinkage of Concrete</u> = ε_s x E_s

= 1192.7 t/m^2

= **7.06 t =** 0.90 % of Jacking Force

Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.691

Loss due to friction & slip = 786.4653 - 681.3 = 105.2 t

% friction and slip loss $\frac{105.2}{786.46529}$ x 100 = 9.360 %

Ratio of average force to UTS = 0.691 - 0.093597 = 0.597

Corresponding Relaxation Loss = 4796.2 t/m^2
= **28.40 t** 3.61 % of Jacking Force

Loss due to Relaxation & Shrinkage = 35.47 t
Moment due to Relaxation & Shrinkage loss = 16.490 tm

Stresses due to Prestress after initial losses

$\sigma_t = \frac{670.69}{1.539} - \frac{311.85}{0.659} = -36.98$ t/m^2

$\sigma_c = \frac{670.69}{1.539} + \frac{311.85}{1.491} = 645.1$ t/m^2

$\sigma_b = \frac{670.69}{1.539} + \frac{311.85}{0.661} = 907.8$ t/m^2

Stresses due to prestress after initial losses & self weight of the girder

$\sigma_t = 139.9 + -37.0 = 102.9$ t/m^2

$\sigma_c = -61.9 + 645.1 = 583.2$ t/m^2

$\sigma_b = -139.6 + 907.8 = 768.2$ t/m^2

Stresses after Shrinkage and Relaxation losses

$\sigma_t = 102.92 - \frac{35.47}{1.539} + \frac{16.490}{0.659} = 104.88$ t/m^2

$\sigma_c = 583.2 - \frac{35.47}{1.539} - \frac{16.490}{1.491} = 549.08$ t/m^2

$\sigma_b = 768.18 - \frac{35.47}{1.539} - \frac{16.490}{0.661} = 720.18$ t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 566.13 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 1213.6 t/m^2
= **7.19 t =** 0.914 % of Jacking Force

Total Initial & Intermediate losses = **53.27 t =** 0.91 % of Jacking Force
(i.e. losses upto 28th day)

Force after intermediate losses, P_{int} = 628.0 t

Moment due to P_{int} = $M_{pint} = P_{int} \times e$ = 292.01 tm

Effect of prestress after Intermediate loss

$\sigma_t = \frac{628.0}{1.539} - \frac{292.01}{0.659} = -34.6$ t/m^2

$\sigma_c = \frac{628.0}{1.539} + \frac{292.01}{1.491} = 604.1$ t/m^2

$\sigma_b = \frac{628.0}{1.539} + \frac{292.01}{0.661} = 850.1$ t/m^2

Stresses Due to cast-in-situ slab

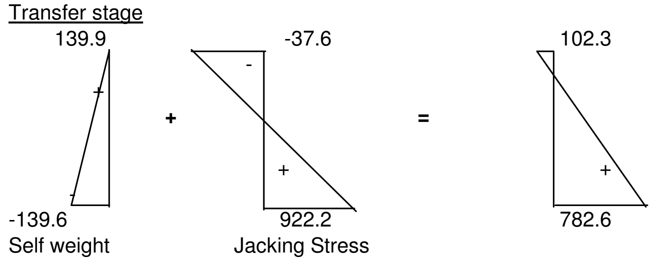
$\sigma_t = \frac{88.0}{0.659} = 133.45$ t/m^2

$\sigma_c = \frac{88.0}{1.491} = -59.04$ t/m^2

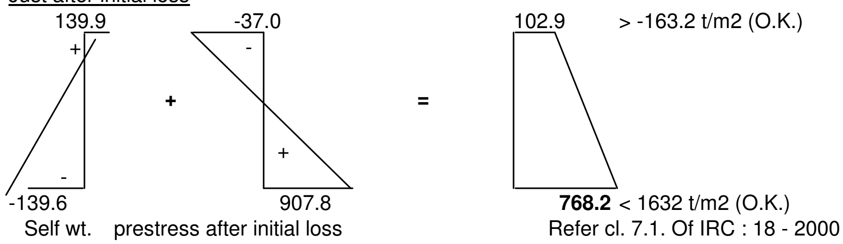
$\sigma_b = \frac{88.0}{0.661} = -133.18$ t/m^2

RESULTANT STRESSES

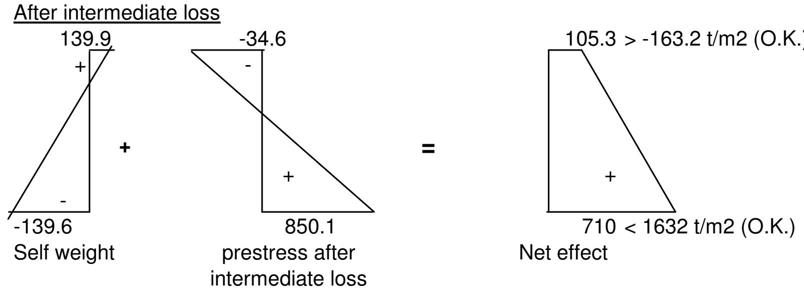
a) Transfer stage



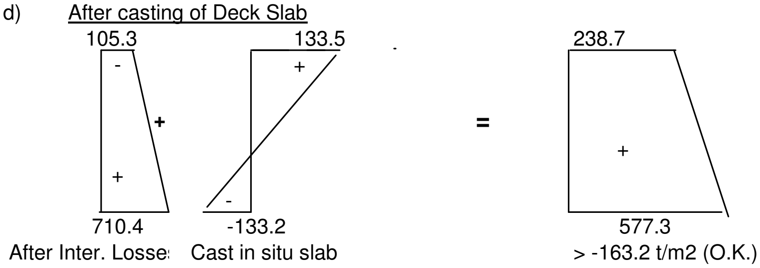
b) Just after initial loss



c) After intermediate loss



d) After casting of Deck Slab



STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section	=	1.58588333	m^3
Section Modulus at junction of deck slab & girder Z_j	=	2.1970	m^3
Section Modulus at C.G. of cables Zc	=	1.995	m^3
Section Modulus at bottom of composite section	=	0.9822	m^3
Composite area of deck slab + outer girder	=	2.3386	m^2
first stage cables			
Prestressing force (after 1st stage) , P_1	=	628.03	t
ecc. of cables from bottom, eb (Ist stage)	=	0.584	m
Eccentricity of cables from c.g., e (Ist stage)	=	0.425	m
second stage cables			
Prestressing force (after friction and slip losses) , P	=	272.14	t
ecc. of cables from bottom, eb (2nd stage)	=	1.090	m
Eccentricity of cables from c.g., e (2nd stage)	=	0.429	m
Net eccentricity of cables of 1st and 2nd stages, eb'	=	0.737	m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, e'	=	0.782	m (from c.g.)

B.M. due to S.Wt. of the girder M_d	=	92.26 tm
B.M.due to Cast in Situ slab M_c	=	32.05 tm
B.M. due to SIDL M_s	=	51.30 tm
B.M. due to Live loads M_L	=	54.36 tm
B.M. due to FPLL	=	0.00 tm

Prestressing force (after 1st stage) , P_1	=	628.03 t
Moment due to P_{int} = $M_{pint} = P_1 \times e$	=	266.92 tm
for second stage prestressing		
Prestressing force (after friction and slip losses) , P	=	272.14 t
Moment due to P, $M_p = P \times e$	=	116.85 tm

Stresses due to self weight of the girder + cast in situ slab

$\sigma_j = \frac{124.31}{0.659} = 188.5$ t/m^2

$\sigma_c = \frac{124.31}{1.491} = -83.4$ t/m^2

$\sigma_b = \frac{124.31}{0.661} = -188.1$ t/m^2

Stresses due to prestress (second stage cables)

$\sigma_t = \frac{272.14}{2.339} - \frac{116.85}{1.586} = 42.7$ t/m^2

$\sigma_j = \frac{272.14}{2.339} - \frac{116.85}{2.197} = 63.2$ t/m^2

$\sigma_c = \frac{272.14}{2.339} + \frac{116.85}{1.995} = 174.9$ t/m^2

$\sigma_b = \frac{272.14}{2.339} + \frac{116.85}{0.982} = 235.3$ t/m^2

Total Stresses due to prestress

σ_t =	0.0	+	42.7	=	42.7	t/m^2
σ_j =	-34.6	+	63.2	=	28.6	t/m^3
σ_c =	604.1	+	174.9	=	779.0	t/m^2
σ_b =	850.1	+	235.3	=	1085.4	t/m^2

Stresses due to prestress & self weight of the girder+deck slab

σ_t =	0.0	+	42.7	=	42.7	t/m^2
σ_j =	188.5	+	28.6	=	217.1	t/m^3
σ_c =	-83.4	+	779.0	=	695.6	t/m^2
σ_b =	-188.1	+	1085.4	=	897.3	t/m^2

Losses due to Elastic Shortening

Loss due to Elastic Shortening = 0.5 x σ x E_s/E_c

Therefore, initial loss	=	1880 t/m^2	
(i.e. Elastic Shortening loss)	=	**15.59 t =**	1.42 % of Jacking Force

For first stage cables

Prestressing force after initial losses P_{ii}	=	623.58 t
Moment due to $P_i = M_{pii} = P_{ii} \times e$	=	265.02 tm

For second stage cables

Prestressing force after initial losses P_i	=	267.69 t
Moment due to $P_i = M_{pi} = P_i \times e$	=	114.94 tm

Loss due to Shrinkage of Concrete	=	$\varepsilon_s \times E_s$	
	=	3776.8 t/m^2	
	=	**31.31 t =**	2.84 % of Jacking Force

Loss due to Shrinkage = 35.77 t
Moment due to Shrinkage loss = 27.983 tm

Loss due to relaxation of steel

For second stage cables:

The loss due to relaxation of steel depends on the stress in HTS after initial losses.
Ratio of average force to UTS = 0.695
Loss due to friction and slip = 314.5861 - 272.1 = 42.4 t
% friction and slip loss $\frac{42.4}{314.58612}$ x 100 = 9.44 %
Ratio of average force to UTS = 0.695 - 0.094441 = 0.601

Corresponding Relaxation Loss = 4976.9 t/m^2
= **11.79 t** 3.75 % of Jacking Force

Loss due to Relaxation & Shrinkage = 16.24 t
Moment due to Relaxation & Shrinkage loss = 12.708 tm

Shrinkage losses (for first stage cables)

$\sigma_t = \frac{-35.77}{2.339} + \frac{27.983}{1.586} = 2.35$ t/m^2

$\sigma_j = \frac{-35.77}{2.339} + \frac{27.983}{2.197} = -2.56$ t/m^2

$\sigma_c = \frac{-35.77}{2.339} - \frac{27.983}{1.995} = -29.32$ t/m^2

$\sigma_b = \frac{-35.77}{2.339} - \frac{27.983}{0.982} = -43.78$ t/m^2

Shrinkage and Relaxation losses (for second stage cables)

$\sigma_t = \frac{-16.24}{2.339} + \frac{12.708}{1.586} = 1.07$ t/m^2

$\sigma_j = \frac{-16.24}{2.339} + \frac{12.708}{2.197} = -1.16$ t/m^2

$\sigma_c = \frac{-16.24}{2.339} - \frac{12.708}{1.995} = -13.31$ t/m^2

$\sigma_b = \frac{-16.24}{2.339} - \frac{12.708}{0.982} = -19.88$ t/m^2

Total Shrinkage and Relaxation losses

σ_t =	2.35	+	1.07	=	3.42	t/m^2
σ_j =	-2.56	+	-1.16	=	-3.72	t/m^2
σ_c =	-29.32	+	-13.31	=	-42.63	t/m^2
σ_b =	-43.78	+	-19.88	=	-63.67	t/m^2

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab

σ_t =	42.7	+	3.4	=	46.10	t/m^2
σ_j =	217.1	+	-3.7	=	213.34	t/m^2
σ_c =	695.6	+	-42.6	=	652.96	t/m^2
σ_b =	897.3	+	-63.7	=	833.62	t/m^2

Loss due to Creep of concrete

Airthmetic mean of initial and final stress after 28 days = 652.96 t/m^2

Creep loss = $(\sigma/10) \times \varepsilon_c \times E_s$ = 5090.0 t/m^2
= **42.20 t =** 3.833 % of Jacking Force

Loss due to Creep = 42.20 t
Moment due to Creep loss = 33.02 tm

Creep losses (for first stage cables)

$\sigma_t = \frac{-42.20}{2.339} + \frac{33.016}{1.586} = 2.77$ t/m^2

$\sigma_j = \frac{-42.20}{2.339} + \frac{33.016}{2.197} = -3.02$ t/m^2

$\sigma_c = \frac{-42.20}{2.339} - \frac{33.016}{1.995} = -34.59$ t/m^2

$\sigma_b = \frac{-42.20}{2.339} - \frac{33.016}{0.982} = -51.66$ t/m^2

Creep losses (for second stage cables)

$\sigma_t = \frac{-12.06}{2.339} + \frac{-9.433}{1.586} = -11.10$ t/m^2

$\sigma_j = \frac{-12.06}{2.339} + \frac{-9.433}{2.197} = -9.45$ t/m^2

$\sigma_c = \frac{-12.06}{2.339} - \frac{-9.433}{1.995} = -0.43$ t/m^2

$\sigma_b = \frac{-12.06}{2.339} - \frac{-9.433}{0.982} = 4.45$ t/m^2

Final Prestressing Force = 766.54 t

Total Creep losses

$\sigma_t = 2.77 + -11.10 = -8.33$ t/m^2

$\sigma_j = -3.02 + -9.45 = -12.47$ t/m^2

$\sigma_c = -34.59 + -0.43 = -35.02$ t/m^2

$\sigma_b = -51.66 + 4.45 = -47.21$ t/m^2

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

$\sigma_t = 46.10 + -8.33 = 37.77$

$\sigma_j = 213.34 + -12.47 = 200.87$

$\sigma_c = 652.96 + -35.02 = 617.95$

$\sigma_b = 833.62 + -47.21 = 786.41$

Stresses Due to SIDL

$\sigma_t = \frac{51.30}{1.586} = 32.3$ t/m^2

$\sigma_j = \frac{51.30}{2.197} = 23.4$ t/m^2

$\sigma_c = \frac{51.30}{1.995} = -25.7$ t/m^2

$\sigma_b = \frac{51.30}{0.982} = -52.2$ t/m^2

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

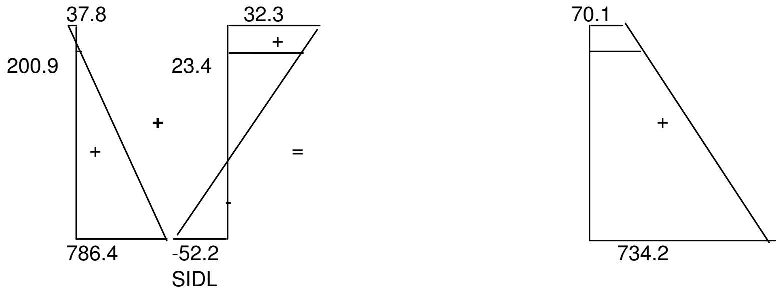
$\sigma_t = 37.8 + 32.35 = 70.1$ t/m^2

$\sigma_j = 200.9 + 23.35 = 224.2$ t/m^2

$\sigma_c = 617.9 + -25.71 = 592.2$ t/m^2

$\sigma_b = 786.4 + -52.23 = 734.2$ t/m^2

After laying of SIDL

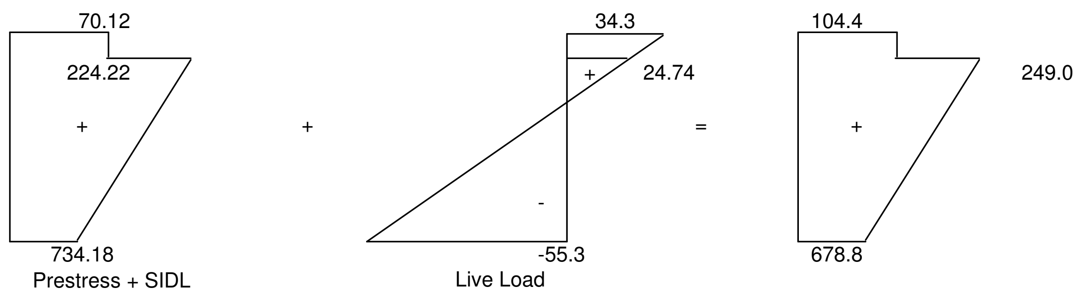


Stresses due to Live load

σ_t =	$\frac{54.36}{1.586}$	=	34.28	t/m^2
σj =	$\frac{54.36}{2.197}$	=	24.74	t/m^2
σ_c =	$\frac{54.36}{1.995}$	=	-27.24261	t/m^2
σ_b =	$\frac{54.36}{0.982}$	=	-55.35	t/m^2

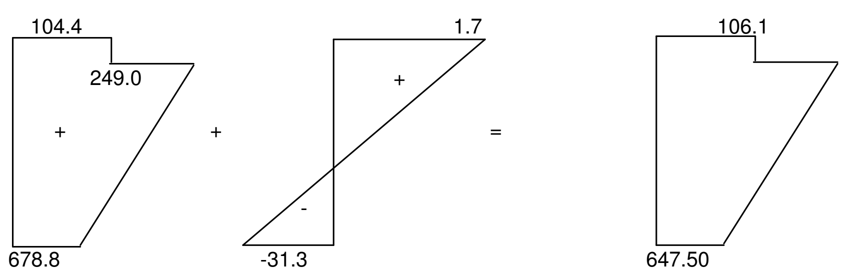
Stresses due to Prestress + DL + SIDL + FPLL + LL

σ_t =	70.12	+	34.3	=	104.4	t/m^2
σ_j =	224.22	+	24.7	=	249.0	t/m^2
σ_c =	592.24	+	-27.2	=	565.0	t/m^2
σ_b =	734.18	+	-55.3	=	678.8	t/m^2



Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

	=	127.95 t
20% extra time dependant losses	=	25.59 t
Stress due to 20% extra time dependant losses at top	=	**1.7 t/m2**
Stress due to 20% extra time dependant losses at bottom	=	**-31.3 t/m2**



SUMMARY OF MAXIMUM BENDING MOMENT & SHEAR FORCE

GIRDER G1 :

a) BENDING MOMENT (tm)

LOADING	SECTIONS					
	1-1	2-2	3-3	3-3	4-4	5-5
a) Self Weight of Girder	316.27	296.83	238.50	238.50	141.29	92.26
b) Deck Slab	268.46	248.82	201.05	201.05	114.01	72.45
c) SIDL	163.30	164.75	126.54	126.54	81.97	51.30
d)FPLL	0.00	0.00	0.00	0.00	0.00	0.0
e) LL	238.92	207.59	175.11	175.11	89.10	54.36
f) TOTAL	986.95	917.99	741.20	741.20	426.37	270.37
g) LL BM corresponding to Max S.F.case	198.15	195.41	170.34	170.34	88.84	54.05

b) SHEAR FORCE (t)

LOADING	SECTIONS					
	1-1	2-2	3-3	3-3	4-4	5-5
a) Self Weight of Girder	0.00	10.37	20.74	20.74	31.11	36.06
b) Deck Slab	1.49	8.99	19.46	19.46	26.96	29.88
c) SIDL	2.00	8.60	10.30	10.30	20.35	21.61
d)FPLL	0.00	0.00	0.00	0.00	0.00	0.00
e) LL	12.22	13.70	22.64	22.64	23.95	24.46
f) TOTAL	15.71	41.66	73.14	73.14	102.37	112.01
g) LL Shear corresponding to max BM case	7.65	8.94	21.94	21.94	23.89	23.76

ULTIMATE BENDING MOMENT AND SHEAR FORCE

a) Maximum Bending Moment & corresspondiing Shear Force

	Section	1-1	2-2	2-2	3-3	4-4	5-5
Bending Moment (tm)	1.25DL	730.9	682.1	682.1	549.4	319.1	205.9
	2SIDL	326.6	329.5	329.5	253.1	163.9	102.6
	2.5LL	597.3	519.0	519.0	437.8	222.8	135.9
	TOTAL M_u	1654.8	1530.5	1530.5	1240.3	705.8	444.4
Shear Force corresponding to Max Bending Moment (t)	1.25DL	1.9	24.2	24.2	50.3	72.6	82.4
	2SIDL	4.0	17.2	17.2	20.6	40.7	43.2
	2.5LL	19.1	22.4	54.9	54.9	59.7	59.4
	TOTAL V_u	25.0	63.8	96.3	125.7	173.0	185.0

b) Maximum Shear force and Corressponding Bending Moment

	Section	1-1	2-2	2-2	3-3	4-4	5-5
SHEAR FORCE (t)	1.25DL	1.9	24.2	24.2	50.3	72.6	82.4
	2SIDL	4.00	17.20	17.20	20.60	40.70	43.22
	2.5LL	30.6	34.3	34.3	56.6	59.9	61.2
	TOTAL Vu	36.4	75.7	75.7	127.5	173.2	186.8
BENDING MOMENTS CORRESSPONDING TO MAX SHEAR (tm)	1.25DL	730.9	682.1	682.1	549.4	319.1	205.9
	2SIDL	326.6	329.5	329.5	253.1	163.9	102.6
	2.5LL	495.4	488.5	488.5	425.9	222.1	135.1
	TOTAL Mu	1552.9	1500.1	1500.1	1228.4	705.2	443.6

SECTION PROPERTIES

Section		1-1	2-2	2-2	3-3	4-4	5-5
Sectional area, A	m^2	1.906	1.906	1.906	1.906	2.016	2.239
Moment of Inertia, Iz	m^4	1.314	1.314	1.314	1.314	1.309	1.362
C.G. of section from top,Yt	m	0.831	0.831	0.831	0.831	0.903	0.933
C.G. of section from bottom, Yb	m	1.519	1.519	1.519	1.519	1.447	1.417
Sectional modulus at top, Zt	m^3	1.581	1.581	1.581	1.581	1.449	1.460
Sectional modulus at bottom, Zb	m^3	0.865	0.865	0.865	0.865	0.905	0.962
Width of the web, b	m	0.275	0.275	0.275	0.275	0.438	0.6
Overall Depth of Section, d	m	2.35					

	Section	1-1	2-2	2-2	3-3	4-4	5-5
Total No. of cables		7	7	7	7	7	7
Final prestressing force, P_f	t	775.57	809.47	809.47	761.88	759.26	766.54
Eccentricity from bottom, e_b	m	0.332	0.363	0.36279805	0.451	0.636	0.737
d_b = D - e_b	m	2.018	1.987	1.987	1.899	1.714	1.613

Average thickness of compression flange, t = 0.347 m B_f = 3200 mm

ULTIMATE MOMENT CAPACITY

As per Clause 13 of IRC:18 - 2000		1-1	2-2	2-2	3-3	4-4	
M_{ults}* = 0.9 db A_s f_p		2856.2	2813.2	2813.2	2688.9	2426.3	2283.5
M_{ultc}** = 0.176 b d_b^2 f_{ck} +	Part1	803.8	779.8	779.8	712.4	922.9	1121.1
2/3 x 0.8 (B_f - b)(d_b - t/2) x t x f_{ck}	Part2	4069.8	4002.8	4002.8	3809.0	3210.9	2824.2
	Total	4873.6	4782.6	4782.6	4521.4	4133.7	3945.2
M_{ult} = min(M_{ults} and M_{ultc})		2856.2	2813.2	2813.2	2688.9	2426.3	2283.5
M_u		1654.8	1530.5	1530.5	1240.3	705.8	444.4
Remarks		**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**

* M_{ults} Moment capacity due to yield of steel.

** M_{ultc} Moment capacity due to crushing of concrete.

CALCULATION OF DEPTH OF TENSILE ZONE

Section		1-1	2-2	2-2	3-3	4-4	5-5
σ_t = 0.8 x (P_f/A - M_f/Z_t) + M_{ult}/Z_t	t/m^2	1666.1	1645.2	1645.2	1608.3	1635.7	1552.9
σ_b = 0.8 x (P_f/A+M_f/Z_b)- M_{ult}/Z_b	t/m^2	-2126.3	-2047.9	-2047.9	-2036.9	-1836.0	-1667.3
Depth of tensile zone, z	m	1.318	1.303	1.303	1.313	1.243	1.217
C.G. of cables in tensile zone from bottom, eb	m	0.332	0.363	0.363	0.451	0.636	0.737
d_b = (D - e_{max}), but not < 0.8D	m	2.018	1.987	1.987	1.899	1.880	1.880

SHEAR CAPACITY AND SHEAR REINFORCEMENT

f_t = Maximum principal tensile stress

= 0.24√fck = 154.83 t/m^2 (As per Clause 14.1.2 of IRC:18)

fcp = Stress due to prestress only at centroidal axis of composite section

fpt = Stress due to prestress only at tensile fibre of composite section

dt = Depth from extreme compression fibre either to the longitudinal bars or to the centroid of the tendons whichever is greater)

= d - 0.060 = 2.290 m

Maximum Ultimate shear stress = **479.4** t/m^2 (As per Table 6 of IRC:18)

	units	1-1	2-2	2-2	3-3	4-4	5-5
b = web thickness = 275 - 75 x 2/3	m	0.225	0.225	0.225	0.225	0.388	0.550
f_{cp}	t/m^2	406.86	424.64	424.64	399.68	376.64	342.41
$V_{co}^* = 0.67bd\sqrt{(f_t^2 + 0.8f_{cp}f_t)}$	t	96.6	98.0	98.0	96.0	162.1	223.1
$d_b = d - e_b$	m	2.018	1.987	1.987	1.899	1.880	1.880
f_{pt}	t/m^2	1471.6	1507.4	1507.4	1341.4	1056.9	884.3
$M_t = (0.37\sqrt{fck} + 0.8f_{pt})I/y$	tm	1224.2	1249.0	1249.0	1134.2	981.0	909.8
$V_{cr} = 0.037bd_b\sqrt{fck} + (M_t/Mu)$	t	39.5	73.7	73.7	127.9	258.3	407.8
$V_{crmin} = 0.1\ bd\sqrt{fck}$	t	34.1	34.1	34.1	34.1	58.7	83.4
V_{cr} = max (V_{cr}, V_{crmin})	t	39.5	73.7	73.7	127.9	258.3	407.8
V_c = min (V_{co},V_{cr})	t	39.5	73.7	73.7	96.0	162.1	223.1
V_u	t	36.4	75.7	75.7	127.5	173.2	186.8
Remarks		Vc>Vu>Vc/2	Vu>Vc	Vu>Vc	Vu>Vc	Vu>Vc	Vc>Vu>Vc/2
$(A_{sv}/S_v)_{min} = 0.4b/(0.87f_{yv})$	(cm^2/m)	2.493	2.493	2.493	2.493	4.293	6.093
$A_{sv}/S_v = (V-V_c)/(0.87f_{yv}d_t)$	(cm^2/m)	0.00	0.24	0.24	3.80	1.33	0.00
Provide A_{sv}/S_v =	(cm^2/m)	2.49	2.49	2.49	3.80	4.29	6.09
$v_u = Vu/bd_b$ where, b = 225mm	t/m^2	80.2	169.2	169.2	298.2	237.7	180.7
Remarks		**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**

V_{co} Uncracked shear capacity of the section.

V_{cr} Cracked shear capacity

V_c Shear capacity

V_u Ultimate shear

Shear reinforcement

f_{yv} Yield strength of shear reinforcement

v_u Shear stress

ULTIMATE T0RSION & SHEAR

GIRDER G_1

CASE 1 MAX. TORSIONAL MOMENT (t.m) & CORR. SHEAR FORCE(t)

	MAXIMUM TORSIONAL MOMENT						2.5 (LL*I.F.		CORRESPONDING SHEAR FORCE						2.5 (LL*I.F.	
SEC.	DL	1.25 DL	SIDL	2.0 SIDL	LL	FPLL	+ FPLL)	ULT	DL	1.25 DL	SIDL	2.0 SIDL	LL	FPLL	+ FPLL)	ULT
1-1	0.00	0.00	2.88	5.76	0.54	0.00	1.36	7.12	1.49	1.86	2.00	4.00	4.20	0.00	10.50	16.36
2- 2	0.00	0.00	3.10	6.20	0.65	0.00	1.62	7.82	19.36	24.20	8.60	17.20	2.77	0.00	6.93	48.33
3- 3	0.00	0.00	1.74	3.48	2.02	0.00	5.04	8.52	40.20	50.25	10.30	20.60	19.88	0.00	49.70	120.55
4- 4	0.00	0.00	5.73	11.46	2.81	0.00	7.03	18.49	58.07	72.59	20.35	40.70	19.38	0.00	48.45	161.74
5- 5	0.00	0.00	5.73	11.46	2.81	0.00	7.03	18.49	65.94	82.43	21.61	43.22	19.38	0.00	48.45	174.10

CASE 2 MAXIMUM SHEAR FORCE (t) & CORR. TORSIONAL MOMENT(t.m)

	MAXIMUM SHEAR FORCE						2.5 (LL*I.F.		CORRESPONDING TORSIONAL MOMENT						2.5 (LL*I.F.	
SEC.	DL	1.25 DL	SIDL	2.0 SIDL	LL	FPLL	+ FPLL)	ULT	DL	1.25 DL	SIDL	2.0 SIDL	LL	FPLL	+ FPLL)	ULT
1-1	1.49	1.86	2.00	4.00	11.53	0.00	28.83	34.69	0.00	0.00	2.88	5.76	0.09	0.00	0.23	5.99
2- 2	19.36	24.20	8.60	17.20	12.16	0.00	30.40	71.80	0.00	0.00	3.10	6.20	0.28	0.00	0.70	6.90
3- 3	40.20	50.25	10.30	20.60	20.29	0.00	50.73	121.58	0.00	0.00	1.74	3.48	0.69	0.00	1.73	5.21
4- 4	58.07	72.59	20.35	40.70	20.50	0.00	51.25	164.54	0.00	0.00	5.73	11.46	1.08	0.00	2.70	14.16
5- 5	65.94	82.43	21.61	43.22	20.95	0.00	52.38	178.02	0.00	0.00	5.73	11.46	1.17	0.00	2.93	14.39

CHECK FOR TOTAL ULTIMATE SHEAR STRESS

Each component Rectangle will be subjected to a Torque = T(hmax * hmin^3)/(Σhmax *hmin^3)=

= T x C (say)

Component Rectangle	hmax	hmin	hmax * hmin ^3	C
1	3200	250	5.00E+10	0.30
2	1250	230	1.52E+10	0.09
3	1450	275	3.02E+10	0.18
4	1000	420	7.41E+10	0.44
		Σ	1.69E+11	

Torsional Shear Stress= 2 T / (hmin ^2)/(hmax-hmin/3)

CASE 1 MAX. TORSIONAL MOMENT & CORR. SHEAR FORCE

SEC.	MAX. TOR. MOM.(t.m)	MOMENT IN RECTANGULAR COMPONENT (t.m)				TOR. SHEAR STRESS IN COMPONENT RECTANGLES OF GIRDER Vtu (t/m^2)				CORR. S.F. (t)	ULTIMATE SHEAR STRESS DUE TO S.F. $\frac{V}{b\,d_b}$	TOTAL ULTIMATE SHEAR STRESS V + Vtu (t/m^2)
		1	2	3	4	1	2	3	4			
1-1	7.12	2.10	0.64	1.27	3.11	21.6	20.6	24.7	41.0	16.36	36.0	61
2-2	7.82	2.31	0.70	1.39	3.42	23.7	22.6	27.1	45.0	48.33	108.1	135
3-3	8.52	2.51	0.76	1.52	3.73	25.8	24.6	29.5	49.1	120.55	163.8	193
4-4	18.49	5.45	1.66	3.29	8.08	56.0	53.5	64.0	106.5	161.74	156.4	220
5-5	18.49	5.45	1.66	3.29	8.08	56.0	53.5	64.0	106.5	174.10	168.4	232

PERMISSIBLE VALUE OF TOTAL ULT. SHEAR STR 484.5 t/m^2

	COMPONENT RECTANGLES			
	1	2	3	4
Hmax	3.2	1.25	1.45	1
Hmin	0.25	0.23	0.275	0.42

CASE 2 MAX. SHEAR FORCE & CORR. TORSIONAL MOMENT

SEC	CORR. TOR. MOM.(t.m)	MOMENT IN COMPONENT RECTANGLES (t.m])				TOR. SHEAR STRESS IN COMPONENT RECTANGLES OF GIRDER Vtu (t/m^2)				MAX. S.F. (t)	ULTIMATE SHEAR STRESS DUE TO S.F. V (t/m^2)	TOTAL ULTIMATE SHEAR STRESS V + Vtu (t/m^2)
		1	2	3	4	1	2	3	4			
1-1	5.99	1.77	0.54	1.07	2.62	18.13	17.31	20.74	34.50	34.69	76.4	97
2-2	6.90	2.04	0.62	1.23	3.02	20.90	19.95	23.91	39.77	71.80	160.6	184
3-3	5.21	1.54	0.47	0.93	2.28	15.77	15.05	18.03	30.00	121.58	165.2	183
4-4	14.16	4.18	1.27	2.52	6.19	42.90	40.95	49.06	81.62	164.54	159.1	208
5-5	14.39	4.24	1.29	2.56	6.29	43.58	41.60	49.84	82.92	178.02	172.2	222

CALCULATION OF TORSION REINFORCEMENT

AS PER CLAUSE 14.2.3.1 OF IRC : 18 - 2000

REINFORCEMENT FOR TORSION Asv/Sv >= T / (0.8 * X1 * Y1 * 0.87 * Fyv)

AREA OR LONG. REINFORCEMENT Asl >= Asv/Sv * (X1 + Y1) * (Fyv / Fyl)

Fyv (t/m^2) = 42330 X1 is the smaller dimension of the link measured between centres of legs

Y1 is the larger dimension of the link measured between centres of legs

According to clause 14.2.2 of IRC-18:2000 , torsion reinforcement is not required if Vtu < 42.75 t/m^2

	COMPONENT RECTANGLES			
	1	2	3	4
Hmax	3.2	1.25	1.45	1
Hmin	0.25	0.23	0.275	0.42

CASE 1 MAX. TORSIONAL MOMENT & CORR. SHEAR FORCE

SEC	MAX. TOR. MOM.(t)	MOMENT IN COMPONENT RECTANGLES (t.m)				TORSIONAL REINFORCEMENT Asv / Sv (cm^2/m)				AREA OF LONG. REINFORCEMENT Asv / Sv (cm^2)			
		1	2	3	4	1	2	3	4	1	2	3	4
1-1	7.12	2.10	0.64	1.27	3.11	N.R.	N.R.	N.R.	NOT REQ.	-	-	-	-
2-2	7.82	2.31	0.70	1.39	3.42	NOT REQ.	IOT REQ	NOT REQ	3.9	-	-	-	4.81
3-3	8.52	2.51	0.76	1.52	3.73	NOT REQ.	IOT REQ	NOT REQ	4.2	-	-	-	5.25
4-4	18.49	5.45	1.66	3.29	8.08	3.8	3.5	4.5	9.2	12.31	4.57	6.92	11.38
5-5	18.49	5.45	1.66	3.29	8.08	3.8	3.5	4.5	9.2	12.31	4.57	6.92	11.38

CASE 2 MAX. SHEAR FORCE & CORR. TORSIONAL MOMENT

SEC	TOR. MOM.(t)	MOMENT IN COMPONENT RECTANGLES (t.m)				TORSIONAL REINFORCEMENT Asv / Sv (cm^2/m)				AREA OF LONG. REINFORCEMENT Asv / Sv (cm^2)			
		1	2	3	4	1	2	3	4	1	2	3	4
1-1	5.99	1.77	0.54	1.07	2.62	NOT REQ.	IOT REQ	NOT REQ	NOT REQ.	-	-	-	-
2-2	6.90	2.04	0.62	1.23	3.02	NOT REQ.	IOT REQ	NOT REQ	NOT REQ.	-	-	-	-
3-3	5.21	1.54	0.47	0.93	2.28	NOT REQ.	IOT REQ	NOT REQ	NOT REQ.	-	-	-	-
4-4	14.16	4.18	1.27	2.52	6.19	2.9	IOT REQ	3.4	7.1	9.43	-	5.30	8.72
5-5	14.39	4.24	1.29	2.56	6.29	2.9	IOT REQ	3.5	7.2	9.58	-	5.39	8.86

TOTAL R/F IN WEB

SEC	CASE 1			CASE 2		
	SHEAR	TORSION	TOTAL	SHEAR	TORSION	TOTAL
	cm^2/m	cm^2/m	cm^2/m	cm^2/m	cm^2/m	cm^2/m
1-1	2.49	0.00	2.49	2.49	FALSE	2.49
2-2	3.80	0.00	3.80	3.80	FALSE	3.80
3-3	4.29	NOT REQ.	4.29	4.29	FALSE	4.29
4-4	6.09	4.49	6.09	6.09	3.44	9.54
5-5	6.09	4.49	10.59	6.09	3.50	9.59

DESIGN OF SHEAR CONNECTOR

Reference : clause 611.4.2 of IRC : 22 - 1986 code for composite construction
The ultimate longitudinal shear V_L per unit length ,

$$V_L = \frac{V \cdot A_c \cdot \overline{Y}}{I}$$

Where ,

V = Vertical shear due to dead load placed after composite section is effective and working live load with impact.
= 21.61 x 1.5 + 24.5 x 2.5
= 93.57 t

A_c = Area of transformed section on one side of interface
= 0.25 x 3.2 (refer section property)
= 0.8 m^2

$\overline{Y}$ = Dist. Of centroid of the area under consideration from the neutral axis of the composite section
= 0.89877 - 0.125 (refer section property)
= 0.77377 m

I = M . I . of the composite section
= 1.42534E+12 mm^4 (refer section property)
= 1.42534 m^4

The ultimate longitudinal shear V_L per unit length ,

$$= \frac{93.57 \times 0.8 \times 0.7738}{1.42534}$$

= 40.63 t/m

ΣQ_u = $A_s f_u$ x 0.001

Where,

Q_u = The ultimate shear resisting capacity of anchorage connector in kN
A_s = The c / s Area of the anchorage connector in sq mm
f_u = The ultimate tensile strength of anchorage connector in Mpa

$$\Sigma Q_u = [\quad 2 \times 113 \times \frac{1000}{125} \times 415$$

= 750.32 kN = 75.0 t > 40.63 t O.K.

Provide 2L - 12 ф @ 125 c / c in addition to shear reinforcement .

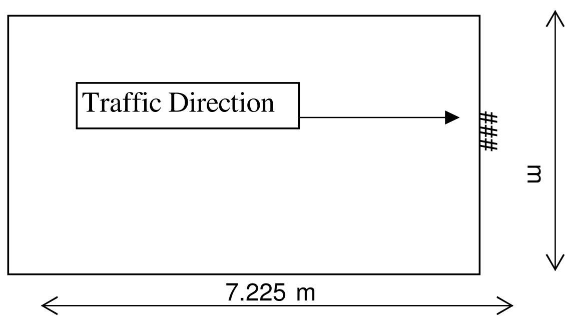
DESIGN OF DECK SLAB

DESIGN OF DECK SLAB

The Deck slab Panel is designed as a Two way slab spanning between the main girders and Cross diaphragm, it is continuous.

C/c of main girders	=	2.8	m
Web width of main girders	=	0.275	m
clear span (lx) between main girders	=	2.525	m
C/c of Cross diaphragms	=	7.5	m
Clear span (ly) between croos diaphragms	=	7.225	m

Ratio of span k = ly/lx = 2.86 > 2.0



The behaviour of the deck slab pannel is considered to be as Oneway slab. And will be analysed and designed accordingly.

ANALYSIS AND DESIGN

The deck slab pannel will be analysed using the tables given in "DESIGN TABLES FOR CONCRETE BRIDGE DECK SLAB". A hand book published by SERC, ROORKEE both for the effects of dead load and concentrated strip loads due to IRC loadings.
Pigueads method will be adopted for evaluating the desihn moments.

Panel dimension = 2.525 x 7.225 = 18.24 m^2
Slab thickness = 250 mm
Wearing coat thickness = 56 mm

Key charts corresponding to the nearest panel size,

Panel size			Key chart No.
2.500	x	7	207
2.5	x	7.25	224
2.75	x	7	208
2.75	x	7.25	225

Dead Load on the Panel

Dead load due to Deck slab	=	0.25	x	18.24	x	25	=	114.0	kN
Dead load due to Wearing coat	=	0.056	x	18.24	x	22	=	22.5	kN
						Total load (W)	=	136.5	kN
								0.75	t/m^2

DEAD LOAD MOMENTS

Treating the slab as simply supported on all four sides, Dead load moments as per Table 4 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207)

Mxc = 151.43 x 0.75 = 113.3 kg-m/m
Myc = 723.55 x 0.75 = 541.4 kg-m/m

II) For panel size = 2.5 x 7.25 (Key chart No. = 224)

Mxc = 147.64 x 0.75 = 110.5 kg-m/m
Myc = 730.7 x 0.75 = 546.7 kg-m/m

For panel size = 2.5 x 2.53 m by inter polation

Mxc = 110.5 '- (110.5 - 113.3) x 4.73
(7.25 - 7)
= 164.06 kg-m/m

Myc = 546.7 '- (546.7 - 541.4) x 4.73
(7.25 - 7)
= 445.60 kg-m/m

III) For panel size = 2.75 x 7 (Key chart No. = 208)

Mxc = 196.66 x 0.75 = 147.1 kg-m/m
Myc = 847.73 x 0.75 = 634.3 kg-m/m

IV) For panel size = 2.75 x 7.25 (Key chart No. = 225)

Mxc = 191.88 x 0.75 = 143.6 kg-m/m
Myc = 858.77 x 0.75 = 642.5 kg-m/m

For panel size = 2.75 x 2.53 m by inter polation

Mxc = 143.6 '- (143.6 - 147.1) x 4.73
(7.25 - 7)
= 211.16 kg-m/m

Myc = 642.5 '- (642.5 - 634.3) x 4.73
(7.25 - 7)
= 486.42 kg-m/m

Actual panel size = 7.23 x 2.53 m by inter polation

Mxc = 164.06 '- (164.06 - 211.16) x -4.48
(2.75 - 2.5)
= -679.03 kg-m/m
= -6.79 kN-m/m

Myc = 486.4 '- (486.4 - 445.6) x -4.48
(2.75 - 2.5)
= 1216.98 kg-m/m
= 12.17 kN-m/m

LIVE LOAD MOMENTS

a) IRC - CLASS A LOADING

Treating the slab as simply supported on all four sides, CLASS A live load moments are evaluated as per Table 3 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207)

Mxc = 835.49 kg-m/m
Myc = 1621.06 kg-m/m

II) For panel size = 2.5 x 7.25 (Key chart No. = 224)

Mxc = 833.74 kg-m/m
Myc = 1622.11 kg-m/m

For panel size = 2.5 x 2.53 m by inter polation

$$Mxc = 833.7 - \frac{(833.7 - 835.5)}{(7.25 - 7)} \times 4.73$$

= 866.82 kg-m/m

$$Myc = 1622.1 - \frac{(1622.1 - 1621.1)}{(7.25 - 7)} \times 4.73$$

= 1602.27 kg-m/m

III) For panel size = 2.75 x 7 (Key chart No. = 208)

Mxc = 907.47 kg-m/m
Myc = 1723.37 kg-m/m

IV) For panel size = 2.75 x 7.25 (Key chart No. = 225)

Mxc = 906 kg-m/m
Myc = 1726.38 kg-m/m

For panel size = 2.75 x 2.53 m by inter polation

$$Mxc = 906.0 - \frac{(906.0 - 907.5)}{(7.25 - 7)} \times 4.73$$

= 933.78 kg-m/m

$$Myc = 1726.4 - \frac{(1726.4 - 1723.4)}{(7.25 - 7)} \times 4.73$$

= 1669.49 kg-m/m

Actual panel size = 7.23 x 2.53 m by inter polation

Mxc = 866.82 '- (866.82 - 933.78) x -4.48 / (2.75 - 2.5)
= -331.91 kg-m/m
= -3.32 kN-m/m

Myc = 1669.5 '- (1669.5 - 1602.3) x -4.48 / (2.75 - 2.5)
= 2872.84 kg-m/m
= 28.73 kN-m/m

Impact Factors For IRC Class A
As per 211.2 of IRC 6 for CLASS A

Impact Factor for Mxc = $1 + \left(\frac{4.5}{6 + 7.23}\right)$ = 1.34 < 1.5
= 1.340

Impact Factor for Myc = $1 + \left(\frac{4.5}{6 + 2.53}\right)$ = 1.528 > 1.5
= 1.50

"CLASS A" - Live Load Moment with Impact are

Mxc	=	-3.319	x	1.340 =	-4.45	kN-m
Myc	=	28.728	x	1.500 =	43.09	kN-m

b) IRC - CLASS 70 R LOADING

Treating the slab as simply supported on all four sides, CLASS 70R live load moments are evaluated as per Table 1 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207)

Mxc = 1305.09 kg-m/m
Myc = 3080.15 kg-m/m

II) For panel size = 2.5 x 7.25 (Key chart No. = 224)

Mxc = 1300.56 kg-m/m
Myc = 3093.95 kg-m/m

For panel size = 2.5 x 2.53 m by inter polation

Mxc = 1300.6 '- (1300.6 - 1305.1) x 4.73 / (7.25 - 7)
= 1386.18 kg-m/m

Myc = 3094.0 '- (3094.0 - 3080.2) x 4.73 / (7.25 - 7)
= 2833.13 kg-m/m

III) For panel size = 2.75 x 7 (Key chart No. = 208)

Mxc = 1445.17 kg-m/m
Myc = 3366.74 kg-m/m

IV) For panel size	=	2.75	x	7.25		(Key chart No. =	225)	
Mxc	=	1437.82	kg-m/m					
Myc	=	3384.6	kg-m/m					

For panel size = 2.75 x 2.53 m by inter polation

Mxc = 1437.8 '- $\dfrac{(1437.8 - 1445.2)}{(7.25 - 7)}$ x 4.73

= 1576.74 kg-m/m

Myc = 3384.6 '- $\dfrac{(3384.6 - 3366.7)}{(7.25 - 7)}$ x 4.73

= 3047.05 kg-m/m

Actual panel size = 7.23 x 2.53 m by inter polation

Mxc = 1386.18 '- $\dfrac{(1386.18 - 1576.74)}{(2.75 - 2.5)}$ x -4.48

= -2024.81 kg-m/m

= -20.25 kN-m/m

Myc = 3047.0 '- $\dfrac{(3047.0 - 2833.1)}{(2.75 - 2.5)}$ x -4.48

= 6876.14 kg-m/m

= 68.76 kN-m/m

As per 211.3 of IRC 6 for CLASS 70 R wheeled vehicle Impact Factor = 1.25

"CLASS 70 R " - Live Load Moment with Impact are

Mxc	=	-20.25	x	1.250	=	-25.31	kN-m >	-4.45	(Class A)
Myc	=	68.76	x	1.250	=	85.95	kN-m >	43.09	(Class A)

Hence IRC Class 70R loading Governs the Design

<u>Total BM</u>

		DL B.M		LL B.M			
Mxc	=	-6.79	+	-25.31	=	-32.10	kN-m
Myc	=	12.17	+	85.95	=	98.12	kN-m

<u>Moments afer considering the coefficient of continuity</u>

Mxc	=	-32.10	x	0.8	=	-25.7	kN-m
Myc	=	98.12	x	0.8	=	78.5	kN-m

DESIGN OF SLAB

Grade of Concrete = **M 40**
Grade of steel = **Fe 415**

As Per 303.3.3

σ_{cbc} = 13.32 Mpa
m = 10
σ_{st} = 200 Mpa

$$n1 = 1 \Big/ \left(1 + \frac{200}{10 \times 13.32} \right)$$
= 0.400

j = 0.867
R = 0.50 x 13.32 x 0.400 x 0.867
= 2.308

$$d\ reqd = \left(\frac{78.50 \times 10^6}{2.308 \times 1000} \right)^{0.5}$$

= 184.44 < 215 mm (Effective depth Provided)

Reinforcement for Myc

$$Ast\ reqd\ For\ Myc = \frac{78.50 \times 10^6}{200 \times 0.867} \times 215$$
= 2106 mm^2/m

Provide Tor - 20 @ 140 mm c/c at top and bottom

Ast Provided = 2244 mm^2/m

Reinforcement for Mxc

$$Ast\ reqd\ For\ Mxc = \frac{-25.68 \times 10^6}{200 \times 0.867} \times 215$$
= -689 mm^2/m

Provide Tor - 12 @ 140 mm c/c at top and bottom

Ast Provided = 808 mm^2/m

DESIGN OF DIAPHRAGM

DESIGN OF END DIAPHRAGM

Sagging moment = 246.92 Kn m

Hogging moment = 622.76 Kn m

Shear force = 1221.1 Kn

Structural Design

Diaphragm span(c/c of girders) = 2.8 m
= 2.8 m

As per clause 29.2 of IS:456
1.15 times span = 3.22 > 2.8 m
Hence adopt 2.8 m as the effective span
Diaphragm depth = 1.8 m

$l / D = \frac{2.8}{1.8}$ 1.56 < 2.5 for continuous beam

Hence the end diaphragm is designed as a deep beam as per clause29 of IS:456 - 2000.

Lever arm Z = 0.2 (L + 1.5 D)
= 0.2 2.8 + 1.5 x 1.8) = 1.1 m
= 1100 mm
= 200 Mpa

Allowable stress in H.Y.S.D σ_{st}

Grade of concrete M 40	σ_{cbc}	=	13.33	N/mm^2
	k	=	0.4000	
	j	=	0.867	
	Q	=	2.311	N/mm^2

+ ve Reinforcement

$$\frac{246.92 \times 10^6}{200 \times 1100} = 1122.364 \text{ mm}^2$$

Min. Reinforcement = $\frac{0.2}{100} \times 450 \times 1800$ = 1620 mm^2

Hence provide 3 nos. Tor - 20 + 3 nos.Tor - 20
reinforcement provided at bottom = 1884.96 mm^2
in 0.25D-0.05L from bottom i.e in 310 mm from bottom of diaphragm.

- ve Reinforcement

$$Ast = \frac{622.76 \times 10^6}{200 \times 1100} = 2830.727 \text{ mm}^2$$

> 1620 mm^2

=

Distribution of -ve Reinforcement

a) at 0.2 x D = 0.36 m

$$Ast = 2830.727 \times \left\{ 0.5 \times \left(\frac{2.8}{1.8} - 0.5 \right) \right\} = 1494.0$$

= 1494

Hence provide 3 nos. Tor- 20 + 3 nos. Tor- 20 = 1884.96 mm^2

b) Balance Ast = 945.77 mm^2

= to be distributed in 0.6D below the Reinforcement at (a) above.

0.6 x 1.8 = 1.08 m

Hence, provide 13 nos Tor- 10 at middle = 1021.02 mm^2

SHEAR

Since this is a deep beam shear does not govern. Hence provide Min. shear Reinforcement of 0.15 %

$$Asv = \frac{0.15}{100} \times 1800 \times 450 = 1215 \ mm^2$$

provide 16 ϕ at 125 mm c/c = 1608.50 mm^2

Total Shear = 1221.1 KN

Required Ast as hanging R/F = 6105.5 mm^2

Required Ast per m length = 847.9861 mm^2/m

Provide 2 Legg 10 ϕ @ 150 mm c/c = 1047.198 mm^2/m

Side face reinforcement

0.1% of web area on either face with spacing not more than 450mm

Required Ast = = 810 mm^2/m

Provide 5 nos 16 ϕ on either side. = 1005.31 mm^2/m

Design of cross girders

Concrete grade "M"		40	N/mm^2
Steel grade "Fe"		415	N/mm^2
Bending compressive stress in concrete (c)		13.333	N/mm^2
Bending tensile stress in steel (t)		200	N/mm^2
Modular ratio		10	
Intermediate cross girder			
Thickness of cross girder		0.3	m
Depth of cross girder including deck slab		2.75	m
End cross girder			
Thickness of cross girder		0.45	m
Depth of cross girder including deck slab		2.75	m
Depth of deck slab		0.25	m
Spacing of longitudinal girders c/c		2.40	m
Web thickness at mid span		0.28	m
Web thickness at end of span		0.65	m

<u>Intermediate cross girder</u>

Maximum Hogging Bending moment		768.09	kN.m
Maximum Sagging Bending moment		1367.5	kN.m
Maximum Shear Force		190.13	kN
Span c/c of supports		2.40	m
Clear span		2.13	m
Effective span		2.40	m
Overall depth		2.75	m
Span / Depth		0.87	
As per IS:456-2000, clause 29.1 Span / Depth	<	2.50	

Hence cross girder will be designed as deep beam

Hogging Bending Moment

Maximum Hogging Bending moment	768.09	kN.m
For continuous beam, Lever arm z = 0.2 (l+1.5D)	1.305	m
Reinforcement required at top Ast = M / t z	2943	mm^2
Proportion of reinforcement in zone 0.2 D from top edge = 0.5 (l/D - 0.5)	0.14	
Area of reinf. in this zone	401	mm^2

Provide :	Dia of bars	20	0		
	No. of bars required	1	0		
	No. of bars provided	4	0		
	Area provided	1257	0		
	Total area			1257	mm^2

Provide this reinf. within a zone of 0.2D from top face i.e. 0.55 m

Proportion of reinf. in remaining zone 0.3 D on either side from mid depth 0.86

Area of reinf. in this zone 2542 mm^2

Provide :	Dia of bars	16	12 mm		
	No. of bars required	13	0		
	No. of bars provided on both faces	16	0		
	No. of bars provided on each face	8	0		
	Area provided	3217	0		
	Total area			3217	mm^2

Sagging Bending Moment

Maximum Sagging Bending moment due to all loads			1367.5	kN.m
Reinforcement required at bottom Ast = M / t z			5239	mm^2
Minimum reinforcement percentage			0.2	%
Minimum area of reinf. Required			1650	mm^2
Reinforcement to be provided			5239	mm^2

Provide :	Dia of bars	32	20 mm		
	No. of bars required	7	0		
	No. of bars provided	7	0		
	Area provided	5630	0 mm^2		
	Total area			5630	mm^2
Provide this reinf. within a zone of (0.25 D-0.05 l) from bottom face i.e.				0.57	m

Design for Shear Force

Maximum Shear Force	190.13	kN
As per IS:456-2000, clause 29.3.3, deep beam will be designed for hanging action		
Reinforcement required for this action	951	mm^2
Reinforcement required per meter length	792	mm^2
Dia of stirrups	12	mm
No. of legs	2	
Spacing required	286	mm
Spacing provided	125	mm

```
INPUT FILE: END.STD
     1. STAAD PLANE END CROSS GIRDER DURING JACKED UP CONDITION
     2. INPUT WIDTH 79
     3. UNIT METER MTON
     4. JOINT COORDINATES
     5. 1 0 0 0; 2 0.51 0 0; 3 2.29 0 0; 4 2.8 0 0; 5 3.31 0 0
     6. 6 5.09 0 0; 7 5.6 0 0; 8 6.11 0 0; 9 7.89 0 0; 10 8.4 0 0
     7. MEMBER INCIDENCES
     8. 1 1 2 9 1 1
     9. MEMBER PROPERTY INDIAN
    10. 1 TO 9 PRIS YD 1.0 ZD 1.0
    11. SUPPORTS
    12. 2 3 5 6 8 9 PINNED
    13. CONSTANTS
    14. E 3.6E+006
    15. POISSON 0.15 ALL
    16. *REACTION ON BEARING DUE TO DL+SIDL
    17. LOAD 1
    18. JOINT LOAD
    19. 1 10 FY -122.11
    20. 4 7 FY -99.38
    21. PERFORM ANALYSIS
    22. PRINT MEMBER FORCES
        MEMBER   FORCES
     ALL UNITS ARE -- MTON METE
  MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y       MOM-Z
      1    1     1      0.00   -122.11     0.00      0.00      0.00        0.00
                 2      0.00    122.11     0.00      0.00      0.00      -62.28

      2    1     2      0.00     36.70     0.00      0.00      0.00       62.28
                 3      0.00    -36.70     0.00      0.00      0.00        3.05

      3    1     3      0.00     42.44     0.00      0.00      0.00       -3.05
                 4      0.00    -42.44     0.00      0.00      0.00       24.69

      4    1     4      0.00    -56.94     0.00      0.00      0.00      -24.69
                 5      0.00     56.94     0.00      0.00      0.00       -4.35

      5    1     5      0.00      0.00     0.00      0.00      0.00        4.35
                 6      0.00      0.00     0.00      0.00      0.00       -4.35

      6    1     6      0.00     56.94     0.00      0.00      0.00        4.35
                 7      0.00    -56.94     0.00      0.00      0.00       24.69

      7    1     7      0.00    -42.44     0.00      0.00      0.00      -24.69
                 8      0.00     42.44     0.00      0.00      0.00        3.05

      8    1     8      0.00    -36.70     0.00      0.00      0.00       -3.05
                 9      0.00     36.70     0.00      0.00      0.00      -62.28

      9    1     9      0.00    122.11     0.00      0.00      0.00       62.28
                10      0.00   -122.11     0.00      0.00      0.00        0.00
    23. FIN
```

APPENDIX

```
INPUT FILE: girder.STD
STAAD PLANE
START JOB INFORMATION
ENGINEER DATE 08-JUN-06
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER MTON
JOINT COORDINATES
1 0.000  0.0 0.0
2 0.400  0.0 0.0
3 2.688  0.0 0.0
4 2.9  0.0 0.0
5 4.150  0.0 0.0
6 5.400  0.0 0.0
7 7.900  0.0 0.0
8 11.650  0.0 0.0
9 15.400  0.0 0.0
10 19.150  0.0 0.0
11 22.900  0.0 0.0
12 25.400  0.0 0.0
13 26.650  0.0 0.0
14 27.9  0.0 0.0
15 28.113  0.0 0.0
16 30.400  0.0 0.0
17 30.800  0.0 0.0
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11
11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17
CONSTANTS
E 3.65E06 ALL
POISSON 0.15 ALL
DENSITY 2.5 ALL
MEMBER PROPERTY INDIAN
****OUTER GIRDER
6 TO 11 PRIS AX 0.9098 IX 0.0248 IZ 0.3951
1 TO 3 14 TO 16 PRIS AX 1.0983 IX 0.0545 IZ 0.4184
4 5 12 13 PRIS AX 1.2868 IX 0.0843 IZ 0.4417
SUPPORTS
2 PINNED
16 FIXED BUT FX MZ
*****************DEAD LOAD (SELF WEIGHT OF THE GIRDER)
LOAD 1
MEMBER LOAD
6 TO 11 UNI GY -2.76525
1 TO 2 15 TO 16 UNI GY -3.84664
3 14 UNI GY -3.846624
4 13 UNI GY -3.306124
5 12  UNI GY -2.76525
***************DIAPHRAM*************
LOAD 2
JOINT LOAD
2 16 FY -4.268
7 9 11 FY -2.9754
PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 2  4 6 7 8
MEMBER   FORCES   LIST     2
```

MEMBER END FORCES   STRUCTURE TYPE = PLANE

ALL UNITS ARE -- MTON METE

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
2	1	2	0.00	44.86	0.00	0.00	0.00	0.31
		3	0.00	-36.06	0.00	0.00	0.00	92.26
	2	2	0.00	4.46	0.00	0.00	0.00	0.00
		3	0.00	-4.46	0.00	0.00	0.00	10.21
4	1	4	0.00	35.24	0.00	0.00	0.00	-99.82
		5	0.00	-31.11	0.00	0.00	0.00	141.29
	2	4	0.00	4.46	0.00	0.00	0.00	-11.16
		5	0.00	-4.46	0.00	0.00	0.00	16.74
6	1	6	0.00	27.65	0.00	0.00	0.00	-178.01
		7	0.00	-20.74	0.00	0.00	0.00	238.50
	2	6	0.00	4.46	0.00	0.00	0.00	-22.32
		7	0.00	-4.46	0.00	0.00	0.00	33.47
7	1	7	0.00	20.74	0.00	0.00	0.00	-238.50
		8	0.00	-10.37	0.00	0.00	0.00	296.83
	2	7	0.00	1.49	0.00	0.00	0.00	-33.47
		8	0.00	-1.49	0.00	0.00	0.00	39.05
8	1	8	0.00	10.37	0.00	0.00	0.00	-296.83
		9	0.00	0.00	0.00	0.00	0.00	316.27
	2	8	0.00	1.49	0.00	0.00	0.00	-39.05
		9	0.00	-1.49	0.00	0.00	0.00	44.63

55. PRINT SUPPORT REACTION

SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE   STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
2	1	0.00	46.40	0.00	0.00	0.00	0.00
	2	0.00	8.73	0.00	0.00	0.00	0.00
16	1	0.00	46.40	0.00	0.00	0.00	0.00
	2	0.00	8.73	0.00	0.00	0.00	0.00

56. PRINT MAXFORCE ENVELOPE LIST 1 TO 16

MAXFORCE ENVELOPE LIST 1

MAX AND MIN FORCE VALUES AMONGST ALL SECTION LOCATIONS

MEMB		FY/ FZ	DIST DIST	LD LD	MZ/ MY	DIST DIST	LD LD	FX	DIST	LD
1	MAX	0.00	0.00	1	0.31	0.40	1			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	-1.54	0.40	1	0.00	0.40	2			
		0.00	0.40	2	0.00	0.40	2	0.00	0.40	2

2	MAX	44.86	0.00	1	0.31	0.00	1			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	4.46	2.29	2	-92.26	2.29	1			
		0.00	2.29	2	0.00	2.29	2	0.00	2.29	2
3	MAX	36.06	0.00	1	-10.21	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	4.46	0.21	2	-99.82	0.21	1			
		0.00	0.21	2	0.00	0.21	2	0.00	0.21	2
4	MAX	35.24	0.00	1	-11.16	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	4.46	1.25	2	-141.29	1.25	1			
		0.00	1.25	2	0.00	1.25	2	0.00	1.25	2
5	MAX	31.11	0.00	1	-16.74	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	4.46	1.25	2	-178.01	1.25	1			
		0.00	1.25	2	0.00	1.25	2	0.00	1.25	2
6	MAX	27.65	0.00	1	-22.32	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	4.46	2.50	2	-238.50	2.50	1			
		0.00	2.50	2	0.00	2.50	2	0.00	2.50	2
7	MAX	20.74	0.00	1	-33.47	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	1.49	3.75	2	-296.83	3.75	1			
		0.00	3.75	2	0.00	3.75	2	0.00	3.75	2
8	MAX	10.37	0.00	1	-39.05	0.00	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	0.00	3.75	1	-316.27	3.75	1			
		0.00	3.75	2	0.00	3.75	2	0.00	3.75	2
9	MAX	0.00	0.00	1	-39.05	3.75	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	-10.37	3.75	1	-316.27	0.00	1			
		0.00	3.75	2	0.00	3.75	2	0.00	3.75	2
10	MAX	-1.49	0.00	2	-33.47	3.75	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	-20.74	3.75	1	-296.83	0.00	1			
		0.00	3.75	2	0.00	3.75	2	0.00	3.75	2
11	MAX	-4.46	0.00	2	-22.32	2.50	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	-27.65	2.50	1	-238.50	0.00	1			
		0.00	2.50	2	0.00	2.50	2	0.00	2.50	2
12	MAX	-4.46	0.00	2	-16.74	1.25	2			
		0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
	MIN	-31.11	1.25	1	-178.01	0.00	1			
		0.00	1.25	2	0.00	1.25	2	0.00	1.25	2
13	MAX	-4.46	0.00	2	-11.16	1.25	2			

	0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
MIN	-35.24	1.25	1	-141.29	0.00	1			
	0.00	1.25	2	0.00	1.25	2	0.00	1.25	2
14 MAX	-4.46	0.00	2	-10.21	0.21	2			
	0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
MIN	-36.06	0.21	1	-99.82	0.00	1			
	0.00	0.21	2	0.00	0.21	2	0.00	0.21	2
15 MAX	-4.46	0.00	2	0.31	2.29	1			
	0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
MIN	-44.86	2.29	1	-92.22	0.00	1			
	0.00	2.29	2	0.00	2.29	2	0.00	2.29	2
16 MAX	1.54	0.00	1	0.31	0.00	1			
	0.00	0.00	1	0.00	0.00	1	0.00	0.00	1
MIN	0.00	0.40	1	0.00	0.37	2			
	0.00	0.40	2	0.00	0.40	2	0.00	0.40	2

********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

57. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= NOV 22,2006   TIME= 17:37:15 ****

```
110. *****************LIVE LOAD****************************
111. DEFINE MOVING LOAD FILE SATY1
112. TYPE 1 CLASSA 1.125
113. TYPE 2 70RW 1.125
114. TYPE 3 70RT 1.1
115. ********************
116. *FROM MEDIAN SIDE
117. *TWO LANE OF CLASS A
118. LOAD GENERATION 105
119. TYPE 1 -18.8 0 4.45 XINC 0.5
120. TYPE 1 -18.8 0 7.95 XINC 0.5
121. *70R WHEELED
122. LOAD GENERATION 92
123. TYPE 2 -13.4 0 5.81 XINC 0.5
124. *70R TRACKED TRAIN
125. LOAD GENERATION 75
126. TYPE 3 -4.576 0 5.93 XINC 0.5
127. *********************
128. *FROM FOOTPATH SIDE
129. *TWO LANE OF CLASS A
130. *LOAD GENERATION 105
131. *TYPE 1 -18.8 0 4.3 XINC 0.5
132. *TYPE 1 -18.8 0 7.8 XINC 0.5
133. *70R WHEELED
134. *LOAD GENERATION 92
135. *TYPE 2 -13.4 0 5.66 XINC 0.5
136. *70R TRACKED
137. *LOAD GENERATION 75
138. *TYPE 3 -4.576 0 5.78 XINC 0.5
139. PERFORM ANALYSIS
140. *PRINT FORCE ENVELOPE NSECTION 2 LIST 18 34 50 66
141. *PRINT FORCE ENVELOPE NSECTION 2 LIST 20 36 52 68
142. *PRINT FORCE ENVELOPE NSECTION 2 LIST 22 38 54 70
143. PRINT FORCE ENVELOPE NSECTION 2 LIST 18 20 22 23 24
     FORCE    ENVELOPE NSECTION 2
MEMB  DISTANCE         FY     LD        MZ     LD       FZ     LD        MY     LD
 18     0.00  MAX    25.11  136       5.99   30       0.00  272       0.00  272
              MIN    -0.59  106      -0.23  210       0.00  272       0.00  272
        1.14  MAX    24.92  138       3.42    1       0.00  272       0.00  272
              MIN    -0.59  106     -26.84  137       0.00  272       0.00  272
        2.29  MAX    24.46  140       3.57    1       0.00  272       0.00  272
              MIN    -0.59  106     -54.36  138       0.00  272       0.00  272
|----------------------------------------------------------------------------
| MAX/MIN FORCE VALUES FOR MEMB     18, AMONGST ALL SECT LOCATIONS
|             FY/   DIST   LD         MZ/   DIST   LD
|             FZ    DIST   LD         MY    DIST   LD        FX       DIST   LD
|
| MAX.      25.11   0.00  136       5.99   0.00   30
|            0.00   0.00    1       0.00   0.00    1       0.00      0.00    1
| MIN.      -0.59   2.29  106     -54.36   2.29  138
|            0.00   2.29  272       0.00   2.29  272       0.00      2.29  272
|----------------------------------------------------------------------------
 20     0.00  MAX    24.46  140       3.60    1       0.00  272       0.00  272
              MIN    -0.59  106     -59.40  138       0.00  272       0.00  272
        0.62  MAX    23.95  136       3.68  106       0.00  272       0.00  272
```

```
               MIN    -0.59  106    -74.25  138      0.00  272      0.00  272
        1.25   MAX    23.95  136      4.05  106      0.00  272      0.00  272
               MIN    -0.59  106    -89.10  137      0.00  272      0.00  272
|----------------------------------------------------------------------------
| MAX/MIN FORCE VALUES FOR MEMB     20, AMONGST ALL SECT LOCATIONS
|              FY/   DIST   LD         MZ/   DIST   LD
|              FZ    DIST   LD         MY    DIST   LD        FX      DIST   LD
|
| MAX.      24.46   0.00  140        4.05   1.25  106
|            0.00   0.00    1        0.00   0.00    1      0.00     0.00    1
| MIN.      -0.59   1.25  106      -89.10   1.25  137
|            0.00   1.25  272        0.00   1.25  272      0.00     1.25  272
|----------------------------------------------------------------------------
 22     0.00   MAX    24.07  138      4.71  106      0.00  272      0.00  272
               MIN    -0.54  106   -119.01  137      0.00  272      0.00  272
        1.25   MAX    23.68  140      5.39  106      0.00  272      0.00  272
               MIN    -0.54  106   -147.66  138      0.00  272      0.00  272

        2.50   MAX    22.64  143      6.06  106      0.00  272      0.00  272
               MIN    -0.54  106   -175.11  140      0.00  272      0.00  272
|----------------------------------------------------------------------------
| MAX/MIN FORCE VALUES FOR MEMB     22, AMONGST ALL SECT LOCATIONS
|              FY/   DIST   LD         MZ/   DIST   LD
|              FZ    DIST   LD         MY    DIST   LD        FX      DIST   LD
|
| MAX.      24.07   0.00  138        6.06   2.50  106
|            0.00   0.00    1        0.00   0.00    1      0.00     0.00    1
| MIN.      -0.54   2.50  106     -175.11   2.50  140
|            0.00   2.50  272        0.00   2.50  272      0.00     2.50  272
|----------------------------------------------------------------------------
 23     0.00   MAX    14.86  150      6.02  106      0.00  272      0.00  272
               MIN    -4.47  216   -174.57  140      0.00  272      0.00  272
        1.87   MAX    14.35  154      5.44  106      0.00  272      0.00  272
               MIN    -4.63  218   -191.07  142      0.00  272      0.00  272
        3.75   MAX    13.70  150      4.85  106      0.00  272      0.00  272
               MIN    -4.63  218   -207.59  143      0.00  272      0.00  272
|----------------------------------------------------------------------------
| MAX/MIN FORCE VALUES FOR MEMB     23, AMONGST ALL SECT LOCATIONS
|              FY/   DIST   LD         MZ/   DIST   LD
|              FZ    DIST   LD         MY    DIST   LD        FX      DIST   LD
|
| MAX.      14.86   0.00  150        6.02   0.00  106
|            0.00   0.00    1        0.00   0.00    1      0.00     0.00    1
| MIN.      -4.63   3.75  218     -207.59   3.75  143
|            0.00   3.75  272        0.00   3.75  272      0.00     3.75  272
|----------------------------------------------------------------------------
 24     0.00   MAX    13.75  151      4.85  106      0.00  272      0.00  272
               MIN    -4.60  218   -207.47  143      0.00  272      0.00  272
        1.87   MAX    13.29  154      4.28  106      0.00  272      0.00  272
               MIN    -4.60  218   -224.33  146      0.00  272      0.00  272
        3.75   MAX    12.22  158      3.72  106      0.00  272      0.00  272
               MIN    -4.60  218   -238.92  148      0.00  272      0.00  272
|----------------------------------------------------------------------------
| MAX/MIN FORCE VALUES FOR MEMB     24, AMONGST ALL SECT LOCATIONS
|              FY/   DIST   LD         MZ/   DIST   LD
|              FZ    DIST   LD         MY    DIST   LD        FX      DIST   LD
```

```
 |
 | MAX.      13.75   0.00  151       4.85   0.00  106
 |            0.00   0.00    1       0.00   0.00    1      0.00     0.00    1
 | MIN.      -4.60   3.75  218    -238.92   3.75  148
 |            0.00   3.75  272       0.00   3.75  272      0.00     3.75  272
 |---------------------------------------------------------------------------
144. PRINT FORCE ENVELOPE NSECTION 2 LIST 24 40 56 72
     FORCE    ENVELOPE NSECTION 2

MEMB  DISTANCE         FY    LD        MZ    LD       FZ    LD        MY    LD

 24      0.00  MAX    13.75  151      4.85  106      0.00  272      0.00  272
               MIN    -4.60  218   -207.47  143      0.00  272      0.00  272
         1.87  MAX    13.29  154      4.28  106      0.00  272      0.00  272
               MIN    -4.60  218   -224.33  146      0.00  272      0.00  272
         3.75  MAX    12.22  158      3.72  106      0.00  272      0.00  272
               MIN    -4.60  218   -238.92  148      0.00  272      0.00  272
 |---------------------------------------------------------------------------
 | MAX/MIN FORCE VALUES FOR MEMB     24, AMONGST ALL SECT LOCATIONS
 |            FY/   DIST   LD        MZ/   DIST   LD
 |            FZ    DIST   LD        MY    DIST   LD        FX      DIST   LD
 |
 | MAX.      13.75   0.00  151       4.85   0.00  106
 |            0.00   0.00    1       0.00   0.00    1      0.00     0.00    1
 | MIN.      -4.60   3.75  218    -238.92   3.75  148
 |            0.00   3.75  272       0.00   3.75  272      0.00     3.75  272
 |---------------------------------------------------------------------------
 40      0.00  MAX    19.27  232      2.94  106      0.00  272      0.00  272
               MIN   -12.84  223   -188.10  142      0.00  272      0.00  272
         1.87  MAX    13.02  236      2.83  106      0.00  272      0.00  272
               MIN   -20.07  227   -190.96  146      0.00  272      0.00  272
         3.75  MAX     8.91  240      2.73  106      0.00  272      0.00  272
               MIN   -27.70  231   -177.70  147      0.00  272      0.00  272
 |---------------------------------------------------------------------------
 | MAX/MIN FORCE VALUES FOR MEMB     40, AMONGST ALL SECT LOCATIONS
 |            FY/   DIST   LD        MZ/   DIST   LD
 |            FZ    DIST   LD        MY    DIST   LD        FX      DIST   LD
 |
 | MAX.      19.27   0.00  232       2.94   0.00  106
 |            0.00   0.00    1       0.00   0.00    1      0.00     0.00    1
 | MIN.     -27.70   3.75  231    -190.96   1.87  146
 |            0.00   3.75  272       0.00   3.75  272      0.00     3.75  272
 |---------------------------------------------------------------------------
 56      0.00  MAX    12.36   55      2.90  106      0.00  272      0.00  272
               MIN    -6.36  223   -145.28   52      0.00  272      0.00  272
         1.87  MAX     9.48  236      2.38  106      0.00  272      0.00  272
               MIN    -7.32  134   -148.00   56      0.00  272      0.00  272
          3.75  MAX     8.06  240      1.87  106      0.00  272      0.00  272
               MIN    -8.38  138   -146.68  147      0.00  272      0.00  272
 |---------------------------------------------------------------------------
 | MAX/MIN FORCE VALUES FOR MEMB     56, AMONGST ALL SECT LOCATIONS
 |            FY/   DIST   LD        MZ/   DIST   LD
 |            FZ    DIST   LD        MY    DIST   LD        FX      DIST   LD
 |
 | MAX.      12.36   0.00   55       2.90   0.00  106
 |            0.00   0.00    1       0.00   0.00    1      0.00     0.00    1
```

```
 | MIN.      -8.38   3.75  138    -148.00   1.87   56
 |            0.00   3.75  272       0.00   3.75  272      0.00     3.75  272
 |---------------------------------------------------------------------------
  72     0.00  MAX     6.11   64       2.22  106      0.00  272      0.00  272
               MIN    -0.97   24    -111.98   52      0.00  272      0.00  272
         1.87  MAX     5.87   59       2.12  106      0.00  272      0.00  272
               MIN    -0.97   24    -117.69   53      0.00  272      0.00  272
         3.75  MAX     5.65   63       2.01  106      0.00  272      0.00  272
               MIN    -0.97   24    -123.46   54      0.00  272      0.00  272
 |---------------------------------------------------------------------------
 | MAX/MIN FORCE VALUES FOR MEMB     72, AMONGST ALL SECT LOCATIONS
 |             FY/   DIST   LD         MZ/   DIST   LD
 |             FZ    DIST   LD         MY    DIST   LD       FX      DIST   LD
 |
 | MAX.       6.11   0.00   64       2.22   0.00  106
 |            0.00   0.00    1       0.00   0.00    1      0.00     0.00    1
 | MIN.      -0.97   3.75   24    -123.46   3.75   54
 |            0.00   3.75  272       0.00   3.75  272      0.00     3.75  272
 |---------------------------------------------------------------------------
 ********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********
 145. LOAD LIST 148 158 132
 146. PRINT MEMBER FORCES LIST  24
 MEMBER   FORCES   LIST     24

 MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y       MOM-Z

    24  132    25      0.00     -1.74     0.00      0.54      0.00     -158.60
               26      0.00      4.20     0.00     -0.54      0.00      144.50
        148    25      0.00     11.85     0.00      0.41      0.00     -201.84
               26      0.00     -7.65     0.00     -0.41      0.00      238.92
        158    25      0.00     13.37     0.00      0.33      0.00     -152.03
               26      0.00    -12.22     0.00     -0.33      0.00      198.15

 147. *****
 148. LOAD LIST 143 150 139
 149. PRINT MEMBER FORCES LIST 23
 MEMBER   FORCES   LIST     23

 MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y       MOM-Z
    23  139    24      0.00      7.69     0.00      0.65      0.00     -174.39
               25      0.00     -2.77     0.00     -0.65      0.00      198.64
        143    24      0.00     12.41     0.00      0.62      0.00     -169.78
               25      0.00     -8.94     0.00     -0.62      0.00      207.59
        150    24      0.00     14.86     0.00      0.63      0.00     -144.01
               25      0.00    -13.70     0.00     -0.63      0.00      195.41
 150. *************
 151. LOAD LIST 140 143 150
 152. PRINT MEMBER FORCES LIST 22
 MEMBER   FORCES   LIST     22
 MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y       MOM-Z

    22  140    23      0.00     23.68     0.00      1.72      0.00     -117.69
               24      0.00    -21.94     0.00     -1.72      0.00      175.11
        143    23      0.00     22.64     0.00      1.87      0.00     -113.76
               24      0.00    -22.64     0.00     -1.87      0.00      170.34
        150    23      0.00     19.88     0.00      2.02      0.00      -94.93
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               24      0.00    -19.88     0.00     -2.02      0.00     144.63
     153. LOAD LIST 137 136 150
   154. PRINT MEMBER FORCES LIST 20
 MEMBER   FORCES   LIST     20
    MEMBER END FORCES    STRUCTURE TYPE = FLOOR
   -----------------
   ALL UNITS ARE -- MTON METE
  MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y      MOM-Z
     20  136    21      0.00     23.95     0.00      1.96      0.00     -58.91
                22      0.00    -23.95     0.00     -1.96      0.00      88.84
         137    21      0.00     23.89     0.00      2.07      0.00     -59.24
                22      0.00    -23.89     0.00     -2.07      0.00      89.10
         150    21      0.00     19.38     0.00      2.81      0.00     -46.20
                22      0.00    -19.38     0.00     -2.81      0.00      70.42
   155. LOAD LIST 138 140 150
   156. PRINT MEMBER FORCES LIST 18
 MEMBER   FORCES   LIST     18
  MEMBER  LOAD  JT     AXIAL   SHEAR-Y  SHEAR-Z   TORSION     MOM-Y      MOM-Z

     18  138    19      0.00     24.92     0.00      2.18      0.00       1.76
                20      0.00    -23.76     0.00     -2.18      0.00      54.36
         140    19      0.00     24.46     0.00      2.37      0.00       1.92
                20      0.00    -24.46     0.00     -2.37      0.00      54.05
         150    19      0.00     19.38     0.00      2.81      0.00       2.25
                20      0.00    -19.38     0.00     -2.81      0.00      42.09
      157. FINISH
          *********** END OF THE STAAD.Pro RUN ***********
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INPUT FILE: SIDL.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 26-SEP-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0.000  0.0 0.0
     9. 2 1.080  0.0 0.0
    10. 3 3.368  0.0 0.0
    11. 4 3.580  0.0 0.0
    12. 5 4.830  0.0 0.0
    13. 6 6.080  0.0 0.0
    14. 7 8.580  0.0 0.0
    15. 8 12.330  0.0 0.0
    16. 9 16.080  0.0 0.0
    17. 10 19.830  0.0 0.0
    18. 11 23.580  0.0 0.0
    19. 12 26.080  0.0 0.0
    20. 13 27.330  0.0 0.0
    21. 14 28.580  0.0 0.0
    22. 15 28.793  0.0 0.0
    23. 16 31.080  0.0 0.0
    24. 17 32.160  0.0 0.0
    25. REPEAT ALL 1 0.0 0.0 1.8
    26. REPEAT 3 0.0 0.0 2.8
    27. REPEAT 1 0.0 0.0 1.8
    28. MEMBER INCIDENCES
    29.1 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
    30.11 11 12;12 12 13;13 13 14;14 14 15;15 15 16; 16 16 17;17 18 19; 18 19 20
    31.19 20 21;20 21 22; 21 22 23;22 23 24;23 24 25; 24 25 26;25 26 27;26 27 28
    32.27 28 29;28 29 30; 29 30 31;30 31 32;31 32 33;32 33 34;33 35 36; 34 36 37
    33. 35 37 38; 36 38 39;37 39 40;38 40 41;39 41 42;40 42 43;41 43 44;42 44 45
    34. 43 45 46; 44 46 47;45 47 48;46 48 49;47 49 50;48 50 51;49 52 53;50 53 54
    35. 51 54 55; 52 55 56;53 56 57;54 57 58;55 58 59;56 59 60;57 60 61;58 61 62
    36. 59 62 63;60 63 64;61 64 65;62 65 66;63 66 67;64 67 68;65 69 70; 66 70 71
    37. 67 71 72;68 72 73;69 73 74;70 74 75;71 75 76;72 76 77;73 77 78; 74 78 79
    38. 75 79 80; 76 80 81;77 81 82;78 82 83;79 83 84;80 84 85;81 86 87;82 87 88
    39. 83 88 89; 84 89 90;85 90 91;86 91 92;87 92 93;88 93 94;89 94 95;90 95 96
    40. 91 96 97; 92 97 98; 93 98 99; 94 99 100; 95 100 101; 96 101 102; 97 1 18
    41. 98 18 35; 99 35 52; 100 52 69; 101 69 86; 102 2 19; 103 19 36; 104 36 53
    42. 105 53 70; 106 70 87; 107 5 22; 108 22 39; 109 39 56;110 56 73;111 73 90
    43. 112 7 24; 113 24 41; 114 41 58; 115 58 75; 116 75 92;117 8 25; 118 25 42
    44. 119 42 59; 120 59 76; 121 76 93; 122 9 26; 123 26 43;124 43 60;125 60 77
    45. 126 77 94; 127 10 27; 128 27 44; 129 44 61;130 61 78;131 78 95;132 11 28
    46. 133 28 45; 134 45 62; 135 62 79; 136 79 96;137 13 30;138 30 47;139 47 64
    47. 140 64 81; 141 81 98; 142 16 33;143 33 50;144 50 67;145 67 84;146 84 101
    48. 147 17 34; 148 34 51; 149 51 68; 150 68 85; 151 85 102
    49. MEMBER PROPERTY INDIAN
    50. *OUTER GIRDER
    51. 22 TO 27 70 TO 75 PRIS AX 1.906 IX 0.0404 IZ 1.3135
    52. 17 18 31 32 65 66 79 80 PRIS AX 2.339 IX 0.1104 IZ 1.4253
    53. 19 TO 21 28 TO 30 67 TO 69 76 TO 78 PRIS AX 2.122 IX 0.0754 IZ 1.3694
    54. *INNER GIRDER
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55. 38 TO 43 54 TO 59 PRIS AX 1.806 IX 0.0393 IZ 1.2605
56. 33 34 47 TO 50 63 64 PRIS AX 2.239 IX 0.1093 IZ 1.3623
57. 35 TO 37 44 TO 46 51 TO 53 60 TO 62 PRIS AX 2.022 IX 0.0743 IZ 1.3114
58. *END DIAPHRAM
59. 103 TO 105 143 TO 145 PRIS AX 1.549 IX 0.054 IZ 0.6285
60. *CENTRAL DIAPHRAM
61. 113 TO 115 123 TO 125 133 TO 135 PRIS AX 1.478 IX 0.0240 IZ 0.5107
62. *CANTILEVER PORTION
63. 1 TO 16 81 TO 96 PRIS AX 0.001 IX 0.0001 IZ 0.0001
64. *TRANSVERSE MEMBERS
65. 107 TO112 116 TO122 126 TO132 136 TO141 PRIS AX 0.938 IX0.0098 IZ 0.0049
66. 102 106 142 146 PRIS AX 0.739 IX 0.0077 IZ 0.0038
67. 97 TO 101 147 TO 151 PRIS AX 0.0001 IX 0.0001 IZ 0.0001
68. CONSTANTS
69. E 2.2146E06
70. DEN CONC ALL
71. POISSON 0.15
72. SUPPORTS
73. 19 36 53 70 PINNED
74. 33 50 67 84 FIXED BUT FX MX MY MZ
75. **** SIDL INCLUDING FOOTPATH LL
76. LOAD 1
77. MEMBER LOAD
78. 1 TO 16 UNI GY -1.503
79. 17 TO 32 UNI GY -0.862
80. 33 TO 48 UNI GY -0.552
81. 49 TO 64 UNI GY -0.552
82. 65 TO 80 UNI GY -0.862
83. 81 TO 96 UNI GY -1.503
84. PERFORM ANALYSIS
85. PRINT MEMBER FORCES  LIST 24 40 56 72
  MEMBER   FORCES   LIST     24
MEMBER END FORCES    STRUCTURE TYPE = FLOOR
-----------------
ALL UNITS ARE -- MTON METE
```

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
24	1	25	0.00	1.23	0.00	-2.88	0.00	-164.74
		26	0.00	2.00	0.00	2.88	0.00	163.30
40	1	42	0.00	6.91	0.00	0.52	0.00	-142.92
		43	0.00	-4.84	0.00	-0.52	0.00	164.96
56	1	59	0.00	6.91	0.00	-0.52	0.00	-142.92
		60	0.00	-4.84	0.00	0.52	0.00	164.96
72	1	76	0.00	1.23	0.00	2.88	0.00	-164.74
		77	0.00	2.00	0.00	-2.88	0.00	163.30

```
86. PRINT MEMBER FORCES  LIST 18 20 22 23 24
  MEMBER   FORCES   LIST     18
```

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
18	1	19	0.00	23.58	0.00	5.73	0.00	0.39
		20	0.00	-21.61	0.00	-5.73	0.00	51.30

20	1	21	0.00	21.42	0.00	5.73	0.00	-55.86
		22	0.00	-20.35	0.00	-5.73	0.00	81.97
22	1	23	0.00	12.45	0.00	-1.74	0.00	-98.10
		24	0.00	-10.30	0.00	1.74	0.00	126.54
23	1	24	0.00	11.84	0.00	3.10	0.00	-126.42
		25	0.00	-8.60	0.00	-3.10	0.00	164.75
24	1	25	0.00	1.23	0.00	-2.88	0.00	-164.74
		26	0.00	2.00	0.00	2.88	0.00	163.30

87. FINISH

```
INPUT FILE: SLAB.STD
STAAD PLANE
START JOB INFORMATION
ENGINEER DATE 26-SEP-06
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER MTON
JOINT COORDINATES
1 0.000  0.0 0.0
2 1.080  0.0 0.0
3 3.368  0.0 0.0
4 3.580  0.0 0.0
5 4.830  0.0 0.0
6 6.080  0.0 0.0
7 8.580  0.0 0.0
8 12.330  0.0 0.0
9 16.080  0.0 0.0
10 19.830  0.0 0.0
11 23.580  0.0 0.0
12 26.080  0.0 0.0
13 27.330  0.0 0.0
14 28.580  0.0 0.0
15 28.793  0.0 0.0
16 31.080  0.0 0.0
17 32.160  0.0 0.0
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11
11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17
DEFINE MATERIAL START
ISOTROPIC MATERIAL1
E 3.65E+006
POISSON 0.15
DENSITY 2.4026
DAMP 0.001
END DEFINE MATERIAL
MEMBER PROPERTY INDIAN
****OUTER GIRDER
6 TO 11 PRIS AX 0.9098 IX 0.0248 IZ 0.3951
1 TO 3 14 TO 16 PRIS AX 1.0983 IX 0.0545 IZ 0.4184
4 5 12 13 PRIS AX 1.2868 IX 0.0843 IZ 0.4417
CONSTANTS
MATERIAL MATERIAL1 MEMB 1 TO 16
SUPPORTS
2 16 PINNED
*****************DEAD LOAD (SELF WEIGHT OF THE SLAB)
LOAD 1
MEMBER LOAD
*(2.8*0.5+1.8)*0.25*2.4
1 TO 16 UNI GY -2
* ***  SHUTTERING SLAB
LOAD 2
MEMBER LOAD
1 TO 16 UNI GY -0.5
PERFORM ANALYSIS
PRINT MEMBER FORCES LIST 2 4 6 TO 8
```

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
2	1	2	0.00	30.00	0.00	0.00	0.00	1.17
		3	0.00	-25.42	0.00	0.00	0.00	62.24
	2	2	0.00	7.50	0.00	0.00	0.00	0.29
		3	0.00	-6.36	0.00	0.00	0.00	15.56
4	1	4	0.00	25.00	0.00	0.00	0.00	-67.58
		5	0.00	-22.50	0.00	0.00	0.00	97.27
	2	4	0.00	6.25	0.00	0.00	0.00	-16.90
		5	0.00	-5.62	0.00	0.00	0.00	24.32
6	1	6	0.00	20.00	0.00	0.00	0.00	-123.83
		7	0.00	-15.00	0.00	0.00	0.00	167.58
	2	6	0.00	5.00	0.00	0.00	0.00	-30.96
		7	0.00	-3.75	0.00	0.00	0.00	41.90
7	1	7	0.00	15.00	0.00	0.00	0.00	-167.58
		8	0.00	-7.50	0.00	0.00	0.00	209.77
	2	7	0.00	3.75	0.00	0.00	0.00	-41.90
		8	0.00	-1.88	0.00	0.00	0.00	52.44
8	1	8	0.00	7.50	0.00	0.00	0.00	-209.77
		9	0.00	0.00	0.00	0.00	0.00	223.83
	2	8	0.00	1.88	0.00	0.00	0.00	-52.44
		9	0.00	0.00	0.00	0.00	0.00	55.96

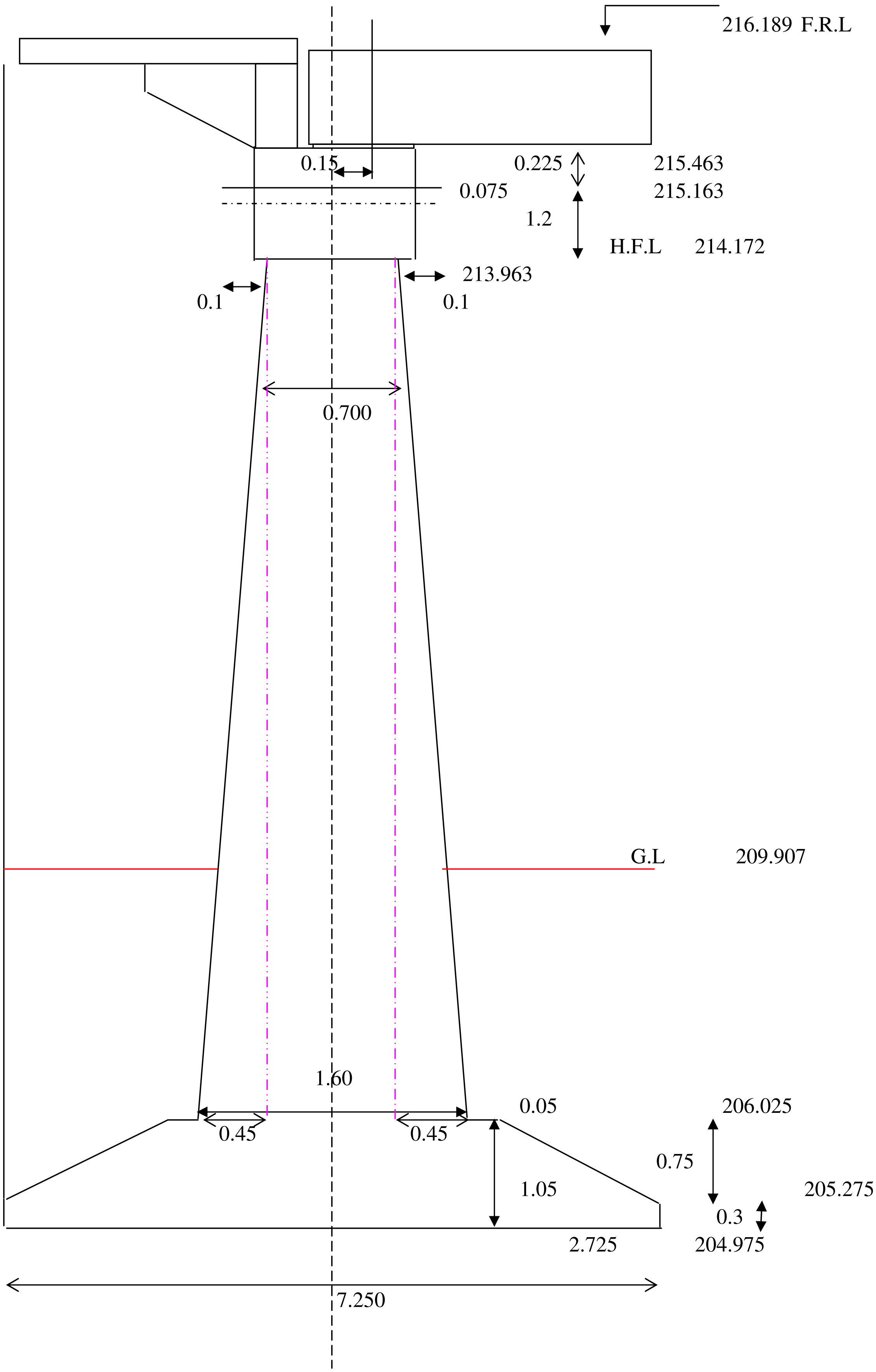
55. PRINT SUPPORT REACTION
SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE

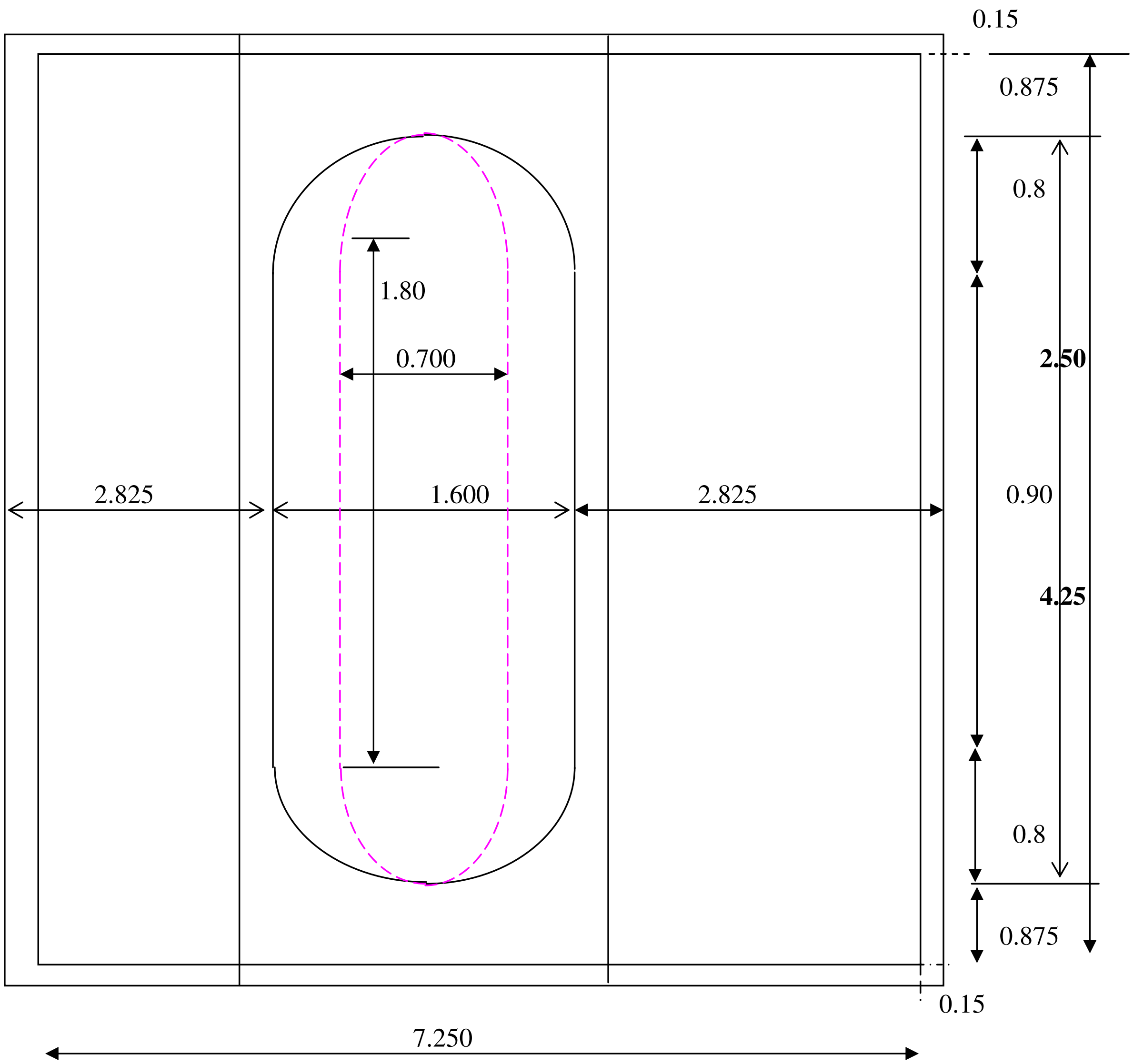
JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
2	1	0.00	32.16	0.00	0.00	0.00	0.00
	2	0.00	8.04	0.00	0.00	0.00	0.00
16	1	0.00	32.16	0.00	0.00	0.00	0.00
	2	0.00	8.04	0.00	0.00	0.00	0.00

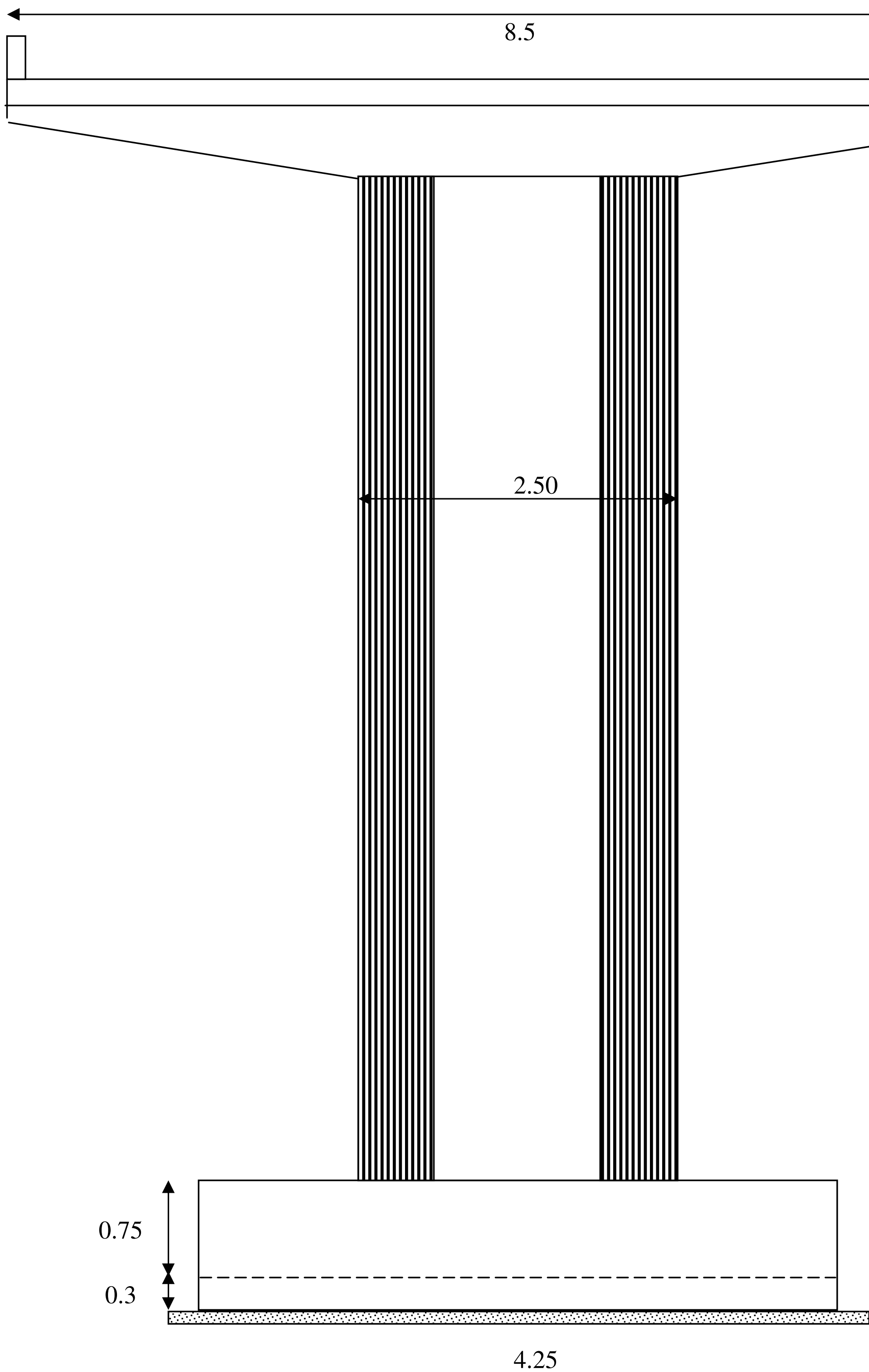
56. FINISH

BRIDGE AT CH:29+400

DESIGN OF SUBSTRUCTURE (ABUTMENT - PIER)


216.189 F.R.L
0.15
0.225
215.463
0.075
215.163
1.2
H.F.L 214.172
213.963
0.1
0.1
0.700
G.L 209.907
1.60
0.05
206.025
0.45
0.45
0.75
1.05
205.275
0.3
2.725
204.975
7.250


0.15
0.875
0.8
1.80
0.700
2.50
2.825
1.600
2.825
0.90
4.25
0.8
0.875
0.15
7.250


8.5
2.50
0.75
0.3
4.25

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete	=	M - 25	
Abutment Cap	=	M - 25	
Grade of reinforcement steel	=	Fe - 415	
Centre to Centre distance of A / Expansion joints	=	9.200	m
Centre to Centre distance of Bearing	=	8.800	m
Depth of superstructure	=	670	mm
Thickness of wearing coat	=	56.00	mm
Formation level along C of carriage way	=	216.189	m
Soffit level	=	215.463	
Pedestal top level	=	215.463	
Height of bearing and Pedestal	=	0.000	m
L.W.L./Bed level	=	209.907	m
H.F.L	=	214.172	m
M.S.L	=	202.700	m
Founding Level	=	204.975	m
Abutment cap top level	=	215.463	m
Gross safe bearing capacity	=	29.400	t/m^2

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

(b) Class 70R wheeled

Bearing neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:

1 IRC : 6 - 2000

2 IRC : 21 - 2000

3 IRC : 78 - 2000

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

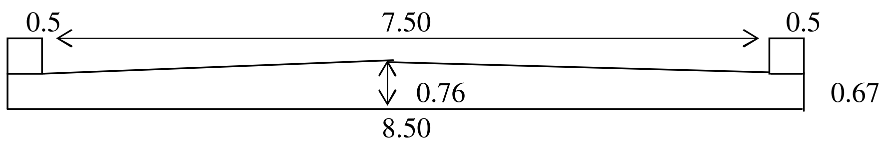
For Class A 2 lane : $\mathbf{F_h}$ **=** 0.2 x 49.5 = 9.896 t

For class 70R wheeled : $\mathbf{F_h}$ **=** 0.2 x 47.72 = 9.544 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 2 lane	**4.95**
70R	**4.77**

Dead load



Dead load of slab	=	14.561	t/m
Wearing coat	=	0.924	t/m
Crashbarrier	=	1.000	t/m
Dead load reaction	=	66.978	t
Sidl reaction	=	13.45	t

Dry Condition with L.L

a) Vertical load and their moments about C/L of Foundation base.

I

	P	e_L	M_L	e_T	M_T
1 D.L Reaction	**P**	**e_L**	**M_L**	**e_T**	**M_T**
a Left span	66.98	0.15	10.047	0	0
2 S.I.D.L					
a Left span	13.45	0.15	2.0176	0	0
	80.429		**12.064**		**0**
4 **L.L**					
a Left span					
70R Wheeled	47.72	0.15	7.158	1.16	55.355
Class A 2 Lane	49.48	0.15	7.422	0.7	34.636
	128.15		**19.222**		**55.355**

II Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1 Pedestal							
a Left span	0	2.4	0	0.15	0	0	0
2 Abut cap R	2.1465	2.4	5.1516	0	0	0	0
Abut cap Tr	4.779	2.4	11.47	0	0	0	0
3 Abut shaft							
up to G.L	10.334	2.4	24.801	0	0	0	0
below G.L	9.8903	2.4	23.737	0	0	0	0
4 Footing	23.348	2.4	56.036	0	0	0	0
5 Earth above footing	118.73	1.80	213.71	0	0	0	0
6							
7 Earth wt on heel side	66.079	1.8	118.942	-2.213	-263.22		
			334.91		**-263**		**0**

b) Horizantal Forces and Moments withrespect to Base

	H_L	H_T	e_L	M_L	e_T	M_T
1 Longitudinal Forces at bearing level						
	H_L	**H_T**	**e_L**	**M_L**	**e_T**	**M_T**
	4.77	0	10.488	50.049	10.488	0
2 Earth pressure				462.11		
	4.77	0		**512.16**		**0**

Summary

P = 463.05 t

M_L = 268 t-m

M_T = 55.355 t-m

A	=	30.813
Z_L	=	37.232
Z_T	=	21.826

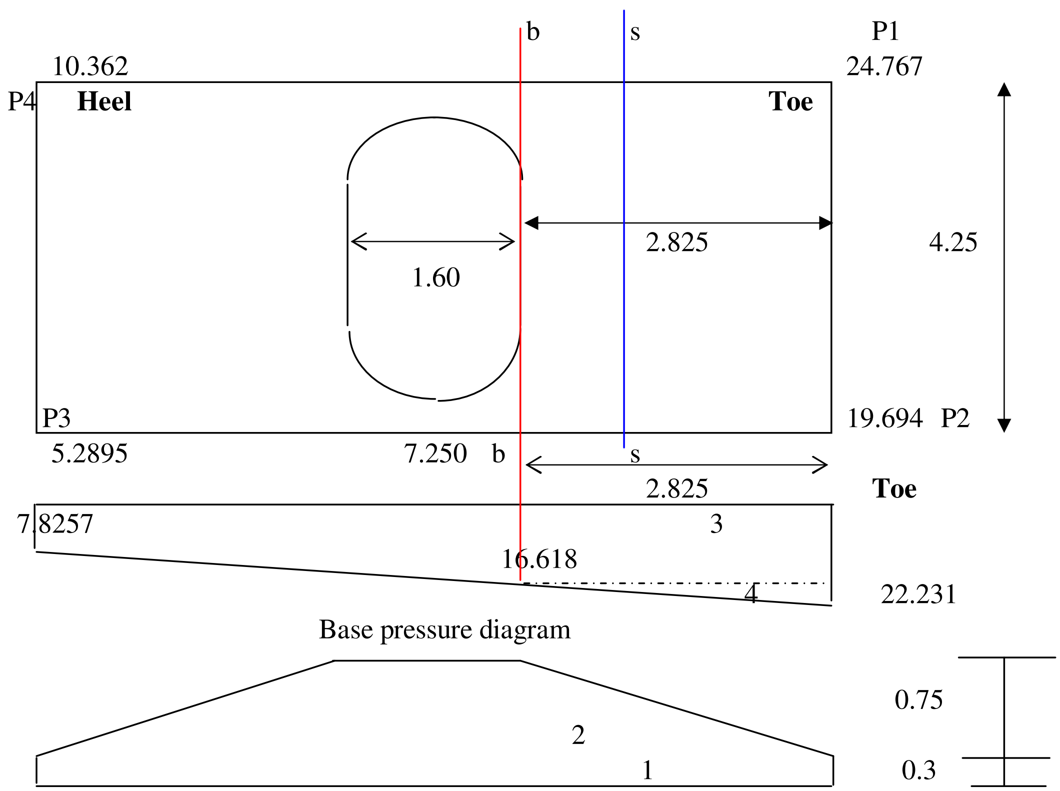
Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{463.05}{30.813} + \frac{268.16}{37.232} + \frac{55.355}{21.826} = 24.767 \ t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{463.05}{30.813} - \frac{268.16}{37.232} - \frac{55.355}{21.826} = 5.2895 \ t/m^2$$



Base pressure diagram

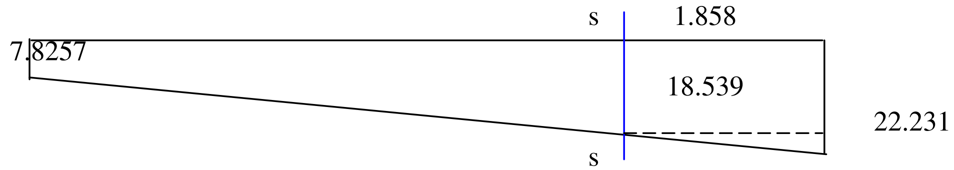
Design of Toe slab

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		2.034		1.4125		2.873
	2		0.5		2.5425		0.9417		2.3942
Upward base pressure									
	3		1		-16.618		1.4125		-23.472
	4		0.5		-7.9283		0.9417		-7.4658

Net bending moment at face of stem	=	25.671	t-m/m
Effective depth required	=	0.4772	m
Effective depth Provided at face of stem	=	0.965	m
Area of reinforcement required	=	1445.8	mm^2
Minimum steel req	=	1447.5	mm^2
Provide **16** φ **120** mm c/c	=	1675.5	mm^2

Shear

Effective depth d of footing	=	0.967	m
Total depth of section at distance d from Abutment	=	0.7933	m
Effective depth at distance d from face of Abutment	=	0.7023	m



width b = 4.25

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		1.3378		0.929		1.2428
	2		0.5		1.6722		0.6193		1.0356
Upward base pressure									
	3		1		-18.539		0.929		-17.223
	4		0.5		-3.4295		0.6193		-2.124
					18.96				**17.07**

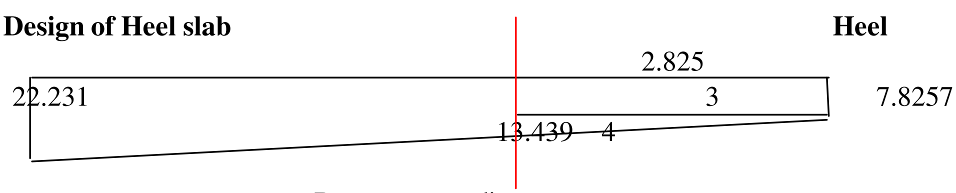
Effective depth at a distance of d eff	=	0.7023	m
Shear at critical section	=	18.96	t
Bending moment at critical section	=	17.07	t-m

$\tan(\beta)$ = 0.404

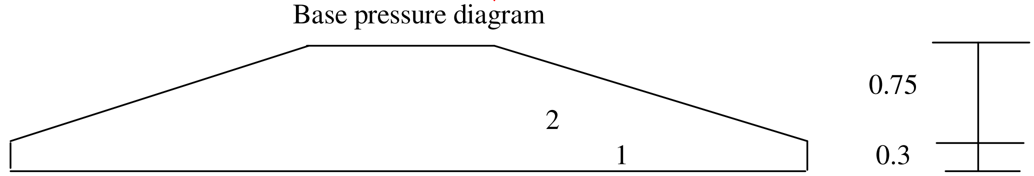
Net shear force	=	9.15	t
Shear stress	=	13.026	t/m^2
% of reinforcement	=	0.2386	
Permissible shear stress	=	26.89	t/m^2

NO SHEAR REIN FORCEMENT REQ

Design of Heel slab **Heel**



Base pressure diagram

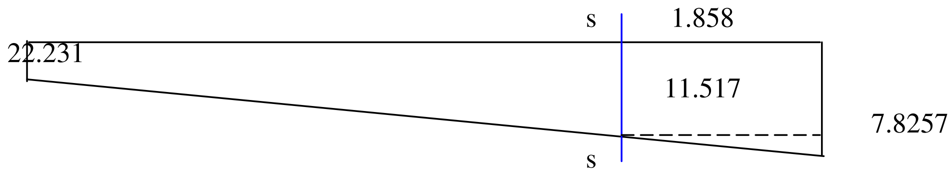


Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		2.034		1.4125		2.873
	2		0.5		2.5425		0.9417		2.3942
	earth				55.498		1.4125		78.39
Upward base pressure									
	3		1		-7.8257		1.4125		-11.054
	4		0.5		-7.9283		0.9417		-7.4658

Net bending moment at face of stem = 65.138 t-m/m
Effective depth required = 0.7601 m
Effective depth Provided at face of stem = 0.9975 m
Area of reinforcement required = 3549 mm^2
Minimum steel req = 1496.3 mm^2
Provide **25** ϕ **125** mm c/c = 3927 mm^2

Shear

Effective depth d of footing = 0.9625 m
Total depth of section at distance d from Abutment = 0.7945 m
Effective depth at distance d from face of Abutment = 0.6945 m



width b = 4.25

Loadings	Element		Area factor		Force		L.A		Moment
Downward loads	1		1		1.3378		0.929		1.2428
	2		0.5		1.6722		0.6193		1.0356
	earth				36.501		0.929		33.909
Upward base pressure									
	3		1		-7.8257		0.929		-7.2701
	4		0.5		-3.4295		0.6193		-2.124
					28.26				**26.79**

Effective depth at a distance of d eff = 0.6945 m
Shear at critical section = 28.26 t
Bending moment at critical section = 26.79 t-m
tan(β) = 0.404
Net shear force = 12.68 t
Shear stress = 18.261 t/m^2
% of reinforcement = 0.5655
Permissible shear stress = 30.478 t/m^2
NO SHEAR REIN FORCEMENT REQ

Dry Condition with L.L											
a) Vertical load and their moments about Abutment Shaft bottom											
II Substructure	**V**		ρ	**P**		e_L	**M_L**		e_T		**M_T**
1 Pedestal											
a Left span	0		2.4	0		0.15	0		0		0
2 Abut cap R	1.6099		2.4	3.8637		0	0		0		0
Abut cap Tr	4.779		2.4	11.47		0	0		0		0
3 Abutment shaft											
up to G.L	10.334		2.4	24.801		0	0		0		0
below G.L	9.8903		2.4	23.737		0	0		0		0
4 Earth pressure							351				
				63.871			**351**				**0**

b) Horizantal Forces and Moments withrespect to Abutment Shaft bottom									
1 Longitudinal Forces at bearing level									
	H_L		**H_T**		e_L	**M_L**	e_T		**M_T**
	4.77		0		9.438	45.038	9.438		0
	4.77		**0**			**45.038**			**0**

Summary *for design of Abutment*

P = 192.02 t

M_L = 414.81 t-m

M_T = 55.355 t-m

H.F.L Condition with L.L						
a) Vertical load and their moments about C/L of Foundation base.						
I						
1 D.L Reaction	**P**	**e_L**	**M_L**	**e_T**	**M_T**	
a Left span	66.978	0.150	10.047	0.000	0.000	
2 S.I.D.L						
a Left span	13.450	0.150	2.018	0.000	0.000	
	80.429		***12.064***		***0.000***	
3 L.L						
a Left span						
70R Wheeled	47.720	0.150	7.158	1.160	55.355	
Class A 2 Lane	49.480	0.150	7.422	0.700	34.636	
	128.149		**19.222**		**55.355**	

II Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1 Pedestal							
a Left span	0.000	2.400	0.000	0.150	0.000	0.000	0.000
2 Abut cap R	2.147	2.400	5.152	0.000	0.000	0.000	0.000
Abut cap Tr	4.779	2.400	11.470	0.000	0.000	0.000	0.000
3 Abut shaft							
up to H.F.L	0.000	2.400	0.000	0.000	0.000	0.000	0.000
below H.F.L	20.224	1.400	28.313	0.000	0.000	0.000	0.000
4 Footing	23.348	1.400	32.688	0.000	0.000	0.000	0.000
5 Earth above footing	118.729	1.000	118.729	0.000	0.000	0.000	0.000
6 Earth Pressure					282.871		
7 Earth wt on heel side	66.079	1.000	66.079	-2.213	-146.23		
			196.351		**136.639**		**0.000**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L	**H_T**	**e_L**	**M_L**	**e_T**	**M_T**
	4.772	0.000	10.488	50.049	10.488	0.000
	4.772	0.000		**50.049**		**0.000**

Summary

P = 324.500 t

M_L = 205.910 t-m

M_T = 55.355 t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{324.500}{30.813} + \frac{205.910}{37.232} + \frac{55.355}{21.826} = 18.598 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{324.500}{30.813} - \frac{205.910}{37.232} - \frac{55.355}{21.826} = 2.465 \quad t/m^2$$

H.F.L Condition with L.L

a) Vertical load and their moments about Abutment Shaft bottom

II	Substructure		V	ρ	P		e_L	M_L		e_T		M_T
1	Pedestal											
a	Left span		0.000	2.400	0.000		0.150	0.000		0.000		0.000
2	Abut cap	R	1.610	2.400	3.864		0.000	0.000		0.000		0.000
	Abut cap	Tr	4.779	2.400	11.470		0.000	0.000		0.000		0.000
3	Abut shaft											
	up to H.F.L		0.000	2.400	0.000		0.000	0.000		0.000		0.000
	below H.F.L		20.224	1.400	28.313		0.000	0.000		0.000		0.000
					43.647			**0.000**				**0.000**

b) Horizantal Forces and Moments withrespect to Abutment Shaft bottom

1 Longitudinal Forces at bearing level

	H_L		H_T	e_L	M_L	e_T	M_T
	4.772		0.000	9.438	45.038	9.438	0.000
Earth pressure					219.644		
	4.772		0.000		264.682		0.000

Summary	*for design of Abutment shaft*			
	P	=	171.795	t
	$\mathbf{M_L}$	=	283.904	t-m
	$\mathbf{M_T}$	=	55.355	t-m

Summary of Loads at Abutment Shaft bottom :

1 DRY condition	**P**		**M_L**		**M_T**
With L.L	192.02		414.81		55.36
2 H.F.L					
With L.L	171.80		283.90		55.36

Dimensions of Substructure & Foundation

1	**Pedestal**								
		Length			=	0.7			
		Width			=	0.55		Volume =	0
		Height			=	0			
2	**Pier cap**								
a	*Top uniform portion*								
		Width			=	0.90			
		Depth			=	0.225		Volume =	1.60988 m^3
		Length			=	7.95			
b	*Top uniform portion*								
		Width			=	0.90			
		Depth			=	0.075		Volume =	0.53663 m^3
		Length			=	7.95			
c	*Bottom trapezoidal portion*								
		Width			=	0.90			
		Depth			=	1.2		Volume =	4.779 m^3
		Length			=	7.95			
	Area at level 215.163 m	= 7.95	x	0.9	=	7.155	m^2		
	Area at level 213.963 m	= 0.9	x	0.9	=	0.81	m^2	=	**6.9255** m^3
	H.F.L								
	Pier cap				=	1.60988	m^3		
					=	0.53663	m^3		
					=	4.779	m^3		
3	**Abutment shaft**								
	Area at abutment shaft bottom				=	3.45062	m^2		
	Area at abutment shaft top				=	1.64485	m^2		
	Average area				=	2.54773	m^2		
	Height of Abutment shaft				=	7.94			
	Height above Ground level				=	4.056	m	Volume =	10.3336
	Height below Ground level				=	3.882	m	Volume =	9.8903
								=	**20.2239** m^3
	Height above H.F.L				=	0.000	m	Volume =	0
	Height below H.F.L				=	7.94	m	Volume =	20.2239
								=	**20.2239** m^3
4	**Pedestal at footing top**		Width		=	1.6			
			Length		=	2.50		Volume =	0 m^3
			Height		=	0			

5 Footing

at foundation level

Width	=	7.250	m
Length	=	4.25	m
Thickness at Root	=	1.05	m
Thickness at Tip	=	0.3	m
	=	23.3484	m^3

Volume of Overburdden earth below ground level

Total volume 7.250 x 4.250 x 4.932 = 151.967 m^3

Net volume below Ground level = 118.729 m^3

Wt of earth on Heel side above ground level = 66.0788 t

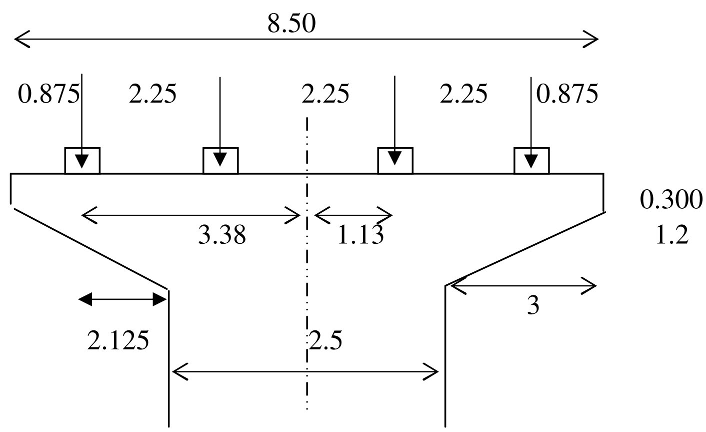
Sectional Properties of Footing

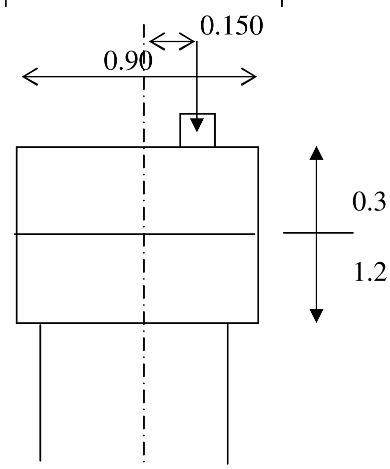
$A = 7.250 \times 4.250 = 30.813\ m^2$

$Z_L = 4.250 \times 8.76042 = 37.232\ m^3$

$Z_T = 7.250 \times 3.01042 = 21.826\ m^3$

DESIGN OF ABUTMENT CAP:

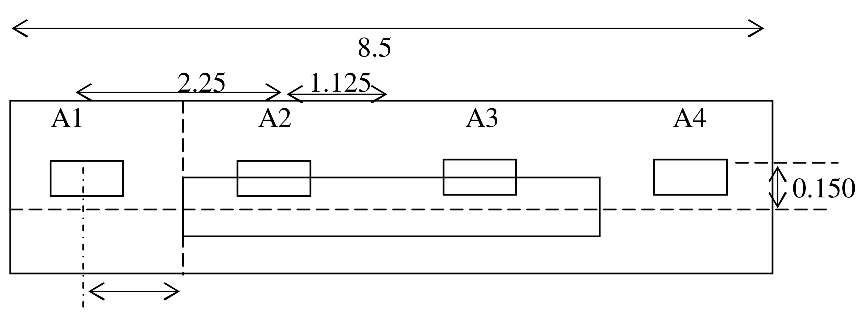




For Outer bearing	a	=	2.125
	D	=	1.50
Available effective depth			
using 25 mm dia of bars	d	=	1.5 - 0.05 - 0.025
		=	1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= **Pier cap designed as Cantilever**



Bearings A1 & A4 are effective.

	A1	
DL	16.745	
SIDL	3.3626	
LL		
70RW	11.93	

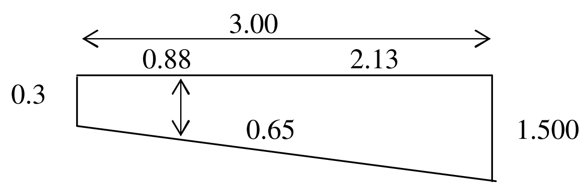
due to Transverse moment A1 = 47.72 x 1.16 = 55.355

$$A1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	A1
DL	16.745
SIDL	3.3626
LL	7.3807
	11.93
	39.418



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 0.9 \times 2.4 = 5.832 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 39.42 x 2.13 = 83.76297 t-m

Due to selfweight of Abutment cap = 5.832 x 1.1667 = 6.804 t-m

Total moment at the face of support = **90.567** t-m

Torsion at outerbearing section:

= 39.418

Eccentricity = 0.15

Moment = 5.9127 t-m

Longitudinal Reinforcement:

M = 90.567 = 90.567 t-m

For, M - 25 & Fe - 415

σcbc = 8.33 N/mm^2

m = 10

σst = 200 N/mm^2

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \dfrac{90.567 \times 10^7}{1.105 \times 900}$

= 954.15 < 1425 O.K

A_{st} required $= \dfrac{90.567}{200} \times 0.902 \times 1425$

= 3523.2002 mm^2

Provide 6 nos 20 mm dia bars in 2 layers 3769.9 mm^2

This reinforcement will provided in full length of Abutment cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 39.418

= 39.418 t

Downward Load due to self wt of Abutment cap = 5.832 t

Total downward Load = 45.25 t

After shear correction, S.F $= V - \dfrac{M \tan\beta}{d}$

$= 45.25 - \dfrac{90.567 \times 0.4}{1.425}$

= 19.827561 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

b $= \dfrac{0.9 + 0.9}{2}$ = 0.9 m

$V_e = V + 1.6 \times \dfrac{T}{b}$ = 30.339 t

Shear stress $= \dfrac{30.339 \times 10000}{900 \times 1425}$ = 0.2366 N/mm^2

Area of tension reinforcement = 0.294 %

τ_c = 0.2711 N/mm^2

Provide minimum shear

EARTH PRESSURE CALCULATION

Formation level	=	216.189 m
Founding level	=	204.975 m
Low Water Level	=	209.907 m
Highest flood level	=	214.172 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		=	2.50 m

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

* (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 11.21 = 6 t/m^2

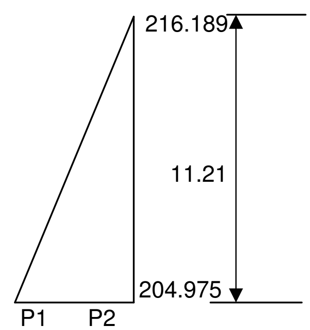
P2 = 0.2973 x 1.0 x 0.00 = 0 t/m^2

F1 = 0.5 x 6.001 x 11.21 x 2.5 x cos 0.3

= 79.051 t say 80 t

M = 79.051 x 0.42 x 11.21

= 372 t-m



CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

P4 = 0.2973 x 1.8 x 2.02 = 1.08 t/m^2

P5 = 0.2973 x 1.0 x 9.20 = 2.73 t/m^2

F4 = 0.5 x 1.079 x 2.02 x 2.5 x cos 0.3

= 2.5574 t

F5 = 0.5 x 2.734 x 9.20 x 2.5 x cos 0.3

= 29.54 t

F6 = 1.08 x 9.20 x 2.5 x cos 0.3

= 23.32 t

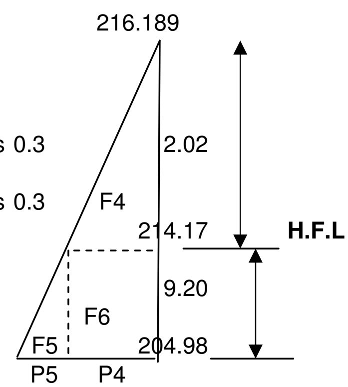
Total force, F = 2.5574 + 29.54 + 23.32

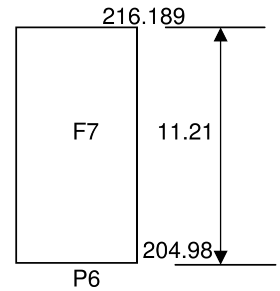
= 55 t

M = 2.5574 x (0.42 x 2.02 + 9.20)

+ 29.54 x 0.33 x 9.20 + 23.3 x $\frac{9.20}{2}$

= 223 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 11.21 x 3 x cos 0.3

= 17 t

M = 17 x $\frac{10.61}{2}$ = 90 t-m

H.F.L

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

F7 = 0.6422 x 2.02 x 3 x cos 0.3 = 3.043011 t

= 0.3568 x 9.20 x 3 x cos 0.3 = 7.708525 t

= 10.75154

M = 3.043 + 7.708524503 x 5.607 = 60.28386 tm

EARTH PRESSURE CALCULATION

Formation level	=	216.189 m
Abutment shaft bottom level	=	206.025 m
Low Water Level	=	209.907 m
Highest flood level	=	214.172 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

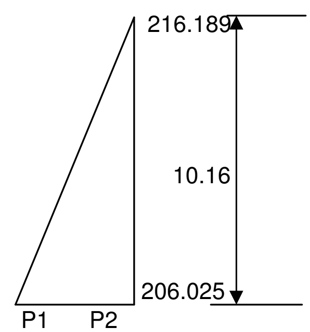
Angle of internal friction,	ϕ	=	30 ^{o}
Angle of friction between soil and concrete,	δ	=	20 ^{o}
Surcharge angle	i	=	0 ^{o}
Angle of wall face with horizontal	α	=	90 ^{o}
Bulk density of earth	γ	=	1.8 t/m^3
Submerged density of earth	γ_{sub}	=	1.0 t/m^3
Width of abutment		=	2.50 m

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

* (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 10.16 = 5.44 t/m^2

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

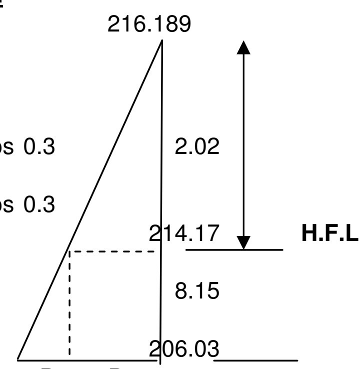
F1 = 0.5 x 5.439 x 10.16 x 2.5 x cos 0.3

= 64.941 t say 65 t

M = 64.941 x 0.42 x 10.16

= 277 t-m

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION



P4 = 0.2973 x 1.8 x 2.02 = 1.08 t/m^2

P5 = 0.2973 x 1.0 x 8.15 = 2.42 t/m^2

F4 = 0.5 x 1.079 x 2.02 x 2.5 x cos 0.3

= 2.5574 t

F5 = 0.5 x 2.422 x 8.15 x 3 x cos 0.3

= 23.18 t

F6 = 1.08 x 8.15 x 2.5 x cos 0.3

= 20.66 t

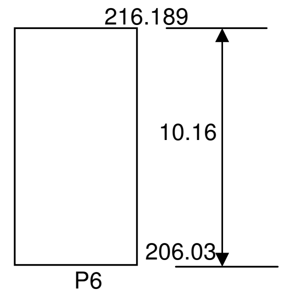
Total force, F = 2.5574 + 23.18 + 20.66

= 46 t

M = 2.5574 x (0.42 x 2.02 + 8.15)

+ 23.18 x 0.33 x 8.15 + 20.7 x $\frac{8.15}{2}$

= 169 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 10.16 x 3 x cos 0.3

= 15 t

M = 15 x $\frac{9.56}{2}$ = 73 t-m

H.F.L

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

F7 = 0.6422 x 2.02 x 3 x cos 0.3 = 3.043011 t

= 0.3568 x 8.15 x 3 x cos 0.3 = 6.82846 t

= 9.871471

M = 3.043 + 6.828460272 x 5.082 = 50.16682 tm

```
    ABUT SHAFT .case 1 L.W.L

    Depth of Section             =     1.600 m
    Width of Section             =     2.500 m

along width-compression face- no of bar:  18   tension face- no of bar:  18
Dia (mm)                                 20                              32
Cover (cm)                             7.50                             7.5
along depth-compression face- no of bar:   6   tension face- no of bar:   6
Dia (mm)                                 16                              16
Cover  (cm)                            7.50                             7.5

    Modular Ratio : Compression  =      10.0
    Modular Ratio : Tension      =      10.0
    Allowable Concrete Stress    =     85.00 Kg/cm^2
    Allowable Steel Stress       =   2000.00 Kg/cm^2

    Axial Load                   =   192.020 T
    Mxx                          =   414.810 Tm
    Myy                          =    55.360 Tm


    Intercept of Neutral axis : X axis : =   9.879 m
                              : y axis : =    .543 m

              Concrete Stress Governs Design

    Stress in Concrete due to Loads =   72.81 Kg/cm^2
    Stress in Steel    due to Loads = 1496.34 Kg/cm^2
    Percentage of Steel             =     .56 %

    ABUT SHAFT .case 2 H.F.L

    Depth of Section             =     1.600 m
    Width of Section             =     2.500 m

    Modular Ratio : Compression  =      10.0
    Modular Ratio : Tension      =      10.0
    Allowable Concrete Stress    =     85.00 Kg/cm^2
    Allowable Steel Stress       =   2000.00 Kg/cm^2

    Axial Load                   =   171.800 T
    Mxx                          =   284.270 Tm
    Myy                          =    55.360 Tm


    Intercept of Neutral axis : X axis : =   7.574 m
                              : y axis : =    .614 m

              Concrete Stress Governs Design


    Stress in Concrete due to Loads =   52.35 Kg/cm^2
    Stress in Steel    due to Loads =  944.22 Kg/cm^2
    Percentage of Steel             =     .56 %
```

```
INPUT FILE: CLASS A.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2 3    PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    20. LOAD GENERATION 100
    21. TYPE 1 -18.8 0. 0. XINC .2
    22. PERFORM ANALYSIS
    23. LOAD LIST 74
    24. PRINT SUPPORT REACTION
        SUPPORT  REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


     2   74      0.00     24.74      0.00      0.00      0.00      0.00
     3   74      0.00     11.66      0.00      0.00      0.00      0.00


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


    25. FINISH
```

```
INPUT FILE: 70RW.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2 3    PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    20. LOAD GENERATION 175
    21. TYPE 1 -13.4 0. 0. XINC .2
    22. PERFORM ANALYSIS
    23. LOAD LIST 30
    24. PRINT SUPPORT REACTION
        SUPPORT  REACTION

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

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


      2   30      0.00     47.72      0.00      0.00      0.00      0.00
      3   30      0.00     20.28      0.00      0.00      0.00      0.00


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


    25. FINISH
```

```
INPUT FILE: 9.2 M SPAN SIDL.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;3 0.4 0.0 0.0 10 8.4 0.0 0.0;11 8.8 0.0 0.0
     9. 12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22180.00.0;2318.380.0 0.0
    10. 24 18.4 0.0 0.0
    11. REPEAT ALL 1 0.0 0.0 1.0
    12. REPEAT 3 0.0 0.0 2.0
    13. REPEAT 1 0.0 0.0 1.0
    14. MEMBER INCIDENCES
    15. 1 1 2 23 1 1
    16. REPEAT 5 23 24
    17. 139 1 25 143 1 24
    18. REPEAT 23 5 1
    19. CONSTANTS
    20. E 2.7386E+006
    21. POISSON 0.15
    22. DENSITY 2.4 MEMB  1 TO 138
    23. DENSITY 0.0001 MEMB 139 TO 258
    24. MEMBER PROPERTY INDIAN
    25. 2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
    26. 25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
    27. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
    28. 1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
    29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    31. SUPPORTS
    32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
    33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
    34. 35 59 83 107 PINNED
    35. 46 70 94 118 PINNED
    36. MEMBER RELEASE
    37. 1 24 47 93 116 END MZ
    38. 23 46 92 115 138 START MZ
    39. 12 35 58 81 104 127 START MZ
    40. 12 35 58 81 104 127 END MZ
    41. LOAD 1 SIDL
    42. *** CRASH BARRIER
    43. MEMBER LOAD
    44. 25 TO 45 UNI GY -1.0
    45. 94 TO 114 UNI GY -1.0
    46. 25 TO 45 UMOM GX -1.0
    47. 94 TO 114 UMOM GX 1.0
    48. LOAD 2 WEARINGCOAT
    49. MEMBER LOAD
    50. 25 TO 114 UNI GY -0.2464
    51. PERFORM ANALYSIS
```

```
    52. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70
94 118
 SUPPORT  REACTION LIST     27       
     STAAD FLOOR                                              -- PAGE NO.    3

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

     27    1      0.00      7.42      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     51    1      0.00     -2.84      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     75    1      0.00     -2.84      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     99    1      0.00      7.42      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     38    1      0.00      6.97      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     62    1      0.00     -2.37      0.00      0.00      0.00      0.00
           2      0.00      1.14      0.00      0.00      0.00      0.00
     86    1      0.00     -2.37      0.00      0.00      0.00      0.00
           2      0.00      1.14      0.00      0.00      0.00      0.00
    110    1      0.00      6.97      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     35    1      0.00      7.19      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     59    1      0.00     -2.59      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     83    1      0.00     -2.59      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
    107    1      0.00      7.19      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     46    1      0.00      7.52      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     70    1      0.00     -2.94      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
     94    1      0.00     -2.94      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00
    118    1      0.00      7.52      0.00      0.00      0.00      0.00
           2      0.00      1.13      0.00      0.00      0.00      0.00


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


    53. FINISH
```

```
INPUT FILE: 9.2 M SPAN DL.STD
     1. STAAD FLOOR
     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER MTON
     7. JOINT COORDINATES
     8. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;3 0.4 0.0 0.0 10 8.4 0.0 0.0;11 8.8 0.0 0.0
     9. 12 9.18 0.0 0.0;139.22 0.0 0.0;14 9.6 0.00.0 22 18 0.00.0;2318.380.0 0.0
    10. 24 18.4 0.0 0.0
    11. REPEAT ALL 1 0.0 0.0 1.0
    12. REPEAT 3 0.0 0.0 2.0
    13. REPEAT 1 0.0 0.0 1.0
    14. MEMBER INCIDENCES
    15. 1 1 2 23 1 1
    16. REPEAT 5 23 24
    17. 139 1 25 143 1 24
    18. REPEAT 23 5 1
    19. CONSTANTS
    20. E 2.7386E+006
    21. POISSON 0.15
    22. DENSITY 2.4 MEMB  1 TO 138
    23. DENSITY 0.0001 MEMB 139 TO 258
    24. MEMBER PROPERTY INDIAN
    25. 2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
    26. 25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
    27. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
    28. 1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
    29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
    31. SUPPORTS
    32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
    33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
    34. 35 59 83 107 PINNED
    35. 46 70 94 118 PINNED
    36. MEMBER RELEASE
    37. 1 24 47 93 116 END MZ
    38. 23 46 92 115 138 START MZ
    39. 12 35 58 81 104 127 START MZ
    40. 12 35 58 81 104 127 END MZ

41. LOAD 1 SELFWEIGHT
    42. SELF Y -1
    43. PERFORM ANALYSIS
    44. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70
94 118
```

SUPPORT REACTION LIST 27

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = FLOOR

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
27	1	0.00	16.77	0.00	0.00	0.00	0.00
51	1	0.00	12.24	0.00	0.00	0.00	0.00
75	1	0.00	12.24	0.00	0.00	0.00	0.00
99	1	0.00	16.77	0.00	0.00	0.00	0.00
38	1	0.00	16.42	0.00	0.00	0.00	0.00
62	1	0.00	12.60	0.00	0.00	0.00	0.00
86	1	0.00	12.60	0.00	0.00	0.00	0.00
110	1	0.00	16.42	0.00	0.00	0.00	0.00
35	1	0.00	16.58	0.00	0.00	0.00	0.00
59	1	0.00	12.44	0.00	0.00	0.00	0.00
83	1	0.00	12.44	0.00	0.00	0.00	0.00
107	1	0.00	16.58	0.00	0.00	0.00	0.00
46	1	0.00	16.86	0.00	0.00	0.00	0.00
70	1	0.00	12.16	0.00	0.00	0.00	0.00
94	1	0.00	12.16	0.00	0.00	0.00	0.00
118	1	0.00	16.86	0.00	0.00	0.00	0.00

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

45. FINISH

DESIGN OF SUBSTRUCTURE (PIER)

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete	=	M - 25	
Pier Cap	=	M - 25	
Grade of reinforcement steel	=	Fe - 415	
Centre to Centre distance between Expansion joints	=	9.200	m
Centre to Centre distance of Bearing	=	8.800	m
Depth of superstructure	=	670	mm
Thickness of wearing coat	=	56.00	mm
Formation level along ℄ of carriage way	=	216.189	m
Soffit level	=	215.438	
Pedestal top level	=	215.463	
Height of bearing and Pedestal	=	0.025	m
L.W.L./Bed level	=	209.300	m
H.F.L	=	214.172	m
M.S.L	=	205.452	m
Founding Level	=	204.800	m
Pier cap top level	=	215.438	m
Gross safe bearing capacity	=	29.715	t/m^2

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

(b) Class 70R wheeled

Bearing : neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:

1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000

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

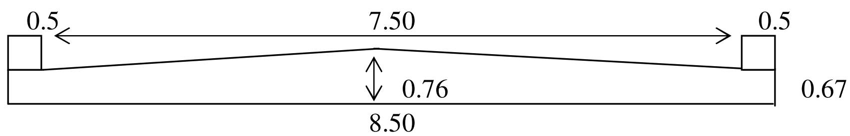
For Class A 2 lane : $\mathbf{F_h} =$ (0.2×59.2) = 11.836 t

For class 70R wheeled : $\mathbf{F_h} =$ (0.2×66.1) = 13.210 t

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
Class A 2 lane	**5.92**
70R	**6.61**
Span dislodged condition	**0.00**

Dead load



Dead load of slab	=	14.561	t/m
Wearing coat	=	0.924	t/m
Crashbarrier	=	1	t/m
Dead load reaction	=	66.978	t
Sidl reaction	=	13.45	t

H.F.L Condition with L.L					
a) Vertical load and their moments about C/L of Foundation base.					
I					
1 D.L Reaction	**P**	**e_L**	**M_L**	**e_T**	**M_T**
a Left span	66.978	-0.400	-26.791	0.000	0.000
b Right span	66.978	0.400	26.791	0.000	0.000
2 S.I.D.L					
a Left span	13.450	-0.400	-5.380	0.000	0.000
b Right span	13.450	0.400	5.380	0.000	0.000
	160.857		***0.000***		***0.000***
3 L.L Max Reaction case					
a Left span					
70R Wheeled	36.000	-0.400	-14.400	1.160	41.760
Class A 2 Lane	33.600	-0.400	-13.440	0.700	23.520
b Right span					
70R Wheeled	30.050	0.400	12.020	1.160	34.858
Class A 2 Lane	25.580	0.400	10.232	0.700	17.906
4 L.L Max Long.Moment case					
a Left span					
70R Wheeled	0.000	-0.400	0.000	1.160	0.000
Class A 2 Lane	6.880	-0.400	-2.752	0.700	4.816
b Right span					
70R Wheeled	47.150	0.400	18.860	1.160	54.694
Class A 2 Lane	47.740	0.400	19.096	0.700	33.418

Max Reaction case:	**226.907**	**1.420**	**76.618**
Max Long. moment case:	**208.007**	**18.860**	**54.694**

II Substructure	**V**	**ρ**	**P**	**e_L**	**M_L**	**e_T**	**M_T**
1 Pedestal							
a Left span	0.000	2.400	0.000	-0.400	0.000	0.000	0.000
b Right span	0.000	2.400	0.000	0.400	0.000	0.000	0.000
2 Pier cap R	2.550	2.400	6.120	0.000	0.000	0.000	0.000
Pier cap Tr	5.700	2.400	13.680	0.000	0.000	0.000	0.000
3 Pier shaft							
up to H.F.L	0.000	2.400	0.000	0.000	0.000	0.000	0.000
below H.F.L	15.413	1.400	21.579	0.000	0.000	0.000	0.000
4 Footing	23.224	1.400	32.513	0.000	0.000	0.000	0.000
5 Earth above footing	113.752	1.000	113.752	0.000	0.000	0.000	0.000
6 Water current			0.000	0.000	17.615	0.000	7.040
			187.643		**17.615**		**7.040**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L	**H_T**	**e_L**	**M_L**	**e_T**	**M_T**
	6.605	0.000	10.663	70.429	10.638	0.000
	6.605	0.000		**70.429**		**0.000**

Summary

		Max Reaction case		Max Long.moment case
P	=	414.551	t	395.651
M_L	=	89.464	t-m	106.904
M_T	=	83.658	t-m	61.734

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{414.551}{31.900} + \frac{89.464}{30.837} + \frac{83.658}{29.242} = 18.757 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{414.551}{31.900} - \frac{89.464}{30.837} - \frac{83.658}{29.242} = 7.233 \quad t/m^2$$

Max Long

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{395.651}{31.900} + \frac{106.904}{30.837} + \frac{61.734}{29.242} = 17.981 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{395.651}{31.900} - \frac{106.904}{30.837} - \frac{61.734}{29.242} = 6.825 \quad t/m^2$$

H.F.L Condition with L.L

a) Vertical load and their moments about Pier Shaft bottom

II Substructure	V	ρ	P		e_L	M_L		e_T		M_T
1 Pedestal										
a Left span	0.000	2.400	0.000		-0.400	0.000		0.000		0.000
b Right span	0.000	2.400	0.000		0.400	0.000		0.000		0.000
2 Pier cap R	2.550	2.400	6.120		0.000	0.000		0.000		0.000
Pier cap Tr	5.700	2.400	13.680		0.000	0.000		0.000		0.000
3 Pier shaft										
up to H.F.L	0.000	1.400	0.000		0.000	0.000		0.000		0.000
below H.F.L	15.413	1.400	21.579		0.000	0.000		0.000		0.000
4 Water current						17.593				6.976
			41.379			**17.593**				**6.976**

b) Horizantal Forces and Moments withrespect to Pier Shaft bottom

1 Longitudinal Forces at bearing level

	H_L		H_T		e_L		M_L	e_T		M_T
	6.605		0.000		9.613		63.494	9.588		0.000
	6.605		0.000				63.494			0.000

Gross Pressure Diagram

Summary	*for design of Pier*		Max Reaction case			Max Long.moment case	
	P	=	268.286	t		249.386	t
	M_L	=	82.507	t-m		99.947	t-m
	M_T		83.594	t-m		61.670	t-m

H.F.L Condition One.Span dislodged

a) Vertical load and their moments about C/L of Foundation base.

II	Substructure	V		ρ	P		e_L	M_L		e_T		M_T
1	Pedestal											
a	Left span	0.000		2.400	0.000		-0.400	0.000		0.000		0.000
b	Right span	0.000		2.400	0.000		0.400	0.000		0.000		0.000
2	Pier cap R	2.550		2.400	6.120		0.000	0.000		0.000		0.000
	Pier cap Tr	5.700		2.400	13.680		0.000	0.000		0.000		0.000
3	Pier shaft											
	up to H.F.L	0.000		2.400	0.000		0.000	0.000		0.000		0.000
	below H.F.L	15.413		1.400	21.579		0.000	0.000		0.000		0.000
4	Footing	23.224		1.400	32.513		0.000	0.000		0.000		0.000
5	Earth above footing	113.752		1.000	113.752		0.000	0.000		0.000		0.000
6	Water current				0.000		0.000	17.615		0.000		7.040
					187.643			**17.615**				**7.040**

Summary				
	P	=	348.501	t
	M_L	=	44.407	t-m
	M_T	=	7.040	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{348.501}{31.900} + \frac{44.407}{30.837} + \frac{7.040}{29.242} = 12.606 \quad t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{348.501}{31.900} - \frac{44.407}{30.837} - \frac{7.040}{29.242} = 9.244 \quad t/m^2$$

H.F.L Condition One.Span dislodged
a) Vertical load and their moments about Abutment Shaft bottom.

II Substructure	V		ρ	P		e_L	M_L		e_T		M_T
1 Pedestal											
a Left span	0.000		2.400	0.000		-0.400	0.000		0.000		0.000
b Right span	0.000		2.400	0.000		0.400	0.000		0.000		0.000
2 Pier cap R	2.550		2.400	6.120		0.000	0.000		0.000		0.000
Pier cap Tr	5.700		2.400	13.680		0.000	0.000		0.000		0.000
3 Pier shaft											
up to H.F.L	0.000		2.400	0.000		0.000	0.000		0.000		0.000
below H.F.L	15.413		1.400	21.579		0.000	0.000		0.000		0.000
4 Water current							17.593				6.976
				41.379			**17.593**				**6.976**

Summary	*for design of Pier*			
	P	=	202.236	t
	M_L	=	44.384	t-m
	M_T	=	6.976	t-m

Dry Condition with L.L					
a) Vertical load and their moments about C/L of Foundation base.					
I					
1 D.L Reaction	**P**	**e_L**	**M_L**	**e_T**	**M_T**
a Left span	66.98	-0.4	-26.791	0	0
b Right span	66.98	0.4	26.791	0	0
2 S.I.D.L					
a Left span	13.45	-0.4	-5.3802	0	0
b Right span	13.45	0.4	5.3802	0	0
	160.86		**0**		**0**
4 **L.L Max Reaction case**					
a Left span					
70R Wheeled + Class A	36	-0.4	-14.4	1.16	41.76
Class A 3 Lane	33.6	-0.4	-13.44	0.7	23.52
b Right span					
70R Wheeled + Class A	30.05	0.4	12.02	1.16	34.858
Class A 3 Lane	25.58	0.4	10.232	0.7	17.906
4 **L.L Max Longitudinal Moment case**					
a Left span					
70R Wheeled + Class A	0	-0.4	0	1.16	0
Class A 3 Lane	6.88	-0.4	-2.752	0.7	4.816
b Right span					
70R Wheeled + Class A	47.15	0.4	18.86	1.16	54.694
Class A 3 Lane	47.74	0.4	19.096	0.7	33.418
Max Reaction case	**226.91**		**1.42**		**76.618**
Max Longitudinal Moment	**208.01**		**18.86**		**54.694**

II Substructure	**V**	**ρ**	**P**	**e_L**	**M_L**	**e_T**	**M_T**
1 Pedestal							
a Left span	0	2.4	0	-0.4	0	0	0
b Right span	0	2.4	0	0.4	0	0	0
2 Pier cap R	2.55	2.4	6.12	0	0	0	0
Pier cap Tr	5.7	2.4	13.68	0	0	0	0
3 Pier shaft							
up to G.L	8.8386446	2.4	21.213	0	0	0	0
below G.L	6.574671	2.4	15.779	0	0	0	0
4 Footing	23.22375	2.4	55.737	0	0	0	0
5 Earth above footing	113.75158	2	227.5	0	0	0	0
			340.03		**0**		**0**

b) Horizantal Forces and Moments withrespect to Base										
1 Longitudinal Forces at bearing level										
	H_L		**H_T**		**e_L**		**M_L**		**e_T**	**M_T**
	6.61		0		10.663		70.429		10.638	0
	6.61		0				**70.429**			**0**

Summary

		Max Reaction		Max Longitudinal
P	=	566.94	t	548.04
M_L	=	71.849	t-m	89.289
M_T	=	76.618	t-m	54.694

A		=	31.9
Z_L		=	30.837
Z_T		=	29.242

Max reaction case

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{566.93952}{31.9} + \frac{71.849}{30.837} + \frac{76.618}{29.242} = 22.723 \ t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{566.93952}{31.9} - \frac{71.849}{30.837} - \frac{76.618}{29.242} = 12.822 \ t/m^2$$

Max Longitudinal moment case

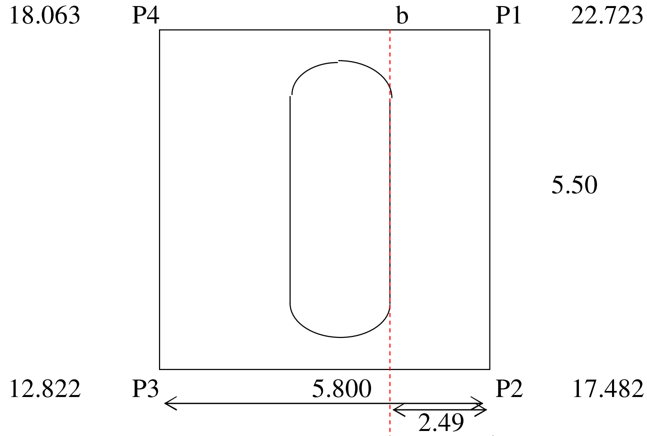
$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

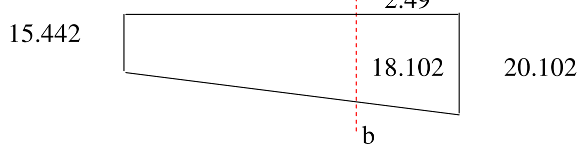
$$= \frac{548.03952}{31.9} + \frac{89.289}{30.837} + \frac{54.694}{29.242} = 21.946 \ t/m^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

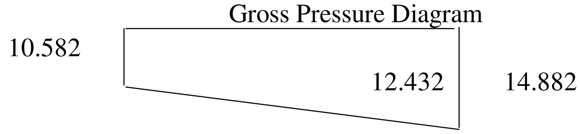
$$= \frac{548.03952}{31.9} - \frac{89.289}{30.837} - \frac{54.694}{29.242} = 12.414 \ t/m^2$$

Design of Footing in Transverse direction



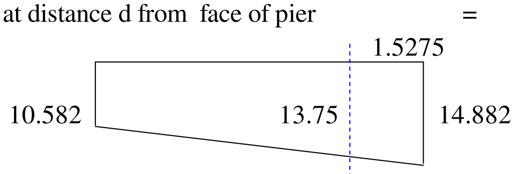


Gross Pressure Diagram



Net Pressure diagram b

Bending moment at the face of pier b-b	=	**239.82**	t-m
Footing is checked for shear at distance of d from face pier			
Effective depth d of footing	=	0.9625	m
Total depth of section at distance d from pier	=	0.7601	m
Effective depth at distance d from face of pier	=	0.6691	m



Bending moment at a distance of d from face of pier	=	42.06	t-m
Shear force at a distance d from face of pier	=	120.27	t
Net shear force at a distance d from face of pier (after correction)	=	**101.34**	t

Design for Flexure:

M = 25 Fe = 415

Permissible compressive stress	σ_{cbc}	=	8.33	N/mm^2
Permissible tensile stress	σ_{st}	=	200	N/mm^2
	m	=	10	
	k	=	0.29	
	j	=	0.9	
	Q	=	1.1053	

width b = 5.50

Effective depth required d = 628.08 mm
Effective depth provided = 962.5 mm
A_{st} Required = 2511.3 mm^2

Provide 6 nos 25 dia bars /m width

Check for shear

Shear stress at effective depth from pier = 27.538 t/m^2
% of steel provided = 0.2392
Permissible shear stress = 23.46 t/m^2

Minimum Shear Reinforcement :

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

$$\frac{A_{sw}}{s} = \frac{0.4 \times b}{0.87 \times f_y} = 6.0933 \; cm^2/m$$

Provide 2 Legged 10 @ 200 = 7.854 cm^2/m

Dry Condition with L.L

a) Vertical load and their moments about Pier Shaft bottom

II Substructure	V		ρ	P		e_L	M_L		e_T	M_T
1 Pedestal										
a Left span	0		2.4	0		-0.4	0		0	0
b Right span	0		2.4	0		0.4	0		0	0
2 Pier cap R	2.55		2.4	6.12		0	0		0	0
Pier cap Tr	5.7		2.4	13.68		0	0		0	0
3 Pier shaft										
up to G.L	8.8386446		2.4	21.213		0	0		0	0
below G.L	6.574671		2.4	15.779		0	0		0	0
				56.792			**0**			**0**

b) Horizantal Forces and Moments withrespect to PierShaft bottom

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	6.61	0	9.613	63.494	9.588	0
	6.61	**0**		**63.494**		**0**

Summary	*for design of Pier*		Max Reaction case		Max Longitudinal moment
	P	=	283.7	t	264.8
	M_L	=	64.914	t-m	82.354
	M_T	=	76.618	t-m	54.694

Dry Condition One.Span dislodged

a) Vertical load and their moments about C/L of Foundation base.

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	0.00	0	10.663	0	10.638	0
	0.00	0		**0**		**0**

Summary				
	P	=	500.89	t
	M_L	=	26.791	t-m
	M_T	=	0	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{500.88952}{31.9} + \frac{26.791}{30.837} + \frac{0}{29.242} = 16.571 \text{ t/m}^2$$

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{500.88952}{31.9} - \frac{26.791}{30.837} - \frac{0}{29.242} = 14.833 \text{ t/m}^2$$

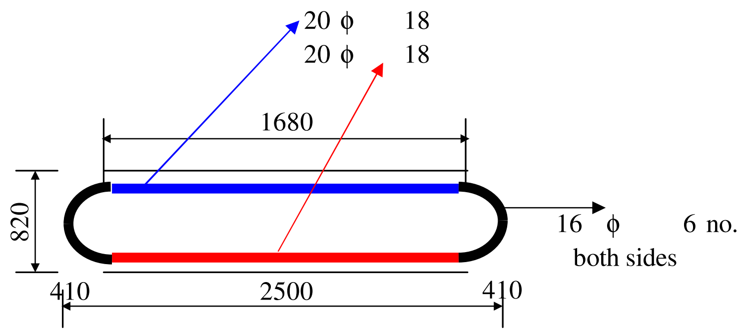
Dry Condition One.Span dislodged

a) Vertical load and their moments about Abutment Shaft bottom.

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	0.00	0	9.613	0	9.588	0

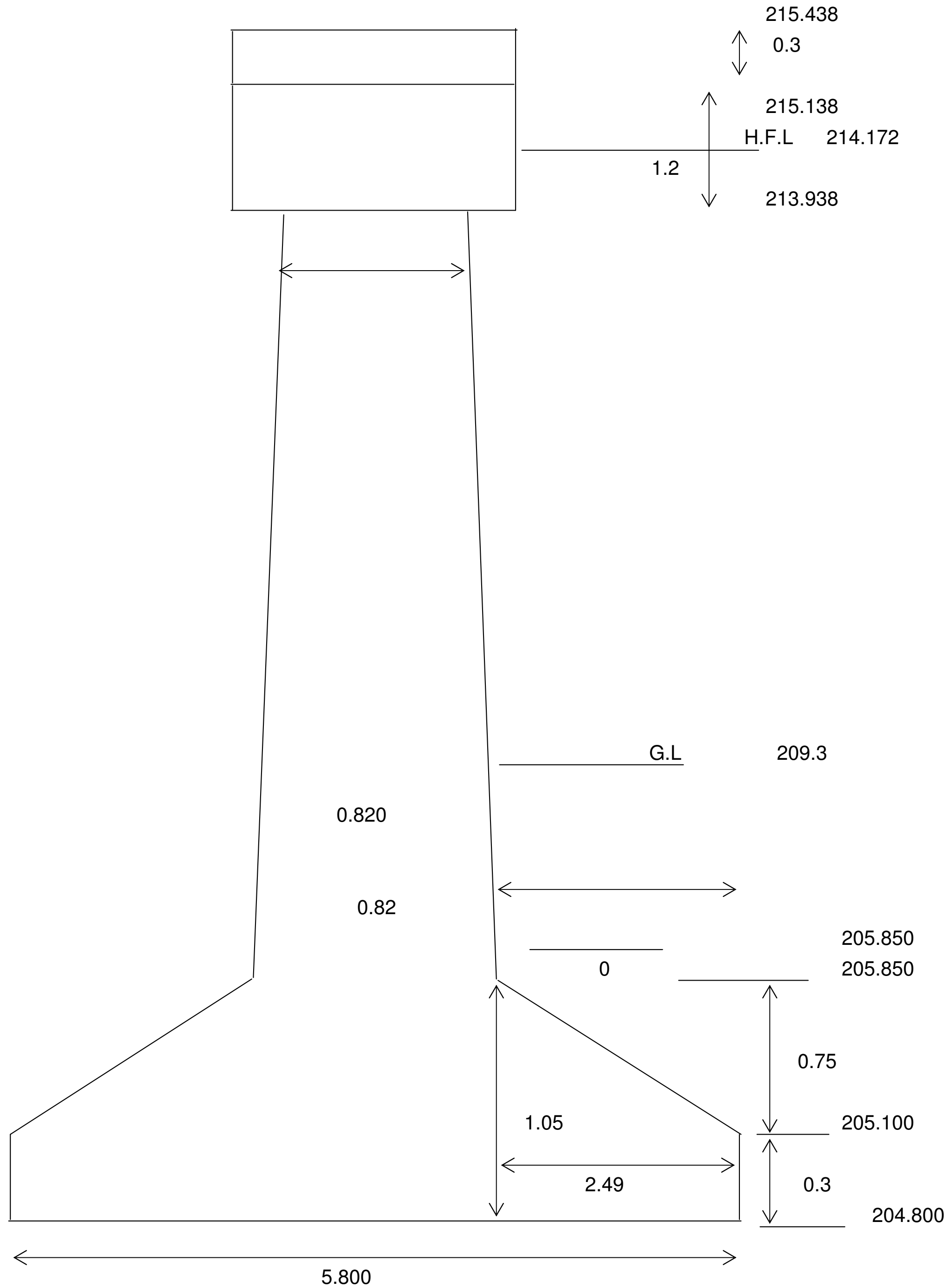
Summary	*for design of Pier*			
	P	=	217.65	t
	M_L	=	26.791	t-m
	M_T	=	0	t-m



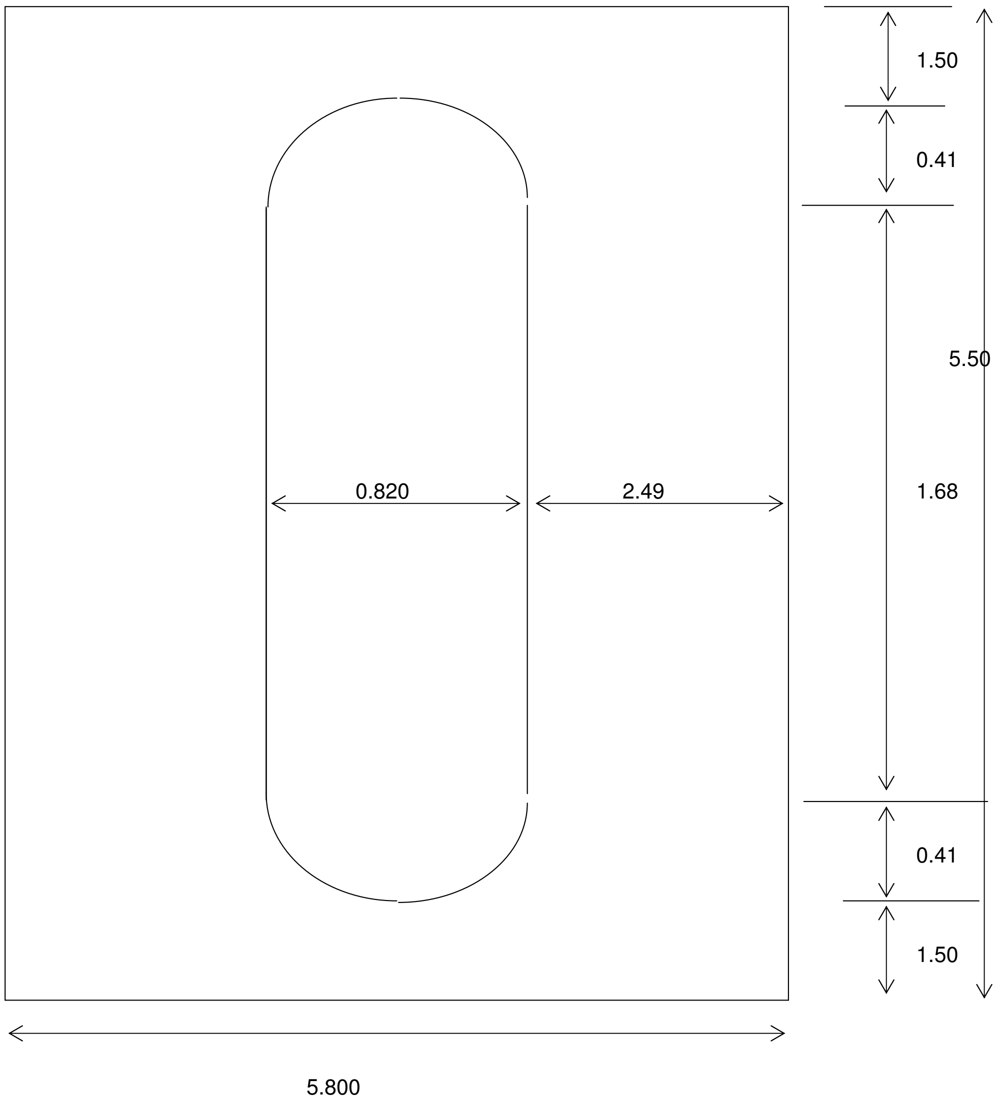
PLAN : PIER SHAFT

Summary of Loads at Pier Shaft bottom :

	P	M_L	M_T
1 DRY condition			
With L.L	283.70	64.91	76.62
Span dislodged	217.65	26.79	0.00
2 H.F.L			
With L.L	268.29	82.51	83.59
Span dislodged	202.24	44.38	6.98



215.438
0.3
215.138
H.F.L 214.172
1.2
213.938
G.L 209.3
0.820
0.82
205.850
0
205.850
0.75
1.05
205.100
2.49
0.3
204.800
5.800



Dimensions of Substructure & Foundation

1	Pedestal											
				Length			=	0.7				
				Width			=	0.55		Volume =	0	
				Height			=	0				
2	Pier cap											
a	Top uniform portion											
				Width			=	1.00				
				Depth			=	0.3		Volume =	2.55	m^3
				Length			=	8.5				
b	Bottom trapezoidal portion											
				Width			=	1				
				Depth			=	1.2		Volume =	5.7	m^3
				Length			=	8.5				
	Area at level	215.138 m	=	8.5	x	1	=	8.5	m^2			
	Area at level	213.938 m	=	1	x	1	=	1	m^2	=	**8.25**	m^3

3 Pier shaft								
Dimension of pier shaft in longitudinal direction		=	0.82					
Dimension of pier shaft in Transverse direction	R	=	1.68		Area	=	1.91	m^2
	C	=	0.41					
Height of pier shaft		=	8.09					
Height above Ground level		=	4.638	m	Volume	=	8.83864	
Height below Ground level		=	3.450	m	Volume	=	6.57467	
						=	**15.4133**	m^3
Height above H.F.L		=	0.000	m	Volume	=	0	
Height below H.F.L		=	8.09	m	Volume	=	15.4133	
						=	**15.4133**	m^3
4 Pedestal at footing top	Width	=	0.82					
	Length	=	5.50		Volume	=	0	m^3
	Height	=	0					
5 Footing								
at foundation level								
	Width	=	5.800	m				
	Length	=	5.50	m				
	Thickness at Root	=	1.05	m				
	Thickr Gross Pressure Diagram	=	0.3	m				
					Volume	=	23.2238	m^3

Net Pressure diagram

Volume of Overburdden earth below ground level

Total volume 5.800 x 5.500 x 4.500 = 143.55 m^3

Net volume below Ground level = 113.752 m^3

Sectional Properties of Footing

A = 5.800 x 5.500 = 31.900 m^2

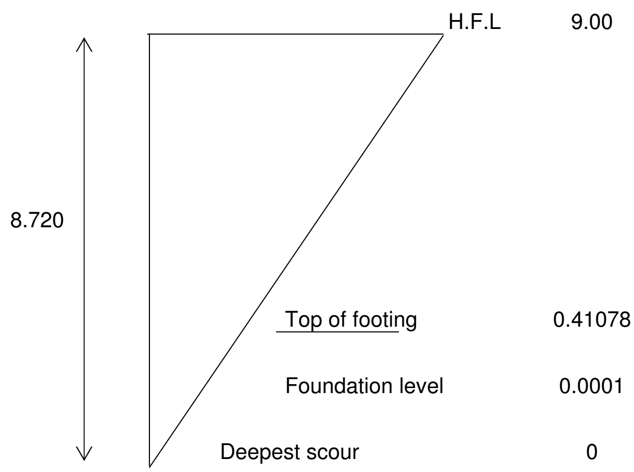
Z_L = 5.500 x 5.60667 = 30.837 m^3

Z_T = 5.800 x 5.04167 = 29.242 m^3

Forces and Moments due to Watercurrent Force

Intensity of water current pressure = 52 k V^2 kg/m^2

Assuming 20 degree variation in water current direction



Water current Pressure in Transverse direction

	R.L	k	Pressure = 52 k V^2 COS 20
H.F.L	214.172	0.66	290.252257
Pier Shaft bottom	205.850	0.66	13.2477521
Top of footing	205.850	1.5	30.1085275
Bottom of footing	204.800	1.5	0.0073296

Water current Force in Transverse direction

Component	Length	Height	Force	HT of force	Moment
	m	m	t		
Pier	0.82	8.322	1.03555	6.73646	6.97593 t-m
Top of footing	5.800	1.050	0.0917	0.69991	0.06418 t-m

Water Current Pressure in Longtudinal direction

	R.L	k	Pressure = 52 k V^2 SIN 20
H.F.L	214.172	1.5	240.098141
Pier Shaft bottom	205.850	1.5	10.9586078
Top of footing	205.850	1.5	10.9586078
Bottom of footing	204.800	1.5	0.00266776

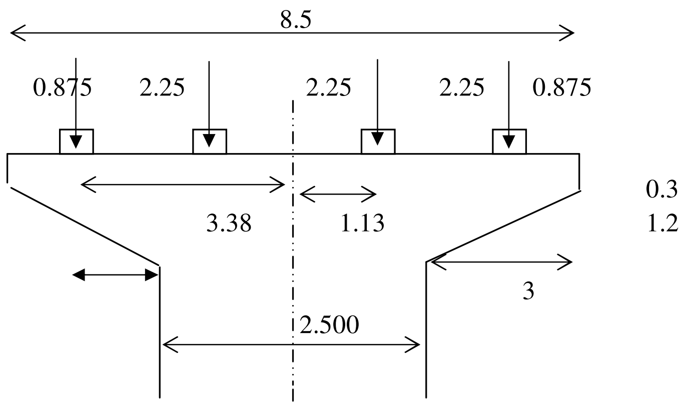
Water current Force in Longtudinal direction

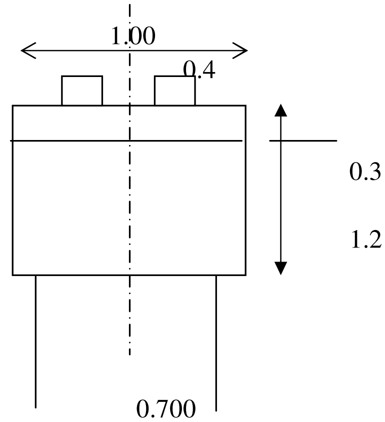
Component	Length	Height	Force	HT of force	Moment
	m	m	t		
Pier	2.50	8.322	2.61162	6.73646	17.5931 t-m
Top of footing	5.500	1.050	0.03165	0.69991	0.02215 t-m

		F	M
Summary	at PierShaft bottom level		
	Transverse direction	1.03555	6.97592582 t-m
	Longitudinal direction	2.61162	17.5930526 t-m

```
PIER SHAFT .case 1 L.W.L (MAX REACTION CASE)

Depth of Section             =      .820 m
Width of Section             =     2.500 m

along width-compression face- no of bar:  18   tension face- no of bar:  18
Dia (mm)                                  20                             20
Cover (cm)                               5.00                            5.0
along depth-compression face- no of bar:   6   tension face- no of bar:   6
Dia (mm)                                  16                             16
Cover  (cm)                              5.00                            5.0

Modular Ratio : Compression  =      10.0
Modular Ratio : Tension      =      10.0
Allowable Concrete Stress    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2000.00 Kg/cm^2

Axial Load                   =   283.700 T
Mxx                          =    64.914 Tm
Myy                          =    76.618 Tm


Intercept of Neutral axis : X axis : =   5.253 m
                          : y axis : =    .757 m

          Concrete Stress Governs Design

Stress in Concrete due to Loads =   47.85 Kg/cm^2
Stress in Steel    due to Loads =  231.46 Kg/cm^2
Percentage of Steel             =     .67 %

PIER SHAFT .case 2 H.F.L (MAX REACTION CASE)

Depth of Section             =      .820 m
Width of Section             =     2.500 m

Modular Ratio : Compression  =      10.0
Modular Ratio : Tension      =      10.0
Allowable Concrete Stress    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2000.00 Kg/cm^2

Axial Load                   =   268.286 T
Mxx                          =    82.507 Tm
Myy                          =    83.594 Tm


Intercept of Neutral axis : X axis : =   4.959 m
                          : y axis : =    .594 m

          Concrete Stress Governs Design



Stress in Concrete due to Loads =   61.64 Kg/cm^2
Stress in Steel    due to Loads =  486.49 Kg/cm^2
Percentage of Steel             =     .67 %
```

```
PIER SHAFT .case 3 L.W.L (SPAN DISLODGED)

Depth of Section              =      .820 m
Width of Section              =     2.500 m

Modular Ratio : Compression   =      10.0
Modular Ratio : Tension       =      10.0
Allowable Concrete Stress     =     85.00 Kg/cm^2
Allowable Steel Stress        =   2000.00 Kg/cm^2

Axial Load                    =   217.650 T
Mxx                           =    26.791 Tm
Myy                           =      .010 Tm


Intercept of Neutral axis : X axis : = ******* m
                          : y axis : =    .891 m

          Concrete Stress Governs Design


Stress in Concrete due to Loads =   18.55 Kg/cm^2
Stress in Steel    due to Loads =  -25.21 Kg/cm^2
Percentage of Steel             =     .67 %


PIER SHAFT .case 4 H.F.L(SPAN DISLODGED)

Depth of Section              =      .820 m
Width of Section              =     2.500 m

Modular Ratio : Compression   =      10.0
Modular Ratio : Tension       =      10.0
Allowable Concrete Stress     =     85.00 Kg/cm^2
Allowable Steel Stress        =   2000.00 Kg/cm^2

Axial Load                    =   202.236 T
Mxx                           =    44.384 Tm
Myy                           =     6.976 Tm


Intercept of Neutral axis : X axis : =  33.465 m
                          : y axis : =    .655 m

          Concrete Stress Governs Design


Stress in Concrete due to Loads =   25.31 Kg/cm^2
Stress in Steel    due to Loads =   63.11 Kg/cm^2
Percentage of Steel             =     .67 %
```

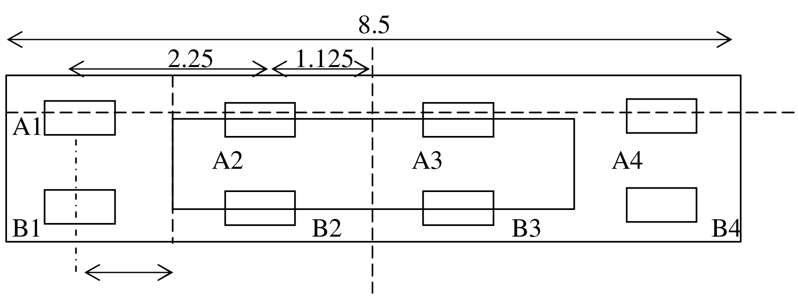




For Outer bearing	a	=	2.125
	D	=	1.5
Available effective depth			
using 25 mm dia of bars	d	=	1.5 - 0.05 - 0.025
		=	1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= Pier cap designed as Cantilever



Loads on bearing

Bearings A1 & B1 are effective.

	A1	B1
DL	16.745	16.745
SIDL	3.3625	3.3625
LL		
70RW	7.5125	9

due to Transverse moment A1 = 30.05 x 1.16 = 34.858

B1 = 36 x 1.16 = 41.76

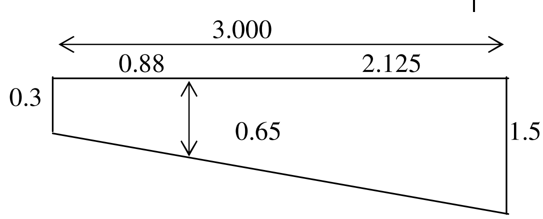
$$A1 = \frac{34.858 \times 3.38}{25.31} = 4.6477 \text{ t}$$

$$B1 = \frac{41.76 \times 3.38}{25.31} = 5.568 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	Left span		Right span	
	B1		A1	
DL	16.745		16.745	
SIDL	3.3625		3.3625	
LL	5.568		4.6477	
	9		7.5125	
	34.675		**32.267**	



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 66.942 x 2.125

= 142.25225 t-m

Due to selfweight of pier cap = 6.48 x 1.1667 = 7.56 t-m

Total moment at the face of support = 149.81 t-m

Torsion at outerbearing section:

= 34.675 - 32.267

= 2.4078

Eccentricity = 0.4

Moment = 0.9631 t-m

Longitudinal Reinforcement:

M = 149.81 = 149.81 t-m

For, M - 25 & Fe - 415

σcbc = 8.33 N/mm^2

m = 10

σst = 200 N/mm^2

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \dfrac{149.81 \times 10\text{^}7}{1.105 \times 1000}$

= 1164.2 < 1425 O.K

A_{st} required $= \dfrac{149.81}{200 \times 0.902 \times 1425}$

= 5827.9363 mm^2

Provide 7 nos 25 mm dia bars in 2 layers 6872.2 mm^2

This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 34.675 + 32.267

= 66.942 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 73.422 t

After shear correction, S.F $= V + \dfrac{M \tan\beta}{d}$

$= 73.422 + \dfrac{149.81 \times 0.4}{1.425}$

= 31.369673 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

b $= \dfrac{1 + 1}{2}$ = 1 m

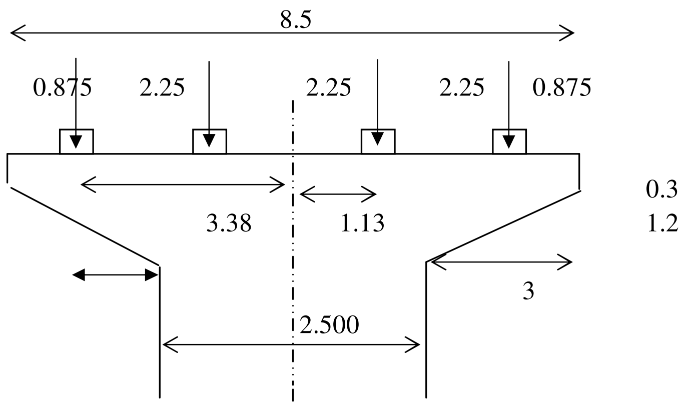
V = 31.37 t

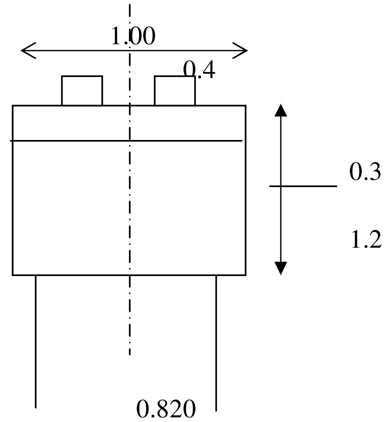
Shear stress $= \dfrac{31.37 \times 10000}{1000 \times 1425}$ = 0.2201 N/mm^2

Area of tension reinforcement = 0.4823 %

τ_c = 0.2966 N/mm^2

Provide minimum shear

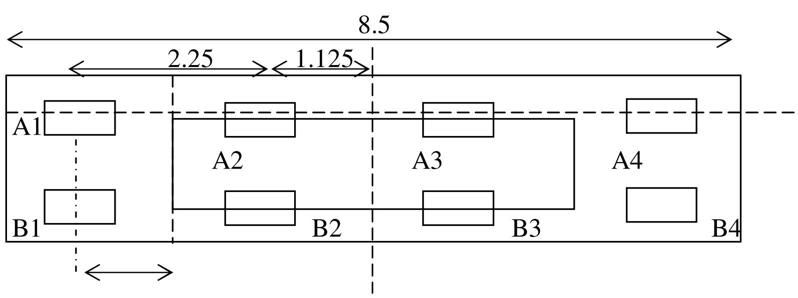




For Outer bearing	a	=	2.125
	D	=	1.5
Available effective depth			
using 25 mm dia of bars	d	=	1.5 - 0.05 - 0.025
		=	1.425 m

$$\frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1$$

= Pier cap designed as Cantilever



Loads on bearing

Bearings A1 & B1 are effective.

	A1	B1
DL	16.745	16.745
SIDL	3.3625	3.3625
LL		
70RW	0	11.93

due to Transverse moment A1 = 0 x 1.16 = 0

B1 = 47.72 x 1.16 = 55.355

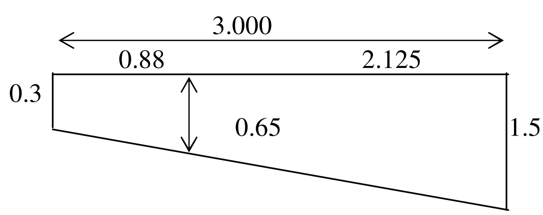
$$A1 = \frac{0 \times 3.38}{25.31} = 0 \text{ t}$$

$$B1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t}$$

Summary of loads from superstructure:

Loads on Outer bearings

	Left span		Right span	
	B1		A1	
DL	16.745		16.745	
SIDL	3.3625		3.3625	
LL	7.3807		0	
	11.93		0	
	39.418		**20.107**	



Selfweight of piercap upto section through outerbearing:

$$= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \text{ t}$$

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

= 59.525 x 2.125

= 126.48997 t-m

Due to selfweight of pier cap = 6.48 x 1.1667 = 7.56 t-m

Total moment at the face of support = 134.05 t-m

Torsion at outerbearing section:

= 39.418 - 20.107

= 19.311

Eccentricity = 0.4

Moment = 7.7243 t-m

Longitudinal Reinforcement:

$M_e = M + M_t$

$M_t = T \times \dfrac{1 + D/b}{1.7}$

$= 6.8155$ t-m

$M_e = 134.05 + 6.8155 = 140.87$ t-m

For, M - 25 & Fe - 415

$\sigma cbc = 8.33$ N/mm^2

m = 10

$\sigma st = 200$ N/mm^2

K = 0.294

j = 0.902

R = 1.105

Effective depth required $= \dfrac{140.87 \times 10^7}{1.105 \times 1000}$

$= 1128.9 < 1425$ O.K

A_{st} required $= \dfrac{140.87}{200 \times 0.902 \times 1425}$

$= 5479.8941$ mm^2

Provide 7 nos 25 mm dia bars in 2 layers 6872.2 mm^2

This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 39.418 + 20.107

= 59.525 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 66.005 t

After shear correction, S.F $= V + \dfrac{M \tan\beta}{d}$

$= 66.005 + \dfrac{134.05 \times 0.4}{1.425}$

= 28.376631 t

Equivalent shear IRC:21-2000,cl:304.2.3.1

$b = \dfrac{1 + 1}{2} = 1$ m

$V_e = V + 1.6 \times \dfrac{T}{b} = 40.735$ t

Shear stress $= \dfrac{40.735 \times 10000}{1000 \times 1425} = 0.2859$ N/mm^2

Area of tension reinforcement = 0.4823 %

$\tau_c = 0.2966$ N/mm^2

```
INPUT FILE: 9.2 M SPAN CLASSA  70R.STD
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     2. START JOB INFORMATION
     3. ENGINEER DATE 09-JUN-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. PAGE LENGTH 1000
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     8. JOINT COORDINATES
     9. 1 0.0 0.0 0.0;2 0.02 0.0 0.0;30.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
    10. 12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38
0.0 0.0
    11. 24 18.4 0.0 0.0
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    13. 1 1 2 23 1 1
    14. CONSTANTS
    15. E 2.7386E+006
    16. POISSON 0.15
    17. DENSITY 2.4 ALL
    18. MEMBER PROPERTY INDIAN
    19. 1 TO 23  PRIS YD 1 ZD 1
    20. SUPPORTS
    21. 3 22 FIXED BUT FX  MZ
    22. 11 14 PINNED
    23. MEMBER RELEASE
    24. 1 START FX FY MZ
    25. 12 START FX FY MZ
    26. 23 END FX FY MZ
    27. DEFINE MOVING LOAD
    28. TYPE 1 LOAD 8 12 12 17 17 17 17
    29. DIST 3.96 1.52 2.13 1.37 3.05 1.37
    30. ** CLASS 70 RW
    31. LOAD GENERATION 82
    32. TYPE 1 -13.4 0 0 XINC 0.5
    33. PERFORM ANALYSIS
    34. **MAX REA
    35. LOAD LIST 29
    36. PRINT SUPPORT REACTION
SUPPORT  REACTION
    -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z

      3   29      0.00     18.95      0.00      0.00      0.00      0.00
     22   29      0.00     15.00      0.00      0.00      0.00      0.00
     11   29      0.00     30.05      0.00      0.00      0.00      0.00
     14   29      0.00     36.00      0.00      0.00      0.00      0.00
      37. ** MAX MOMENT
    38. LOAD LIST 19
    39. PRINT SUPPORT REACTION
      3   19      0.00     32.85      0.00      0.00      0.00      0.00
     22   19      0.00      0.00      0.00      0.00      0.00      0.00
     11   19      0.00     47.15      0.00      0.00      0.00      0.00
     14   19      0.00      0.00      0.00      0.00      0.00      0.00
      40. FINISH
```

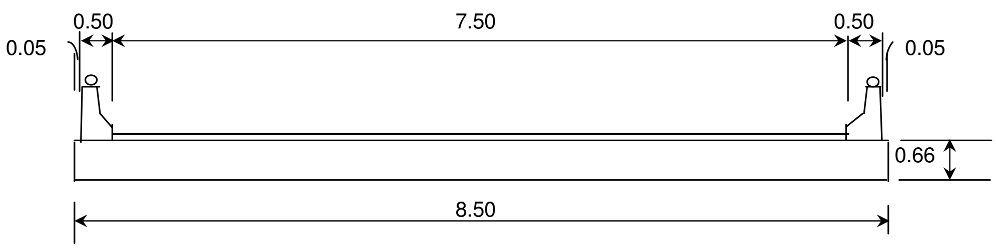
DESIGN OF SUPERSTRUCTURE

DESIGN OF SOLID SLAB SUPERSTRUCTUE

DESIGN DATA

Span (c/c of exp. joint)	=	9.20	m
Eff. Span (c/c brgs.)	=	8.800	m
Overall Width of deck slab	=	8.50	m
Width of carriageway	=	7.50	m
Width of crash barrier	=	0.50	m
Thickness of deck slab	=	0.66	m
Tkickness of wearing coat	=	0.056	m
Unit wt of concrete	=	2.40	t/m^3
Grade of Concrete	-	M 25	25
Grade of Reinforcement	-	Fe 415 (HYSD)	
Live Load	-	One Lane of 70R Wheeled/ Tracked & One Lane of Class A	
Permissible Compressive stress in Concrete	-	850	t/m^2
Permissible Tensile stress in Steel	-	20400	t/m^2
Modular ratio, m	-	10	
Lever arm factor, j	-	0.902	
Moment of resistance coefficient, Q	-	112.75	t/m^2

CALCULATION OF BENDING MOMENTS AT MID SPAN:



CROSS SECTION

DUE TO DEAD LOAD (Per metre width):

Deck slab	=			0.660 x	2.40	=	1.584	t/m
BM at mid span	=	1.584	x	$8.80^2/8$		=	15.333	tm/m

DUE TO SUPERIMPOSED DEAD LOAD (Per metre width):

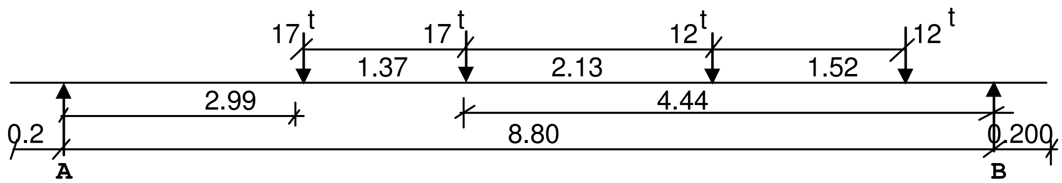
Wearing coat (@ 0.2t/m^2)	=						=	0.200	t/m
Crash barrier (@ 0.9 t/m on each side)	=	2	x	0.90	/	8.50	=	0.212	t/m
						Total	=	0.412	t/m
BM at mid span	=	0.412	x	$8.80^2/8$			=	3.986	tm/m
Tatal BM (Due to DL & SIDL)	=	15.333	+	3.986			=	**19.32**	tm/m

DUE TO LIVE LOAD:

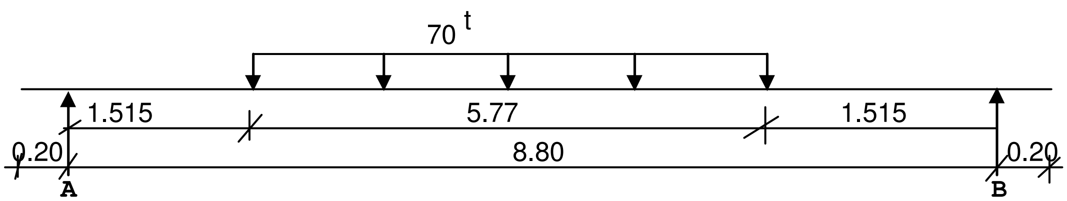
The Superstructure has been analysed for Live load-Class 70R wheeled, class 70R tracked and Class A loading. Load positions for maximum bending moments are evaluated at (i) mid span (L/2) and (ii) at the section 1.5m from support.

CALCULATION OF BENDING MOMENT AT MID SPAN (L/2) :

Due to Class 70R Wheeled Impact factor = 1.25



Due to Class 70R Tracked Impact factor = 1.108



Since bending moment due to Class A is having lesser value as compared to Class 70R loadings.
Hence the superstructure is checked for Class 70R wheeled and Class 70R tracked loading.

EFFECTIVE WIDTH OF SLAB

According to IRC: 21-2000 cl. 305.16.2 (iii) if the effective width of slab for a load overlaps with effective width of slab for an adjecent load, the resultant effective width for two loads equals to sum of respective effective width for each load minus the width of overlap.

Firstly effective width as per IRC:21 is evaluated then actual available width is compared with that value and corresponding load intensity per metre width is evaluated for each axle.

Effective width = $b_{eff} = \alpha.a.(1-a/\ l_0) + b_1$

Where L_0 = the effective span

a = the distance of centre of gravity of the concentrated load from the nearer support

α = A constant depending upon the ratio b/l_0, where b is the width of the slab

b/l_0 = 0.966 Hence as per cl. 305.16.2 of IRC:21, α = 2.439

b_1 = the breadth of concentrated load over the deck slab after 45^0 dispersion through wearing coat

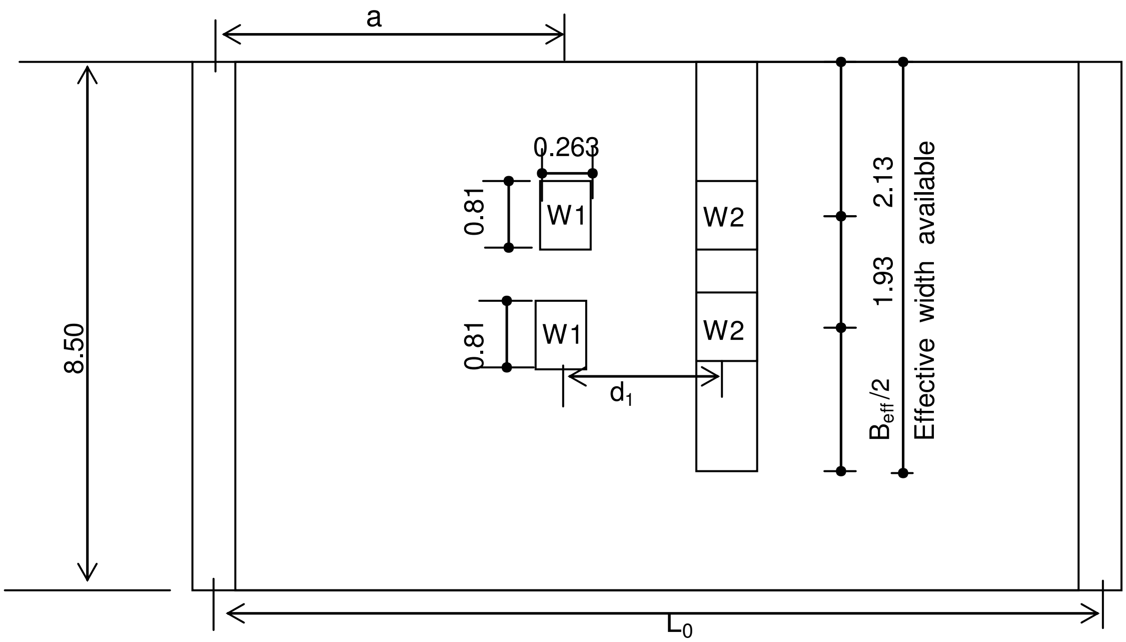
0.81 + 2x 0.065 = 0.922 m (for all axles of class 70R)

0.84 + 2x 0.065 = 0.952 m (for class 70R Tracked)

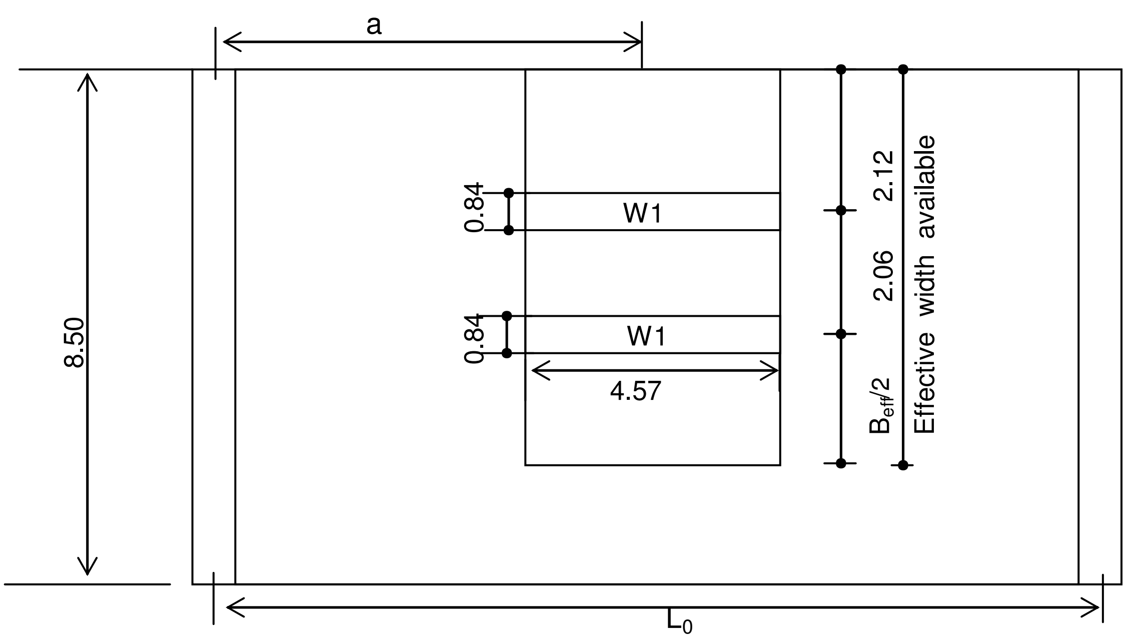
CALCULATION OF EFFECTIVE WIDTH AND LOAD INTENSITY

LIVE LOAD	a (m)	b1 (m)	α	**b_{eff} (m)**	$b_{eff}/2$ (m)	Max. available width (m)	Load intensity (t/m)
FOR MAXIMUM BM AT MID SPAN							
Class 70R Wheeled							
17	2.990	0.922	2.439	5.737	2.869	6.929	2.454
17	4.360	0.922	2.439	6.288	3.144	7.204	2.360
12	2.310	0.922	2.439	5.077	2.539	6.599	1.819
12	0.790	0.922	2.439	2.676	1.338	4.606	2.605
Class 70R Tracked							
70	4.400	0.952	2.439	6.318	3.159	7.339	9.538

TRANSVERSE POSITION OF LIVE LOAD (CLASS 70 R WHEELED LOAD)

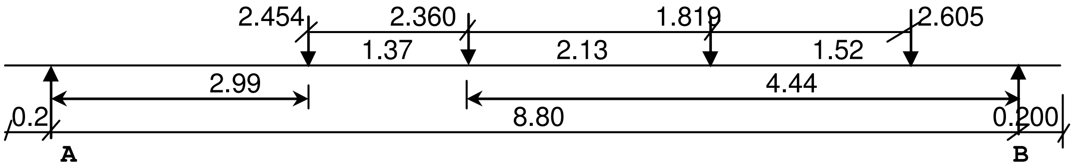


TRANSVERSE POSITION OF LIVE LOAD (CLASS 70 R TRACKED LOAD)



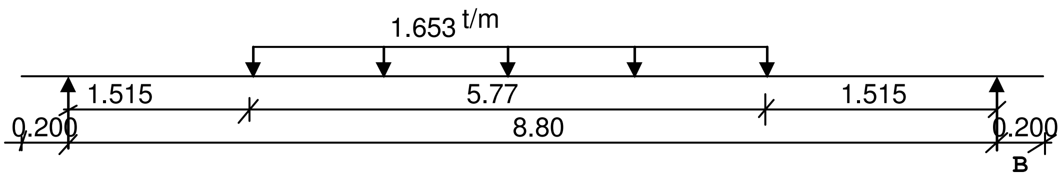
CALCULATION OF LIVE LOAD MOMENTS (Per metre width)

Class 70R (Wheeled) Impact factor = 1.25



Reaction (R_A)	=	3.52	t
BM at mid span	=	12.04	tm/m
BM at mid span (with impact factor)	=	15.05	tm/m

Class 70R (Tracked) Impact factor = 1.108



Reaction (R_A)	=	4.77	t		
BM at mid span	=	14.10	tm/m		
BM at mid span (with impact factor)	=	15.62	tm/m, Say	**15.62**	tm/m

Here 70R Tracked load governs the design

Total Design BM (Due to DL, SIDL & LL)

= 19.32 + 15.62 = 34.94 tm/m

Moment of Resistance (M_R)

lever arm factor, j	=	0.902			
moment of resistance coeff., Q	=	113	t/m^2		
effective cover	=	50	mm		
Eff. available depth	=	600	mm		
M_R	=	**40.59**	tm/m	>	34.94 tm/m

Hence O.K.

Design Reinforcement:

Ast. Reqd. = 31.65 cm^2/m

Hence, Provide 25 **130** c/c, Ast.= **37.76** cm^2/m

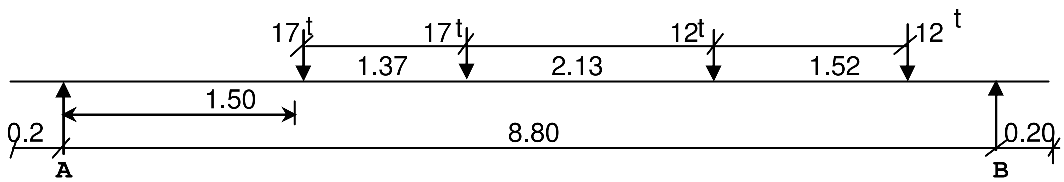
CALCULATION OF BM AT Dist.X = 1.50 m FROM THE SUPPORT

BM AT DISTANCE X, $(BM)_X$ = w. L.x/2.(1-x/L)

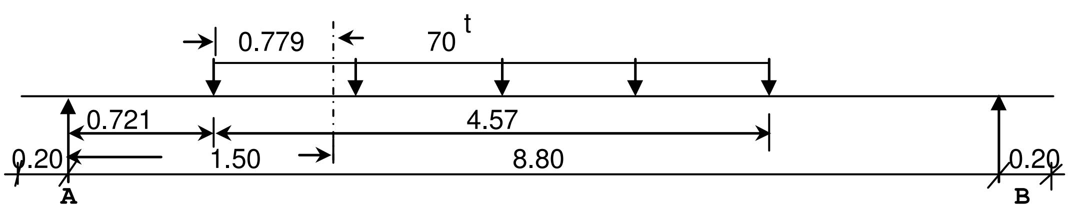
DUE TO DL, $(BM)_X$	**=**	1.584	x	5.475	=	8.67	tm/m
DUE TO SIDL, $(BM)_X$	**=**	0.412	x	5.475	=	2.25	tm/m
TOTAL BM	=	8.67	+	2.25	=	**10.93**	tm/m

DUE TO LIVE LOAD

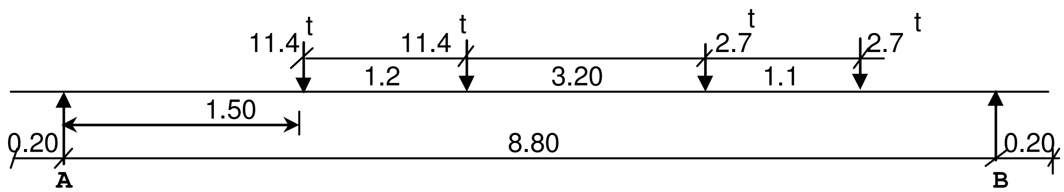
Class 70R (Wheeled) Impact factor = 1.25



Class 70R (Tracked) Impact factor = 1.108



Class A Impact factor = 1.304



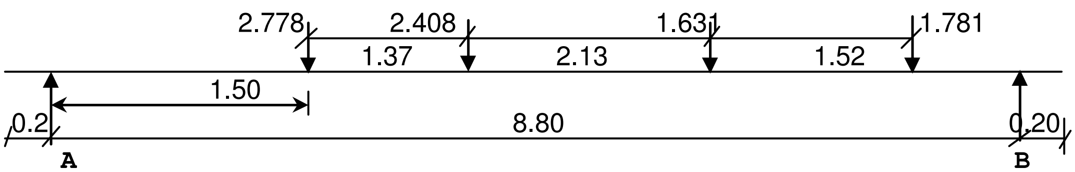
Since bending moment due to Class A is having lesser value as compared to Class 70R loadings. Hence the superstructure is checked for Class 70R wheeled and Class 70R tracked loading.

CALCULATION OF EFFECTIVE WIDTH AND LOAD INTENSITY

LIVE LOAD	a (m)	b1 (m)	α	b_{eff} (m)	$b_{eff}/2$ (m)	Max. available width (m)	Load intensity (t/m)
Class 70R Wheeled							
17	1.500	0.922	2.626	4.190	2.095	6.120	2.778
17	2.870	0.922	2.626	6.001	3.001	7.061	2.408
12	3.800	0.922	2.626	6.593	3.296	7.356	1.631
12	2.280	0.922	2.626	5.359	2.679	6.739	1.781
Class 70R Tracked							
70	3.006	0.952	2.626	6.150	3.075	7.255	9.648

CALCULATION OF LIVE LOAD MOMENTS (Per metre width)

Class 70R (Wheeled) Impact factor = 1.25

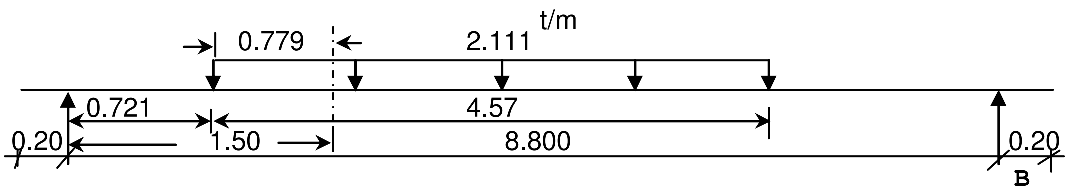


Reaction (R_A) = 5.09 t

BM at the section = 7.64 tm/m

BM at the section (with impact factor) = 9.55 tm/m

Class 70R (Tracked) Impact factor = 1.108



Reaction (R_A) = 6.35 t

BM at the section = 8.89 tm

BM at the section (with impact factor) = 9.84 tm/m, Say **10.00** tm/m

Here 70R Tracked load governs the design

Total Design BM (Due to DL, SIDL & LL)

= 10.93 + 10.00 = 20.93 tm/m

Design Reinforcement:

Ast. Reqd. = 18.64 cm^2/m

Hence, Provide ϕ 20 @ **130** c/c, Ast.= **24.15** cm^2/m

BRIDGE AT CH:54+600

DESIGN OF SUBSTRUCTURE (ABUTMENT)

DESIGN DATA

Formation Level	=	241.909	m
Ground Level	=	236.941	m
Lowest Water Level	=	232.000	m
Highest Flood Level	=	240.009	m
Founding Level	=	232.000	m
Thickness of bearing & pedestal	=	0.000	m
Width of abutment	=	12.000	m
Bouyancy factor	=	1.0	
Safe Bearing Capacity	=	37.600	t/sqm
Dry density of earth	=	1.800	t/cum
Submerged density of earth	=	1.0	t/cum
Saturated density of earth	=	2.000	t/cum
Coefficient of base friction	=	0.55	
Span (c/c of exp. joint)	=	10.800	m
Overall Width of deck slab	=	12.000	m
Width of carriageway	=	11.000	m
Width of crash barrier	=	0.500	m
Depth of Superstructure at mid	=	0.925	m
Depth of Superstructure at end	=	0.775	m
Tkickness of wearing coat	=	0.056	m
Unit wt of concrete	=	2.400	t/m^3
Grade of Concrete	-	M 25	25
Grade of Reinforcement	-	415 (HYSD)	
Live Load	-	One Lane of 70R Wheeled + Class A	
	-	3 lanes of Class A	
Permissible Compressive stress in Concrete	-	850	t/m^2
Permissible Tensile stress in Steel	-	20400	t/m^2
Modular ratio, m	-	10	
factor, k	-	0.294	
Lever arm factor, j	-	0.902	
Moment of Resistance	-	113	t/m^2
Thickness of returnwall		0.5	m

COEFFICIENT OF ACTIVE EARTH PRESSURE

AS PER COULOMB'S THEORY, COEFFICIENT OF ACTIVE EARTH PRESSURE IS

$$Ka = \frac{SIN^2(\alpha+\phi)}{SIN^2\alpha \, . \, SIN(\alpha-\delta)\left[1+\sqrt{\frac{SIN(\alpha+\varphi).SIN(\varphi-\iota)}{SIN(\alpha-\delta).SIN(\varphi+\iota)}}\right]^2}$$

WHERE

ϕ = ANGLE OF INTERNAL FRICTION OF EARTH

α = ANGLE OF INCLINATION OF BACK OF WALL

δ = ANGLE OF INTERNAL FRICTION BETWEEN WALL & EARTH

ι = ANGLE OF INCLINATION OF BACKFILL

HERE					
	ϕ =	30 ^{o}	=	0.524	Radian
	α =	90 ^{o}	=	1.571	Radian
	δ =	20 ^{o}	=	0.349	Radian
	ι =	0 ^{o}	=	0	Radian
			K_a =	0.2973	

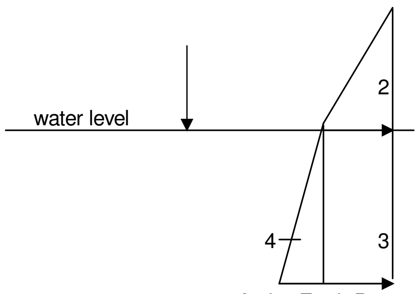
Therefore, Horizontal coefficient of Active earth pressure = **$K_a COS\phi$ = K_{ha} = 0.2794**

HEIGHT OF ABUTMENT

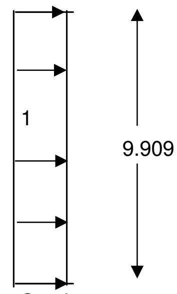
Total height of abutment = Formation Level - Founding Level = 9.909 m

For DESIGN purpose, the height of abutment is considered as, say, = **9.910 m**

CALCULATION OF ACTIVE EARTH PRESSURE



Active Earth Pressure



Surcharge

DRY. condition

a) Service Condition

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	79.73	4.955	395.02
2	Dry Earth	0.5	9.909	4.98	296.27	4.162	1233.00
3		1.0	0.000	4.98	0.00	0.000	0.00
4	SubmgEarth	0.5	0.000	0.00	0.00	0.000	0.00
TOTAL					**376.00**		**1628.02**

b) Span Dislodge Condition

Net force = 376.00 - 79.73 = **296.27** t

Net moment = 1628.02 - 395.02 = **1233.00** tm

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	79.73	4.955	395.02
2	Dry Earth	0.5	1.900	1.06	12.10	8.807	106.59
3		1.0	8.009	1.06	102.03	4.004	408.59
4	SubmgEarth	0.5	8.009	2.24	107.52	2.670	287.06
TOTAL					**301.39**		**1197.26**

b) Span Dislodge Condition

Net force = 301.39 - 79.73 = **221.66** t

Net moment= 1197.26 - 395.02 = **802.24** tm

Forces & moments due to Abutment (Concrete) components

DRY Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment about toe
1	Toe Slab	1.0	2.000	12.00	0.500	2.40	28.80	1.000	28.80
2		0.5	2.000	12.00	0.700	2.40	20.16	1.333	26.88
3	Heel Slab	1.0	3.500	12.00	0.500	2.40	50.40	4.450	224.28
4		0.5	3.500	12.00	0.700	2.40	35.28	3.867	136.42
5	Stem Wall	1.0	0.700	12.00	8.628	2.40	173.94	2.850	495.73
6		0.5	0.500	12.00	8.628	2.40	62.12	2.333	144.95
7	stem rect	1.0	1.200	12.00	1.200	2.40	41.47	2.600	107.83
10	Cap	1.0	0.700	12.00	0.300	2.40	6.05	2.850	17.24
11	Dirt Wall	1.0	0.300	12.00	0.631	2.40	5.45	3.050	16.63
8	Retutnwall	0.5	3.500	1.00	0.700	2.40	2.94	5.533	16.27
8a		1.0	3.500	1.00	9.409	2.40	79.04	4.950	391.23
TOTAL							**505.65**		**1606.24**

H.F.L. Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment ,@ Toe
1	Toe Slab	1.0	2.000	12.00	0.500	1.40	16.80	1.000	16.80
2		0.5	2.000	12.00	0.700	1.40	11.76	1.333	15.68
3	Heel Slab	1.0	3.500	12.00	0.500	1.40	29.40	4.450	130.83
4		0.5	3.500	12.00	0.700	1.40	20.58	3.867	79.58
5	Stem Wall	1.0	0.700	12.00	6.809	1.40	80.07	2.850	228.21
6		0.5	0.500	12.00	6.809	1.40	28.60	2.333	66.73
5	Stem Wall	1.0	0.700	12.00	0.619	2.40	12.48	2.850	35.57
6		0.5	0.500	12.00	0.619	2.40	4.46	2.333	10.40
7	stem rect	1.0	1.200	12.00	1.200	1.40	24.19	2.600	62.90
10	Cap	1.0	0.700	12.00	0.30	2.40	6.05	2.850	17.24
11	Dirt Wall	1.0	0.300	12.00	0.63	2.40	5.45	3.050	16.63
8	Returnwall	0.5	3.500	1.00	0.70	1.40	1.72	5.533	9.49
8c		1.0	3.500	1.00	6.81	1.40	33.36	4.950	165.15
8d		1.0	3.500	1.00	1.90	2.40	15.96	4.950	79.00
TOTAL							**290.88**		**934.20**

Forces & moments due to Earth and LL surcharge

DRY Case : **Self weight of Earth**

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@ Toe
8	DRY EARTH	0.5	3.500	11.00	0.700	1.8	24.26	5.533	134.21
8a		1.0	3.500	11.00	8.709	1.800	603.53	4.950	2987.49
TOTAL							**627.79**		**3121.70**

H.F.L Case :

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@Toe
8	SATURATED SOIL	0.5	3.500	11.000	0.7	1.000	13.48	5.533	74.56
8c		1.0	3.500	11.000	6.809	1.000	262.15	4.95	1297.63
8d	DRY	1.0	3.500	11.000	1.9	1.800	131.67	4.950	651.77
TOTAL							**407.29**		**2023.95**

L.L.SURCHARGE

Element No	Component	Area Factor	Length	Width	Height	Density	Weight	C.G.from toe	Moment `@ Toe
12	DRY EARTH	0.00	3.500	11.00	1.20	1.80	**0.00**	4.950	**0.00**

SUMMARY OF FORCES AND MOMENTS:

LOAD CASE	Case. L.W.L.		Case. H.F.L.	
	Service Cond.	Span dislodged	Service Cond.	Span dislodged
Vertical load from superstructure including LL	**232.07**	0.00	**232.07**	0.00
Vertical load from substructure (b)	1133.44	1133.44	698.17	698.17
Total Vertical Load V = (a) + (b)	1365.51	1133.44	930.24	698.17
Total Horizontal Force H =	**383.51**	296.27	**308.91**	221.66
Moment @ toe due to (a)	**661.40**	0.00	**661.40**	0.00
Moment @ toe due to (b)	4727.95	4727.95	2958.15	2958.15
Total Moment @ toe (M)	5389.35	4727.95	3619.55	2958.15
Dist. of C.G. of V from toe Z = M/V	3.947	4.17	3.89	4.24
eccentricity (e = Z -b/2)	0.597	0.821	0.541	0.887
Relieving Moment @ c/l base (M1)	**814.89**	**930.93**	**503.25**	**619.28**
overturning moment due to				
Horz. braking force	**69.73**	0.00	**69.73**	0.00
Earth Pressure	1628.02	1233.00	1197.26	802.24
Total overturning Moment (M2)	**1697.75**	**1233.00**	**1266.99**	**802.24**
Net moment (M2-M1) = M_L	**882.86**	**302.07**	**763.75**	**182.96**
Factor of Safety				
Against overturning (M / M2)	**3.17**	**3.83**	**2.86**	**3.69**
Against sliding (μ x V / H)	**1.958**	**2.104**	**1.656**	**1.732**

Safe against overturning

Safe against sliding

I.R.C 78-2000:cl 706.3.4

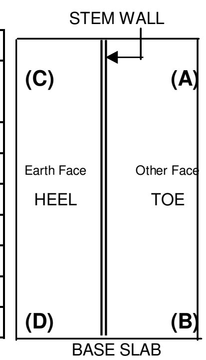
Area of base (A) = 6.700 x 12.00 = 74.40 m^2

Z_L = 89.78 m^3

Z_T = 160.80 m^3

CHECK FOR BASE PRESSURE:

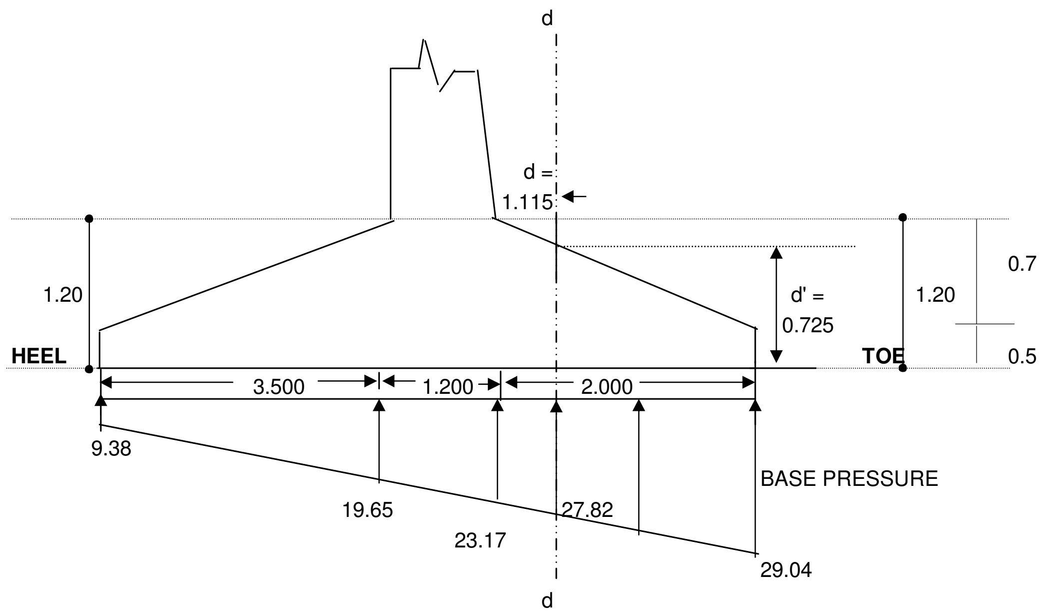
Base Pressure		LWL CASE		HFL CASE	
		Service Cond.	Span dislodged	Service Cond.	Span dislodged
	P/A	18.35	15.23	12.50	9.38
	M_L/Z_L	9.83	3.36	8.51	2.04
	M_T/Z_T	0.86	0.00	0.86	0.00
(A)	$(P/A + M_L/Z_L + M_T/Z_T)$	**29.04**	**18.60**	**21.87**	**11.42**
(B)	$(P/A + M_L/Z_L - M_T/Z_T)$	27.33	18.60	20.15	11.42
(C)	$(P/A - M_L/Z_L + M_T/Z_T)$	9.38	11.87	4.85	7.35
(D)	$(P/A - M_L/Z_L - M_T/Z_T)$	**7.662**	**11.87**	**3.14**	**7.35**



Max.Base Pressure = **29.04** **t/m²** **<** **37.60** **Hence O.K.**

Min. Base Pressure = **3.14** **t/m²** **>** **0** **Hence O.K.**

DESIGN OF TOE SLAB



BENDING MOMENT AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward Loads	1	1.0	2.400	1.000	2.400
	2	0.5	1.680	0.667	1.120
Upward Base pressure	Rect.	1.0	-46.348	1.000	-46.348
	Trian.	0.5	-5.871	1.333	-7.828
TOTAL			-48.139		-50.656

Bending Moment at face of stem 50.656 tm/m

Effective depth required 0.670 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 2469 mm^2

Minimum steel required 0.15% 1673 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 617 mm^2/m

12 ϕ , @ 150 C/C

mainsteel 2469 mm^2 753.9822

Hence provide , 25 ϕ , @ 175 C/C

0 ϕ , @ 175 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 669 mm^2/m

provide 12 ϕ , @ 150 C/C

753.982

Check for Shear

SHEAR FORCE AT "d" FROM FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward loads	1	1.0	1.062	0.443	0.470
	2	0.5	0.743	0.295	0.219
Upward base pressure	Rect.	1.0	-24.624	0.443	-10.896
	Trian.	0.5	-0.540	0.590	-0.319
TOTAL			-23.359		-10.526

Effective depth (d') at distence d 0.725 m

Shear force at critical section 23.4 t

Bending Moment at critical section 10.53 tm

tan β = 0.36

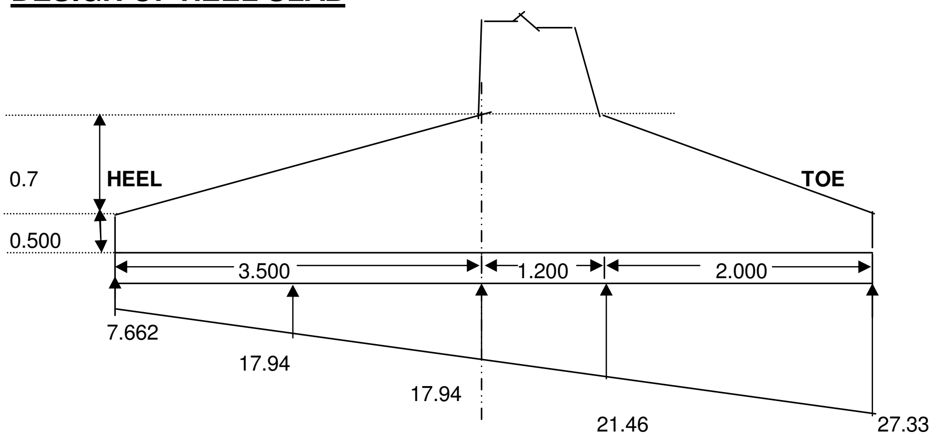
Net shear force S-M*tanβ/d' **18.10** t

Hence, Shear stress 24.97 t/m^2

% of reinforcement 0.39

Permissible shear stress 27.92 t/m^2 **Hence O.K.**

DESIGN OF HEEL SLAB



BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	moment
Downward Loads.	3	1	4.200	1.750	7.350
	4	0.5	2.940	1.167	3.430
	8	0.5	4.410	2.333	10.290
	8a	1	54.867	1.750	96.017
		0.5	4.410	2.333	5.145
		1	6.831	1.750	11.955
Upward base pressure	Rect.	1	-26.819	1.750	-46.932
	Trian.	0.5	-17.979	1.167	-20.976
TOTAL			32.860		66.278

Bending Moment at face of stem **66.278** tm

Effective depth required 0.767 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3232.56 mm^2
Minimum steel 0.15% 1672.50 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 808.14 mm^2/m
mainsteel 3232.56 mm^2

12 φ , @ 150 C/C

Hence provide , 25 φ , @ 150 C/C 753.9822
0 φ , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 836 mm^2/m
provide 16 φ , @ 150 C/C
1340.41

Check for Shear (Critical section at face of stem)

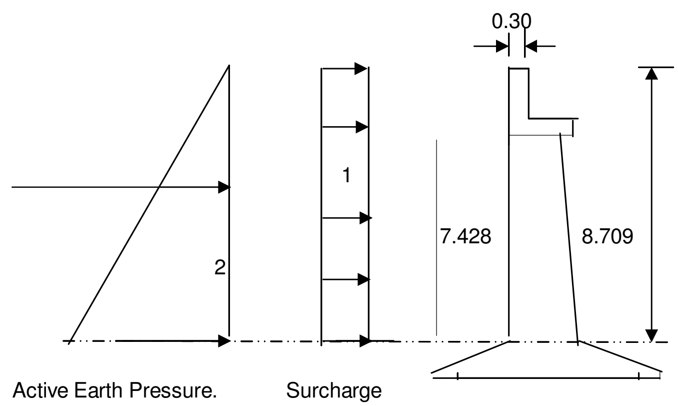
Shear force at face of stem 32.86 t

tan β 0.200
Bending moment at face of stem 66.278 tm
Net shear force S-M*tanβ /d 20.97 t

Hence, Shear stress 18.81 t/m^2
% of reinforcement 0.29

Permissible shear stress 24.75 t/m^2 **Hence O.K.**

DESIGN OF STEM WALL



SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT

R.L. OF SECTION = 233.200 m

DRY Condition

a) Service Condition

Element No.	Area factor	Height of E.P. diagram	Earth Pressure	Force	L.A.	Moment tm
1	1	8.359	0.603	60.5	4.180	253.00
2	0.5	8.359	4.204	210.8	3.511	740.18
HORIZONTAL FORCE				7.52	7.728	58.08
TOTAL				278.88		**1051.256**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 256.712 + 5.45 + 6.05 + 232.07
= 500.282 t
Longitudinal Moment = 1051.256 t-m
Transverse Moment = 137.896 t-m

b) Span Dislodged Condition

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	256.712	+	5.45	+	6.05	+	0.00
	=	268.212	t					
Longitudinal Moment	=	993.176	t-m					
Transverse Moment	=	0.000	t-m					

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.603	60.53	4.180	253.00
2	Dry Earth	0.5	1.494	0.83	7.48	7.307	54.68
3		1.0	6.809	0.83	68.21	3.405	232.22
4	SubmgEarth	0.5	6.809	1.90	77.72	2.270	176.39
TOTAL					**213.94**		**716.289**

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	121.151	+	5.45	+	6.05	+	232.07
	=	364.721	t					
Longitudinal Moment	=	774.368	t-m					
Transverse Moment	=	137.896	t-m					

b) Span Dislodged Condition

Total Vertical Load	=	Stem + dirt wall + cap + Load from superstructure						
	=	121.151	+	5.45	+	6.05	+	0.00
	=	132.651	t					
Longitudinal Moment	=	716.289	t-m					
Transverse Moment	=	0.000	t-m					

Cross Sectional area 0.95 m^2 2375.000

For horizontal reinforcement area of steel required for the stem at the section/metre = 185.4639 mm^2

Providing **12** ,@ 47.62 c/c say, **150** C/C as horizontal reinforcement

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

For Class A Single lane : F_h = (0.2×26.6) = 5.320 t

For class 70R Wheeled : F_h = (0.2×55.2) = 11.032 t

For class A 3 lane F_h = 0.2 x 26.6 + 0.05 x 26.6 = 6.65 t

For class 70R W +class A 1 lane

F_h = 0.2 x 55.2 + 0.05 x 26.6 = 12.362 t

μ R_g = 7.5155 t

$F_h/2$ = 6.181

Summary of Longitudinal Forces :

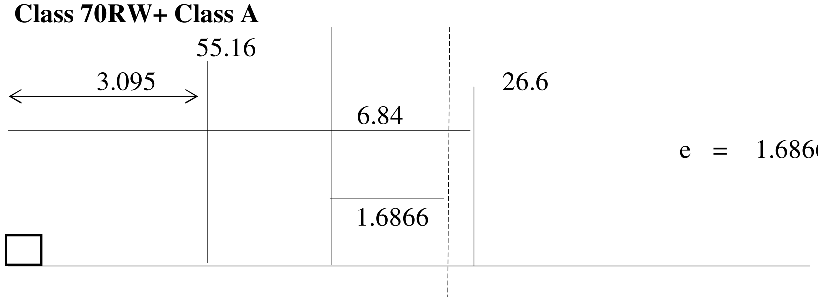
Load Case	Longitudinal horizontal force (t)
70RT+class A 1lane	**7.52**

Dead Load

dead load of slab	=	24.48	t/m		
Total reaction	=	132.19		=	132.19 t
wearing coat	=	1.36	t/m	=	7.3181 t
crashbarrier	=	1.00	t/m	=	10.8 t

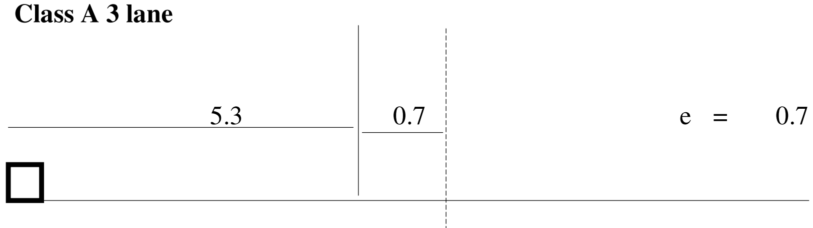
Transverse eccentricity

Class 70RW+ Class A



e = 1.6866

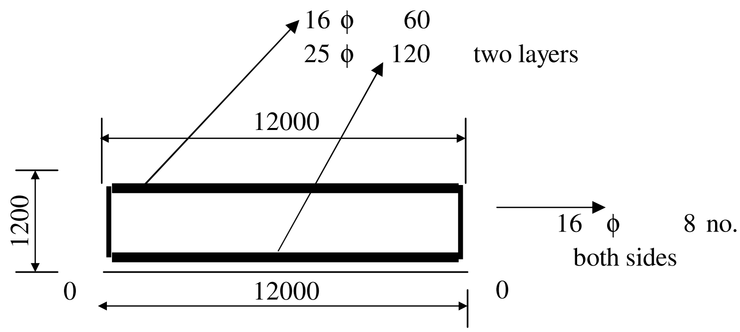
Class A 3 lane



e = 0.7

Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

	P		M_L	M_T
Dead Load Reaction:				
	132.19	t		
SIDL	18.12	t		
L.L Max Reaction Case				
70RT + Class A	81.76	t		137.90
Class A 3 Lane	79.8	t		55.86
	232.07			**137.90**



PLAN : ABUTMENT SHAFT

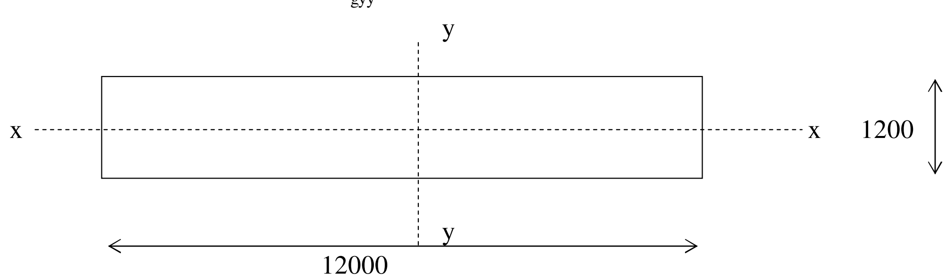
Reinforcement provided = 74185.569 mm^2

Summary of Loads at Abutment Shaft bottom :

		P	M_L	M_T
1	**DRY condition**			
	With L.L	500.28	1051.26	137.90

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}	=	1.728E+12	mm^4
Gross M.O.I of section	I_{gyy}	=	1.728E+14	mm^4



Abutment section

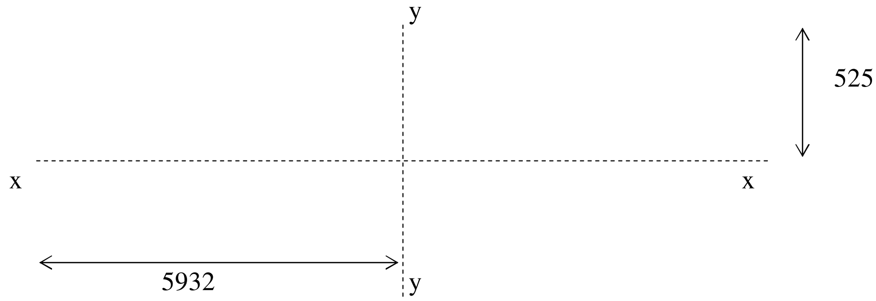
Transformed sectional properties of section:

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$_{txx}$ = $I_{gxx} + 2\left[m - 1\right] A_{s:}\, ax^2$

= 2.02303E+12 mm^4

$Z_{xx} = \frac{M.Itxx}{d/2}$ = 3.372E+09 mm^3

Transformed M.I$_{tyy}$ = $I_{gyy} + 2\left[m - 1\right] A_{s:}\, ay^2$

= 1.73819E+14 mm^4

$Z_{yy} = \frac{M.I_{tyy}}{d/2}$ = 2.897E+10 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.728E+12 mm^4

Area of section = 14400000 mm^2

r = 346.41016 mm

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

Abutment shaft height L = 9.426 m

Effective length L_{eff} = 11.311 m

Slenderness ratio = 32.653 < **50**

Type of member = 1

1 Short Column

2 Long Column

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

β = 1

Permissible stresses : *concrete*

σ_{cbc} = 8.3333 N/mm^2

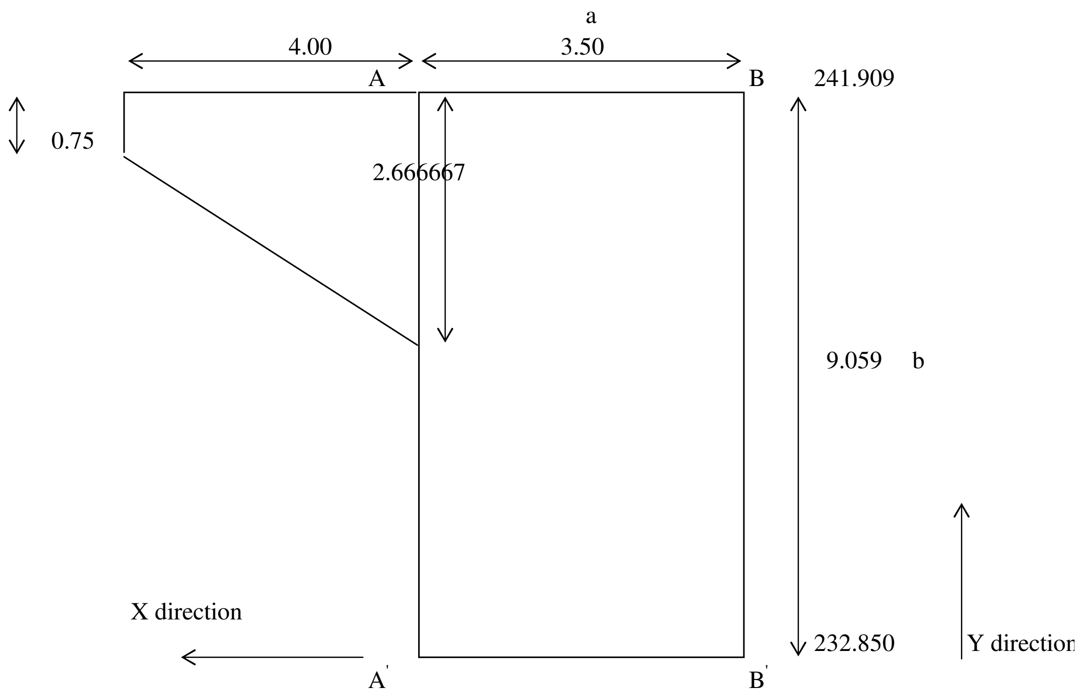
σ_{co} = 6.25 N/mm^2

Tensile stress = 0.61 N/mm^2

Permissible stresses : *Steel*

σ_{st} = 200 N/mm^2

S.No Item	DRY Case
Loads and Moments	With L.L
1 P	500.28 t
2 M_L	1051.26 t-m
3 M_T	137.90 t-m
Actual(calculated) Stresses	
4 $\sigma_{co,cal}$ P/A_{tfm} 5 $\sigma_{cbc,cal}$ M_L/Z_{xx} 6 $\sigma_{cbc,cal}$ M_T/Z_{yy} 7 $\sigma_{cbc,cal}$ = 5 + 6	0.332023197 3.117862579 0.047599842 3.165462421
Permissible Stresses	
8 σ_{cbc} 9 σ_{co}	8.3333333 6.25
Check for Minimum steel area mm^2 10 Conc.Area Required for directstress	
(1)/(9)	800450.56
11 0.8% of area required	6403.6045
12 0.3% of A_g	43200
13 Governing steel mm^2	43200
14 Provided Steel area mm^2	74185.569
Check for safety of section $\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$ *Check for Cracked /Uncracked section*	0.4329792 < 1
$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-2.833439
Permissible Basic tensile stress in concrete	-0.61
Section to be designed as	**Cracked**



Width of Solid return wall (a)	=	3.50
Width of Cantilever return wall	=	4.00
Avg Height of Solid return wall (b)	=	9.059
Height of Cantilever return at Tip	=	0.75
Height of Cantilever return at Root	=	2.666667
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	=	0.333333
j	=	0.888889
R	=	151.1111 t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.3863561	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 = 0.3863561 β_1 = 0.378256

β_2 = 0.419259

Live Load Surcharge:

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

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

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

$\sigma_{bmax} = \dfrac{0.378256 \times 0.603504 \times 82.07}{0.25} = 74.9353$ 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 $= 74.935304 \times 0.041667 = 3.122304$ t-m/m

$\sigma_{amax} = \dfrac{0.4192586 \times 0.603504 \times 82.07}{0.25} = 83.05823$ 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 $= 83.058232 \times 0.041667 = 3.46076$ t-m/m

Case (2) For Triangular loading due to earth pressure

a/b = 0.3863561 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 = 0.3863561 β_1 = 0.222538

β_2 = 0.152724

Earth pressure:

$q = 0.2794 \times 1.8 \times 9.059 = 4.555952$ t/m^2

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

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

$\sigma_{bmax} = \dfrac{0.2225385 \times 4.555952 \times 82.07}{0.25} = 332.8164$ 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	=	332.81644 x 0.041667		=	**13.86735**	t-m/m
σ_{amax}	=	$\dfrac{0.1527241 \times 4.555952 \times 82.07}{0.25}$		=	228.4059	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	=	228.40593 x 0.041667	=	**9.516914**	t-m/m
Total Moment in Solid Return /m height			=	**12.97767**	t-m/m
				Along X-direction	
Total Moment in Solid Return /m width			=	**16.98966**	t-m/m
				Along Y-direction	

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A'

$$M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00$$
$$+ \ 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00$$
$$+ \ 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times dx \times \left(4.00 - X \right)$$
$$+ \ 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left(4.00 - X \right)$$

$$= 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.66667$$

$$= \mathbf{14.11063} \text{ t-m}$$

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

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

$$M = R \times b \times d^2$$
$$= 151.1111 \times 2.666667 \times 0.179776 = 72.44307 \text{ t-m}$$

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

A_{st}/m = 688.231 mm^2/m

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

Provide 10 mm dia @ 150 mm c/c providing 523.5988 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 5.50
\+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 5.50
\+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (7.50 - X)
\+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (7.50 - X)

= 9.957816 + 3.1118175 + 0.11176 x 96 + 0.653796 x 38.66667

= **49.07871** t-m

Moment in Solid Return /m height = 12.97767 + $\frac{49.07871}{9.059}$ = 18.39535 t-m/m

Moment in Solid Return /m width = = 16.98966 t-m/m

Design of face B-B$'$

Moment in Solid Return /m height = **18.39535** t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 20 - 13 = 418 mm

M = R x b x d^2
= 151.1111 x 1 x 0.174306 = 26.33961 t-m

A_{st} = $\frac{18.39535 \times 10\text{^}6}{20400 \times 0.888889 \times 0.4175}$ = 2429.819 mm^2

A_{st}/m = 2429.819 mm^2/m

Provide 20 mm dia @ 125 mm c/c providing 2513.274 mm^2 on earth face.
Provide 12 mm dia @ 125 mm c/c providing 904.7787 mm^2 on other face.
Along Horizantal direction.

Design of face A$'$-B$'$

Moment in Solid Return /m width = **16.98966** t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 0 - 13 = 438 mm

M = R x b x d^2
= 151.1111 x 1 x 0.191406 = 28.92361 t-m

A_{st} = $\frac{16.98966 \times 10\text{^}6}{20400 \times 0.888889 \times 0.4375}$ = 2141.553 mm^2

A_{st}/m = 2141.553 mm^2/m

Provide 20 mm dia @ 125 mm c/c providing 2513.274 mm^2 on earth face.
Provide 12 mm dia @ 125 mm c/c providing 904.7787 mm^2 on other face.
Along Vertical direction.

1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure	K_a	=	0.279384
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		=	0.5
Avg. cover to reinforcement. (FOR 2-3 LAYERS)		=	0.150 m

H (From formation level) = 8.709 m

DESIGN FOR BENDING MOMENT

S.NO.	UNITS	1
H (FROM TOP)	m	8.709
WIDTH OF RETURN AT THIS LEVEL	m	3.500
FORCE DUE TO EARTH PRESSURE	t	107.527
MOMENT DUE TO EARTH PRESSURE	tm	393.310
FORCE DUE TO L.L. SURCHARGE	t	29.632
MOMENT DUE TO L.L. SURCHARGE	tm	129.032
TOTAL MOMENT	tm	522.342
DESIGN MOMENT	tm	522.342
REQUIRED EFFECTIVE DEPTH	m	2.940
EFF. DEPTH AVAILABLE	m	3.350
AREA OF STEEL REQUIRED	cm^2	85.976
DIAMETER OF BAR PROVIDED	mm	32
TOTAL NO. OF BARS	no.	12
AREA OF STEEL PROVIDED	cm^2	96.51
		O.K.
		12.3
CHECK FOR SHEAR STRESS		
SHEAR FORCE	t	142.159
SHEAR STRESS	t/m^2	84.871
Ast/bd x 100	%	0.513
PERMISSIBLE SHEAR STRESS	MPa	0.31
PERMISSIBLE SHEAR STRESS	t/m^2	31.95
Asv/sv	cm^2/m	7.744

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	=	300	mm				
Assuming a cap thickness of	=	300	mm				
Volume of Abutment cap	=	0.3	x	0.70	x	12	
	=	2.52	m^3				
Quantity of steel	=	1 %	of volume				
	=	$\frac{1}{100}$	x	2.52			
	=	0.0252	m^3				
Quantity of steel to be provided at top	=	0.0126	m^3				
Quantity of steel to be provided at bottom	=	0.0126	m^3				

Top & bottom face:

Quantity of steel to be provided in Longitudinal dir = 0.0063 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

$$\frac{0.0063}{11.9} = 529.412 \text{ mm}^2$$

Provide **8** nos of bars **12** mm dia at top & bottom face.

= 904.779 mm^2

Transverse steel:

Quantity of steel to be provided in Longitudinal dir = 0.0063 m^3

Assuming a clear cover of = 50 mm

Assuming a dia of bar = 12 mm

Length of bar 0.70 - 0.100 = 0.6 m

Volume of each stirrup = 6.8E-05 m^3

no of stirrups required for m/length = 8 nos

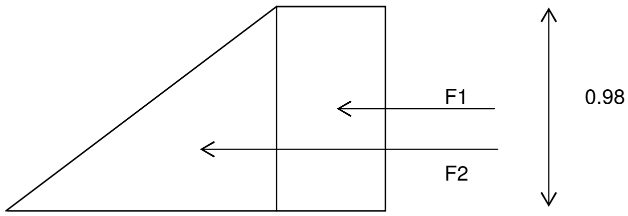
Required Spacing = $\frac{1000}{8}$

= 125 mm

Provide 12 mm dia bar 125 mm c/c stirrups throught in length of abutment cap.

904.779 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	0.98	=	7.893832	t
Intensity for triangular portion	=	0.2794	x	2.00	x	0.98	=	0.548183	t/m^2
F2	=	0.548183	x	12.00	x	0.98	=	3.226604	t
M1	=	7.893832	x	0.49			=	3.871925	t-m
M2	=	3.226604	x	0.41202			=	1.329425	t-m
M1+ M2							=	**5.20135**	t-m
Total moment at base of dirt wall /m length							=	0.433446	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 $= \dfrac{0.433446}{1.51 \times 1000} = 53.57707$ mm

Ast req $= \dfrac{0.433446}{200 \times 0.889 \times 244} = 99.91099$ mm^2/m

Minimum steel $= \dfrac{0.12}{100} \times 300 \times 1000 = 360$ mm^2/m

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	0.433446	=	0.130034	t-m/m

Ast req $= \dfrac{0.130034}{200 \times 0.889 \times 238} = 30.72893$ mm^2/m

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.

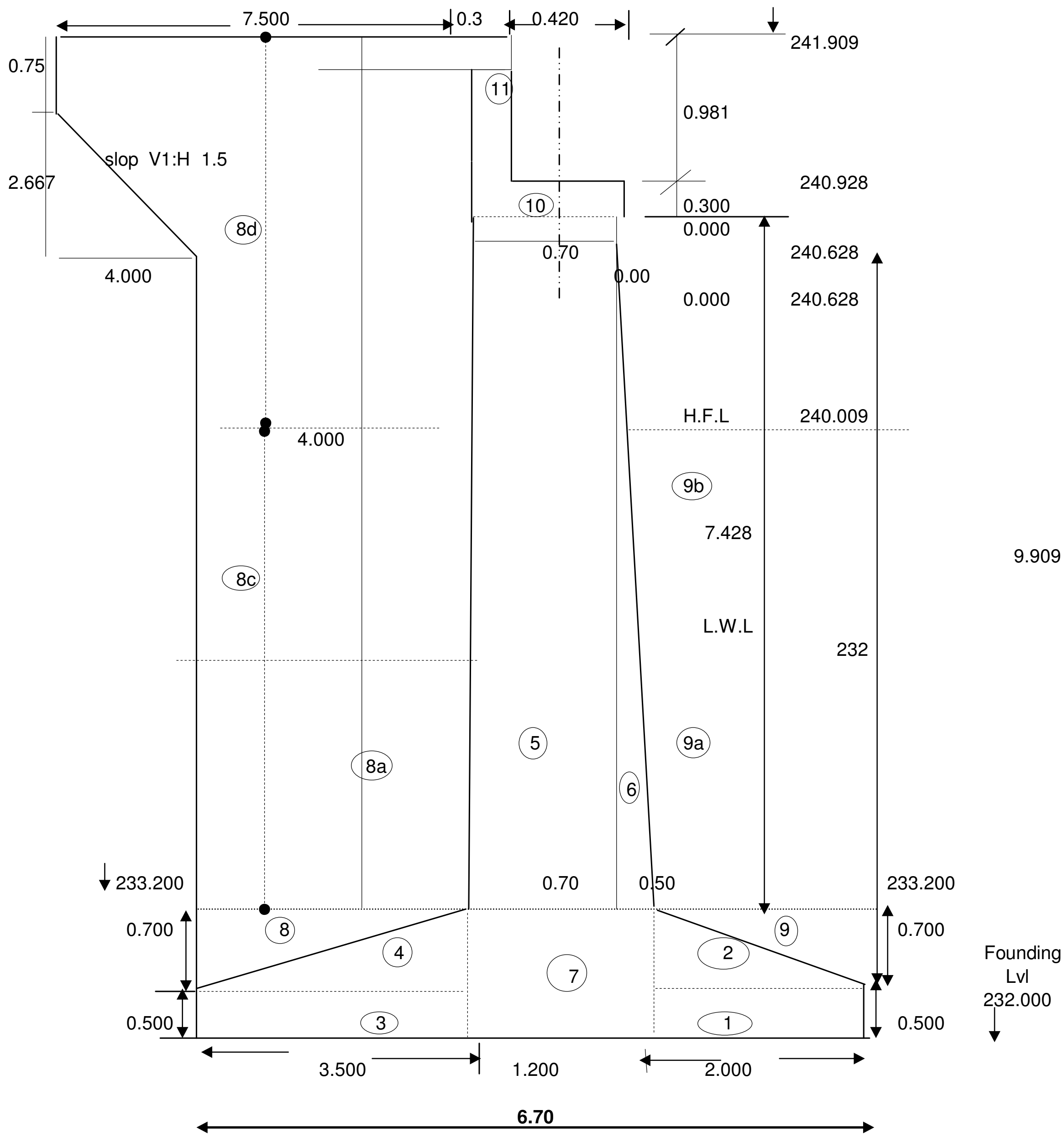
392.6991 mm^2/m

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



7.500
0.3
0.420
241.909
0.75
0.981
2.667
slop V1:H 1.5
240.928
0.300
0.000
240.628
4.000
0.70
0.00
0.000
240.628
H.F.L
240.009
4.000
7.428
9.909
L.W.L
232
233.200
0.70
0.50
233.200
0.700
0.700
Founding
Lvl
232.000
0.500
0.500
3.500
1.200
2.000
6.70
11
10
8d
8c
8a
5
6
9b
9a
8
4
7
2
9
3
1

```
INPUT FILE: 70r.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.4 0 0;3 10.4 0 0;4 10.8 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    22. LOAD GENERATION 175
    23. TYPE 1 -13.4 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 55
    26. PRINT SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = PLANE
   -----------------

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


      2   55      0.00     36.84      0.00      0.00      0.00      0.00
      3   55      0.00     55.16      0.00      0.00      0.00      0.00

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

    27. FINISH
```

```
INPUT FILE: CLAAS A.STD
     3. STAAD PLANE
     4. INPUT WIDTH 72
     5. UNIT METER MTON
     6. PAGE LENGTH 1000
     7. UNIT METER MTON
     8. JOINT COORDINATES
     9. 1 0.00 0 0;2 0.4 0 0;3 10.4 0 0;4 10.8 0 0
    10. MEMBER INCIDENCES
    11. 1 1 2 3
    12. MEMBER PROPERTY CANADIAN
    13. 1 TO 3 PRI YD 1.0 ZD 1.0
    14. CONSTANT
    15. E CONCRETE ALL
    16. DENSITY CONCRETE ALL
    17. POISSON CONCRETE ALL
    18. SUPPORT
    19. 2   3   PINNED
    20. DEFINE MOVING LOAD
    21. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    22. LOAD GENERATION 202
    23. TYPE 1 -18.8 0. 0. XINC .2
    24. PERFORM ANALYSIS
    25. LOAD LIST 74
    26. PRINT SUPPORT REACTION
        SUPPORT  REACTION
    SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = PLANE
   -----------------

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


      2   74      0.00     26.60      0.00      0.00      0.00      0.00
      3   74      0.00      9.80      0.00      0.00      0.00      0.00


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

     27. FINISH
```

```
Abutment SHAFT ..DRY NORMAL


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

along width-compression face- no of bar:  70   tension face- no of bar: 140
Dia (mm)                                  16                             25
Cover (cm)                               7.5                           10.5
along depth-compression face- no of bar:   8   tension face- no of bar:   8
Dia (mm)                                  16                             16
Cover  (cm)                              7.50                           7.5

Modular Ratio : Compression  =      10.0
Modular Ratio : Tension      =      10.0
Allowable Concrete Stress    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   500.282 T
Mxx                          =  1051.256 Tm
Myy                          =   137.896 Tm


Intercept of Neutral axis : X axis : = 273.656 m
                          : y axis : =    .369 m

          Concrete Stress Governs Design



Stress in Concrete due to Loads =   60.05 Kg/cm^2
Stress in Steel    due to Loads = 1206.23 Kg/cm^2
Percentage of Steel             =     .60 %

Abutment SHAFT ..DRY SPAN DISLODGED

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   268.212 T
Mxx                          =   993.176 Tm
Myy                          =      .010 Tm

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

          Concrete Stress Governs Design

Stress in Concrete due to Loads =   54.70 Kg/cm^2
Stress in Steel    due to Loads = 1255.23 Kg/cm^2
```

```
 Percentage of Steel             =     .60 %

Abutment SHAFT ..HFL NORMAL

 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    =     85.00 Kg/cm^2
 Allowable Steel Stress       =   2040.00 Kg/cm^2

 Axial Load                   =   364.721 T
 Mxx                          =   774.368 Tm
 Myy                          =   137.896 Tm


 Intercept of Neutral axis : X axis : = 202.764 m
                           : y axis : =    .371 m

          Concrete Stress Governs Design

 Stress in Concrete due to Loads =   44.57 Kg/cm^2
 Stress in Steel    due to Loads =  894.35 Kg/cm^2
 Percentage of Steel             =     .60 %

 Abutment SHAFT ..HFL SPAN DISLODGED


 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    =     85.00 Kg/cm^2
 Allowable Steel Stress       =   2040.00 Kg/cm^2

 Axial Load                   =   132.289 T
 Mxx                          =   716.289 Tm
 Myy                          =      .010 Tm


 Intercept of Neutral axis : X axis : = ******* m
                           : y axis : =    .321 m

             Steel Stress Governs Design

 Stress in Concrete due to Loads =   39.20 Kg/cm^2
 Stress in Steel    due to Loads =  945.83 Kg/cm^2
 Percentage of Steel             =     .60 %
```

DESIGN OF SUBSTRUCTURE (PIER)

Design Data :

For design purposes, following parameters have been considered.

Grade of concrete		= M - 25
	Pier Cap	= M - 25
Grade of reinforcement steel		= Fe - 415
Centre to Centre distance of A / Expansion joints		= 10.800 m
Centre to Centre distance of Bearing		= 10.400 m
Depth of superstructure		= 925 mm
Thickness of wearing coat		= 56.00 mm
Formation level along C of carriage way		= 241.909 m
Soffit level		= 240.928
Pedestal top level		= 240.928
Height of bearing and Pedestal		= 0.000 m
L.W.L./Bed level		= 236.941 m
H.F.L		= 240.009 m
M.S.L		= 234.384 m
Founding Level		= 233.441 m
Pier cap top level		= 240.928 m
S.B.C of Soil		= 37.600 t/m^2

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

(b) Class 70R wheeled/Tracked + Class A

Maximum surface velocity of water = 2 x 1.200 m/sec = 1.7 m/sec

Say, 1.70 m/sec

The following codes are used for the design of substructure:

1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000

Dead Load

dead load of slab	=	24.28	t/m		
Total reaction	=	262.22		=	262.22 t
wearing coat	=	1.36	t/m	=	14.636 t
crashbarrier	=	1.00	t/m	=	21.6 t

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

For Class A 3 lane : $\mathbf{F_h} =$ 0.2 x 31.5 + 0.05 x 31.5 = 7.883 t

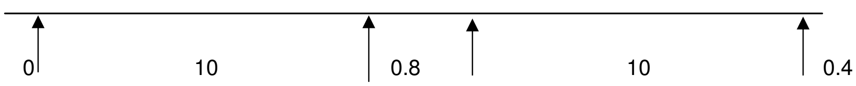
class 70R wheeled + A : $\mathbf{F_h} =$ 0.2 x 71.5 + 0.05 x 31.5 = 15.883 t

$\mu\ R_g$ = 7.4615 t

$F_h/2$ = 7.9413

Summary of Longitudinal Forces :

Load Case	Longitudinal horizontal force (t)
class 70R + A	**7.94**
Span dislodged	**0.00**



SUMMARY sheet 54-600

		R_B	R_C	M_L	**R**	M_T
70RW	max rea	32.11	39.42	2.924	71.53	208
	max long	53.32	0	21.328	53.32	155
70RT	max rea	32.7	32.1	0.24	64.8	184
	max long	0	58.19	23.276	58.19	165
CLASS A	max rea	25	6.53	7.388	31.53	132
	max long	3.64	26.6	9.184	30.24	127
CLASS3A	max rea	75	19.59	22.164	85.131	59.6
	max long	10.92	79.8	27.552	90.72	63.5

H.F.L Condition with L.L					
a) Vertical load and their moments about C/L of Foundation base.					
I					
1 **D.L Reaction**	**P**	**e_L**	**M_L**	**e_T**	**M_T**
a Left span	132.19	-0.4	-52.876	0	0
b Right span	132.19	0.4	52.876	0	0
2 S.I.D.L					
a Left span	18.12	-0.4	-7.248	0	0
b Right span	18.12	0.4	7.248	0	0
	300.62		***0***		***0***
4 **L.L Max Reaction case**					
a Left span					
70R Wheeled + class A	57.978	-0.4	-23.191	1.76	102.04
Class A 3 Lane	67.5	-0.4	-27	0.7	47.25
b Right span					
70R Wheeled + class A	35.307	0.4	14.123	1.76	62.14
Class A 3 Lane	17.631	0.4	7.0524	0.7	12.342
4 **L.L Max Long.Moment case**					
a Left span					
70R Wheeled + class A	71.928	-0.4	-28.771	1.76	126.59
Class A 3 Lane	9.828	-0.4	-3.9312	0.7	6.8796
b Right span					
70R Wheeled + class A	3.276	0.4	1.3104	1.76	5.7658
Class A 3 Lane	71.82	0.4	28.728	0.7	50.274

Max Reaction case:	**385.75**	**19.948**	**164.18**
Max Long. moment case:	**375.82**	**27.461**	**132.36**

II Substructure	**V**	ρ	**P**	**e_L**	**M_L**	**e_T**	**M_T**
1 Pedestal							
a Left span	0	2.4	0	-0.4	0	0	0
b Right span	0	2.4	0	0.4	0	0	0
2 Pier cap R	3.69	2.4	8.856	0	0	0	0
Pier cap Tr	0	2.4	0	0	0	0	0
3 Pier shaft							
up to H.F.L	0	2.4	0	0	0	0	0
below H.F.L	52.688	1.4	73.763	0	0	0	0
4 Footing	35.67	1.4	49.938	0	0	0	0
5 Earth above footing	91.348	1	91.348	0	0	0	0
6 Water current			0	0	7.0012	0	19.236
			223.91		**7.0012**		**19.236**

b) Horizantal Forces and Moments withrespect to Base						
1 Longitudinal Forces at bearing level						
	H_L	**H_T**	**e_L**	**M_L**	**e_T**	**M_T**
	7.94	0	7.487	59.456	7.487	0
	7.94	0		**59.456**		**0**

Summary

		Max Reaction case		Max Long.moment case
P	=	609.66	t	599.73
M_L	=	86.405	t-m	93.918
M_T	=	183.42	t-m	151.59

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{609.66}{43.05} + \frac{86.405}{88.253} + \frac{183.42}{25.113} = 22.444 \text{ t/m}^2$$

$< \mathbf{37.6}$ t/m^2

O.K

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{609.66}{43.05} - \frac{86.405}{88.253} - \frac{183.42}{25.113} = 5.8787 \text{ t/m}^2$$

$> \mathbf{0.00}$

O.K.NO.Tension

Max Long

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{599.73}{43.05} + \frac{93.918}{88.253} + \frac{151.59}{25.113} = 21.032 \text{ t/m}^2$$

$< \mathbf{37.6}$ t/m^2

O.K

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{599.73}{43.05} - \frac{93.918}{88.253} - \frac{151.59}{25.113} = 6.8302 \text{ t/m}^2$$

$> \mathbf{0.00}$

O.K.NO.Tension

H.F.L Condition with L.L

a) Vertical load and their moments about Pier Shaft bottom

II	Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1	Pedestal							
a	Left span	0	2.4	0	-0.4	0	0	0
b	Right span	0	2.4	0	0.4	0	0	0
2	Pier cap R	3.69	2.4	8.856	0	0	0	0
	Pier cap Tr	0	2.4	0	0	0	0	0
3	Pier shaft							
	up to H.F.L	0	1.4	0	0	0	0	0
	below H.F.L	52.688	1.4	73.763	0	0	0	0
4	Water current					19.223		6.9967
				82.619		**19.223**		**6.9967**

b) Horizantal Forces and Moments withrespect to Pier Shaft bottom

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	7.94	0	6.487	51.515	6.487	0
	7.94	0		51.515		0

Summary *for design of Pier*

		Max Reaction case		Max Long.moment case	
P	=	468.37	t	458.44	t
M_L	=	90.686	t-m	98.199	t-m
M_T	=	171.18	t-m	139.36	t-m

H.F.L Condition One.Span dislodged

a) Vertical load and their moments about C/L of Foundation base.

II	Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1	Pedestal							
a	Left span	0	2.4	0	-0.4	0	0	0
b	Right span	0	2.4	0	0.4	0	0	0
2	Pier cap R	3.69	2.4	8.856	0	0	0	0
	Pier cap Tr	0	2.4	0	0	0	0	0
3	Pier shaft							
	up to H.F.L	0	2.4	0	0	0	0	0
	below H.F.L	52.688	1.4	73.763	0	0	0	0
4	Footing	35.67	1.4	49.938	0	0	0	0
5	Earth above footing	91.348	1	91.348	0	0	0	0
6	Water current			0	0	7.0012	0	19.236
				223.91		**7.0012**		**19.236**

Summary				
	P	=	524.53	t
	M_L	=	7.0012	t-m
	M_T	=	19.236	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{524.53}{43.05} + \frac{7.0012}{88.253} + \frac{19.236}{25.113} = 13.029 \ t/m^2$$

$< \mathbf{37.6}$ t/m^2

O.K

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{524.53}{43.05} - \frac{7.0012}{88.253} - \frac{19.236}{25.113} = 11.339 \ t/m^2$$

$> \mathbf{0.00}$

O.K.NO.Tension

H.F.L Condition One.Span dislodged

a) Vertical load and their moments about Abutment Shaft bottom.

II Substructure	**V**	ρ	**P**	**e_L**	**M_L**	**e_T**	**M_T**
1 Pedestal							
a Left span	0	2.4	0	-0.4	0	0	0
b Right span	0	2.4	0	0.4	0	0	0
2 Pier cap R	3.69	2.4	8.856	0	0	0	0
Pier cap Tr	0	2.4	0	0	0	0	0
3 Pier shaft							
up to H.F.L	0	2.4	0	0	0	0	0
below H.F.L	52.688	1.4	73.763	0	0	0	0
4 Water current					19.223		6.9967
			82.619		**19.223**		**6.9967**

Summary *for design of Pier*				
	P	=	383.24	t
	M_L	=	19.223	t-m
	M_T	=	6.9967	t-m

Dry Condition with L.L

a) Vertical load and their moments about C/L of Foundation base.

I

	P		e_L		M_L		e_T		M_T
1 D.L Reaction									
a Left span	132.19		-0.4		-52.876		0		0
b Right span	132.19		0.4		52.876		0		0
2 S.I.D.L									
a Left span	18.12		-0.4		-7.248		0		0
b Right span	18.12		0.4		7.248		0		0
	300.62				**0**				**0**
4 **L.L Max Reaction case**									
a Left span									
70R Wheeled + Class A	57.978		-0.4		-23.191		1.76		102.04
Class A 3 Lane	67.5		-0.4		-27		0.7		47.25
b Right span									
70R Wheeled + Class A	35.307		0.4		14.123		1.76		62.14
Class A 3 Lane	17.631		0.4		7.0524		0.7		12.342
4 **L.L Max Longitudinal Moment case**									
a Left span									
70R Wheeled + Class A	71.928		-0.4		-28.771		1.76		126.59
Class A 3 Lane	9.828		-0.4		-3.9312		0.7		6.8796
b Right span									
70R Wheeled + Class A	3.276		0.4		1.3104		1.76		5.7658
Class A 3 Lane	71.82		0.4		28.728		0.7		50.274
Max Reaction case	**385.75**				**19.948**				**164.18**
Max Longitudinal Moment	**375.82**				**27.461**				**132.36**

II Substructure	V	ρ	P	e_L	M_L	e_T	M_T
1 Pedestal							
a Left span	0	2.4	0	-0.4	0	0	0
b Right span	0	2.4	0	0.4	0	0	0
2 Pier cap R	3.69	2.4	8.856	0	0	0	0
Pier cap Tr	0	2.4	0	0	0	0	0
3 Pier shaft							
up to G.L	34.888808	2.4	83.733	0	0	0	0
below G.L	23.656637	2.4	56.776	0	0	0	0
4 Footing	35.67	2.4	85.608	0	0	0	0
5 Earth above footing	91.348363	1.8	164.43	0	0	0	0
			399.4		**0**		**0**

b) Horizantal Forces and Moments withrespect to Base

1 Longitudinal Forces at bearing level

	H_L	H_T	e_L	M_L	e_T	M_T
	7.94	0	7.487	59.456	7.487	0
	7.94	0		**59.456**		**0**

Summary

			Max Reaction		Max Longitudinal
P	=	785.15	t		775.22
M_L	=	79.404	t-m		86.917
M_T	=	164.18	t-m		132.36

A		=	43.05
Z_L		=	88.253
Z_T		=	25.113

Max reaction case:

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{785.15112}{43.05} + \frac{79.404}{88.253} + \frac{164.18}{25.113} = 25.676 \text{ t/m}^2$$

$< \mathbf{37.6}$ t/m^2

O.K

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{785.15112}{43.05} - \frac{79.404}{88.253} - \frac{164.18}{25.113} = 10.801 \text{ t/m}^2$$

$> \mathbf{0.00}$

O.K.NO.Tension

Max Longmoment case:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{775.22412}{43.05} + \frac{86.917}{88.253} + \frac{132.36}{25.113} = 24.263 \text{ t/m}^2$$

$< \mathbf{37.6}$ t/m^2

O.K

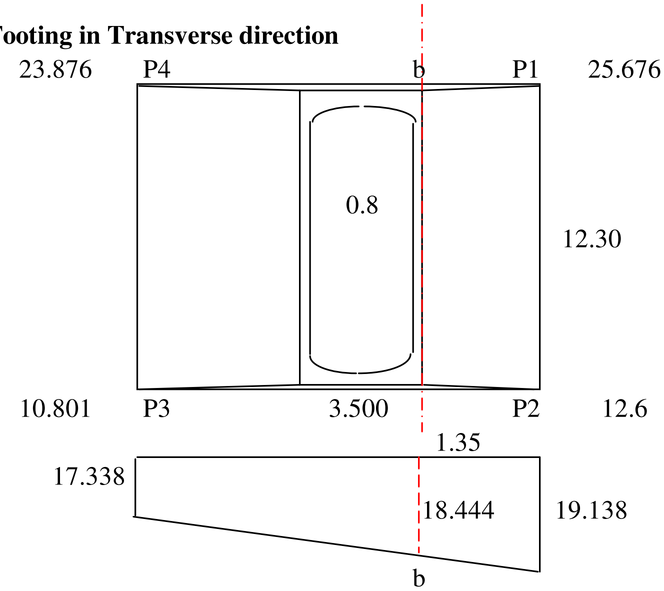
$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{775.22412}{43.05} - \frac{86.917}{88.253} - \frac{132.36}{25.113} = 11.752 \text{ t/m}^2$$

$> \mathbf{0.00}$

O.K.NO.Tension

Design of Footing in Transverse direction



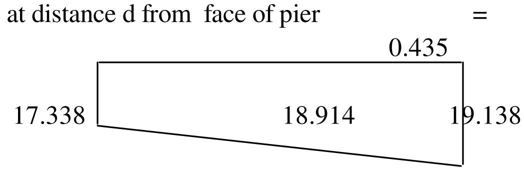
Bending moment at the face of pier b-b = **211.91** t-m

Footing is checked for shear at distance of d from face pier

Effective depth d of footing = 0.915 m

Total depth of section at distance d from pier = 0.6611 m

Effective depth at distance d from face of pier = 0.5701 m



Bending moment at a distance of d from face of pier = 4.2466 t-m

Shear force at a distance d from face of pier = 101.8 t

Net shear force at a distance d from face of pier = **99.04** t

(after correction)

Design for Flexure:

M = 30 Fe = 415

Permissible compressive stress σ_{cbc} = 10 N/mm^2

Permissible tensile stress σ_{st} = 200 N/mm^2

m = 10

k = 0.33

j = 0.89

Q = 1.4815

width b = 12.30

Effective depth required d = 341.02 mm

Effective depth provided = 915 mm

A_{st} Required = 1059.1 mm^2/m

Provide 4 nos 20 dia bars /m width

Check for shear

Shear stress at effective depth from pier	=	14.124	t/m^2	0.1385
% of steel provided	=	0.13		
Permissible shear stress	=	19.38	t/m^2	

Dry Condition with L.L												
a) Vertical load and their moments about Pier Shaft bottom												
II	Substructure	**V**		**ρ**	**P**		**e_L**	**M_L**		**e_T**		**M_T**
1	Pedestal											
a	Left span	0		2.4	0		-0.4	0		0		0
b	Right span	0		2.4	0		0.4	0		0		0
2	Pier cap R	3.69		2.4	8.856		0	0		0		0
	Pier cap Tr	0		2.4	0		0	0		0		0
3	Pier shaft											
	up to G.L	34.888808		2.4	83.733		0	0		0		0
	below G.L	23.656637		2.4	56.776		0	0		0		0
					149.37			**0**				**0**

b) Horizantal Forces and Moments withrespect to PierShaft bottom										
1 Longitudinal Forces at bearing level										
	H_L		**H_T**		**e_L**		**M_L**	**e_T**		**M_T**
	7.94		0		6.487		51.515	6.487		0
	7.94		**0**				**51.515**			**0**

Summary *for design of Pier*

		Max Reaction case		Max Longitudinal moment
P	=	535.12	t	525.19
M_L	=	71.462	t-m	78.976
M_T	=	164.18	t-m	132.36

Dry Condition One.Span dislodged
a) Vertical load and their moments about C/L of Foundation base.

1 Longitudinal Forces at bearing level									
	H_L		**H_T**		**e_L**		**M_L**	**e_T**		**M_T**
	0.00		0		7.487		0	7.487		0
	0.00		0				**0**			**0**

Summary

P		=	700.02	t
M_L		=	0	t-m
M_T		=	0	t-m

Check for Maximum Allowable Base Pressure:

$$P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{700.02012}{43.05} + \frac{0}{88.253} + \frac{0}{25.113} = 16.261 \text{ t/m}^2$$

$$< \mathbf{37.6} \text{ t/m}^2$$

O.K

$$P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{700.02012}{43.05} - \frac{0}{88.253} - \frac{0}{25.113} = 16.261 \text{ t/m}^2$$

$$> \mathbf{0.00}$$

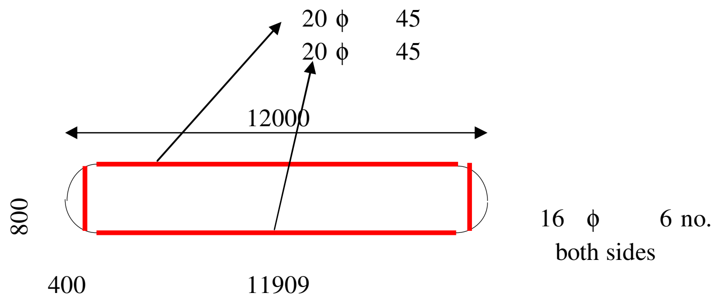
O.K.NO.Tension

Dry Condition One.Span dislodged
a) Vertical load and their moments about Abutment Shaft bottom.
1 Longitudinal Forces at bearing level

	H_L		**H_T**		**e_L**		**M_L**	**e_T**		**M_T**
	0.00		0		6.487		0	6.487		0
	0.00		**0**				**0**			**0**

Summary *for design of Pier*

P		=	449.99	t
M_L		=	0	t-m
M_T		=	0	t-m



PLAN : PIER SHAFT

Reinforcement provided = 30687.077 mm^2

Summary of Loads at Pier Shaft bottom :

1 DRY condition	**P**		**M_L**		**M_T**	
Max Reaction						
With L.L	535.12		71.46		164.18	
Span dislodged	449.99		0.00		0.00	
Max Long						
With L.L	525.19		78.98		132.36	
Span dislodged	449.99		0.00		0.00	
2 H.F.L						
Max Reaction						
With L.L	468.37		90.69		171.18	
Span dislodged	383.24		19.22		7.00	
Max Long						
With L.L	458.44		98.20		139.36	
Span dislodged	383.24		19.22		7.00	

Check for Cracked/Uncracked Section

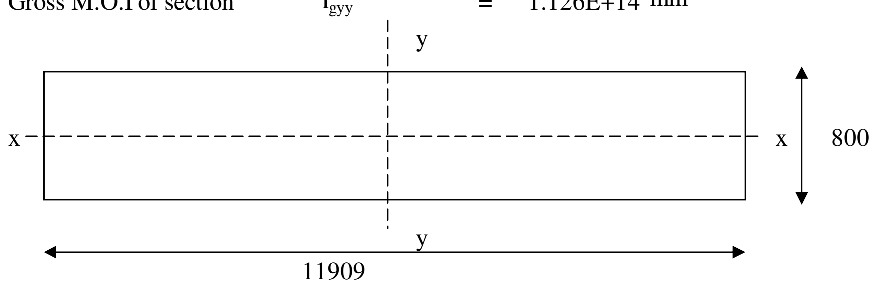
Length of section = 11908.982 mm

Width of section = 800 mm

Gross Area of section A_g = 9527185.2 mm^2

Gross M.O.I of section I_{gxx} = 5.081E+11 mm^4

Gross M.O.I of section I_{gyy} = 1.126E+14 mm^4



Abutment section

Transformed sectional properties of section:

Adopting

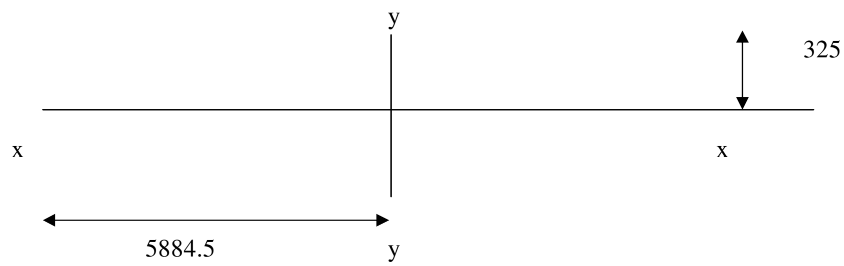
Modular ratio	m	=	10	
Cover			70	70
Dia of Bars		=	20	20
No of bars in tension face (longer)		=	45	
No of bars in compression face		=		45

No of bars in shorter direction = 6

Total bars in section = **102**

Steel Area A_s = 30687 mm^2

% of Steel = 0.3221 %



A_{sx} = 14137 mm^2

A_{sy} = 1885 mm^2

Area of concrete $A_c = A_g - A_s$ = 9496498.2 mm^2

C.G of Steel placed on longer face = 330 mm

C.G of Steel placed on shorter face = 5884.5 mm

Transformed Area of Section A_{tfm} = 9803368.9 mm^2

Transformed M.I_{txx} = I_{gxx} + 2 m - 1 A_{sx} ax^2

= 5.35828E+11 mm^4

$Z_{xx} = \frac{M.Itxx}{d/2}$ = 1.34E+09 mm^3

Transformed M.I_{tyy} = I_{gyy} + 2 m - 1 A_{sy} ay^2

= 1.13773E+14 mm^4

$Z_{yy} = \frac{M.I_{tyy}}{d/2}$ = 1.911E+10 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 5.081E+11 mm^4

Area of section = 9527185.2 mm^2

r = 230.94011 mm

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

Abutment shaft height L = 6.187 m

Effective length L_{eff} = 7.4244 m

Slenderness ratio = 32.149 < **50**

Type of member = 1

1 Short Column

2 Long Column

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

β = 1

Permissible stresses **:** *concrete*

σ_{cbc} = 8.3333 N/mm^2

σ_{co} = 6.25 N/mm^2

Tensile stress = 0.61 N/mm^2

Permissible stresses : *Steel*

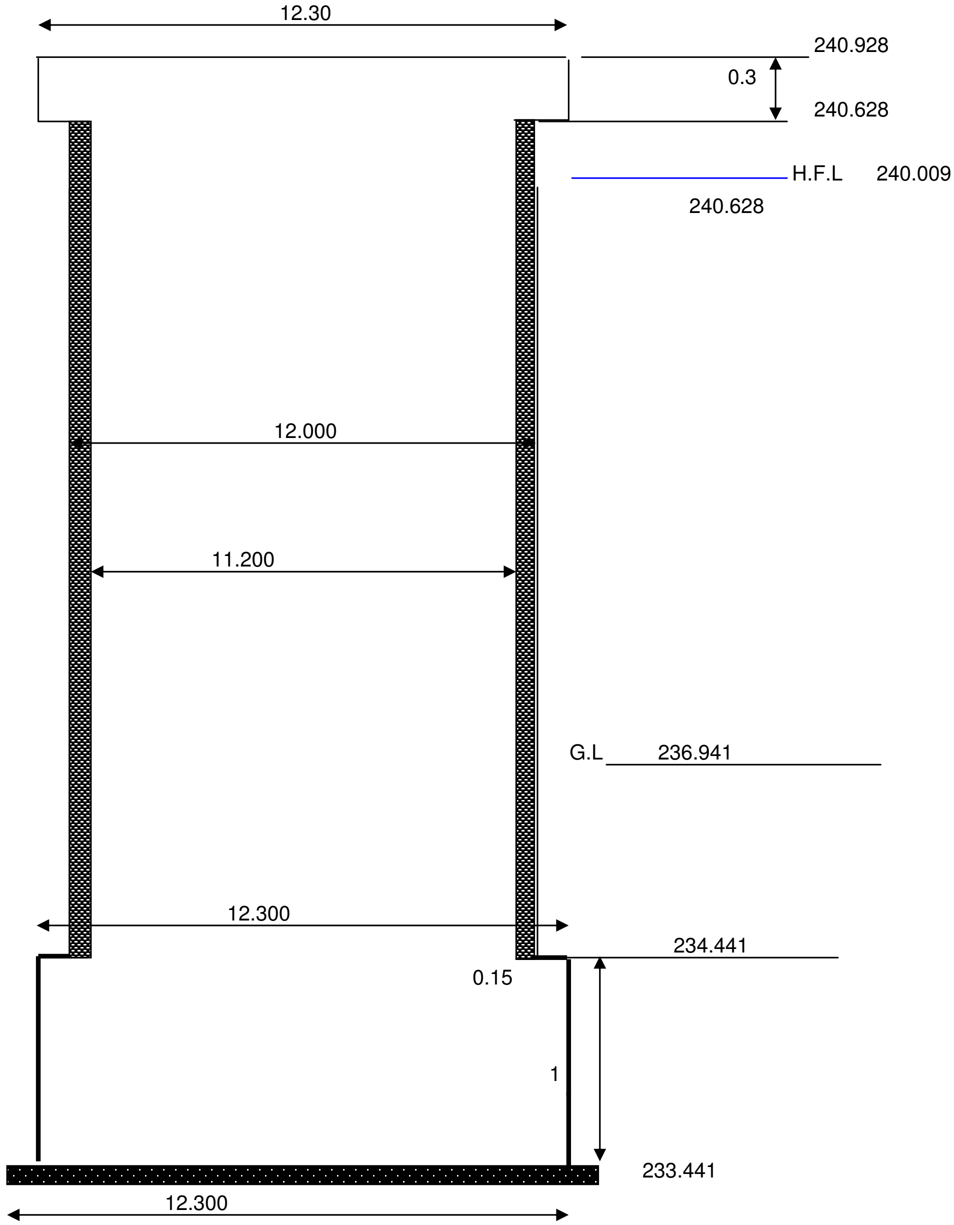
σ_{st} = 200 N/mm^2

S.No	Item	DRY Case (Max Reaction)	
	Loads and Moments	With L.L	Spandislodged
1	P	535.12 t	449.99 t
2	M_L	71.46 t-m	0.00 t-m
3	M_T	164.18 t-m	0.00 t-m
Actual(calculated) Stresses			
4	$\sigma_{co,cal}$ P/A_{tfm}	0.54584916	0.4590106
5	$\sigma_{cbc,cal}$ M_L/Z_{xx}	0.533473125	0
6	$\sigma_{cbc,cal}$ M_T/Z_{yy}	0.085926767	0
7	$\sigma_{cbc,cal}$ = 5 + 6	0.619399891	0
Permissible Stresses			
8	σ_{cbc}	8.33333333	8.3333
9	σ_{co}	6.25	6.25
Check for Minimum steel area mm^2			
10	Conc.Area Required for directstress (1)/(9)	856185.71	719976
11	0.8% of area required	6849.48568	5759.8
12	0.3% of A_g	28581.5557	28582
13	Governing steel mm^2	28581.5557	28582
14	Provided Steel area mm^2	30687.077	30687
Check for safety of section			
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.16166385 < 1	0.0734 < 1
Check for Cracked /Uncracked section			
	$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-0.0735507	0.459
	Permissible Basic tensile stress in concrete	-0.61	-0.61
	Section to be designed as	**Uncracked**	**Uncracked**

S.No	Item		H.F.L Case (Max Reaction)	
	Loads and Moments		With L.L	Spandislodged
1	P		468.37 t	383.24 t
2	M_L		90.69 t-m	19.22 t-m
3	M_T		171.18 t-m	7.00 t-m
Actual(calculated) Stresses				
4	$\sigma_{co,cal}$	P/A_{tfm}	0.477764624	0.3909261
5	$\sigma_{cbc,cal}$	M_L/Z_{xx}	0.676977088	0.143504
6	$\sigma_{cbc,cal}$	M_T/Z_{yy}	0.089588606	0.0036618
7	$\sigma_{cbc,cal}$	= 5 + 6	0.766565695	0.1471658
Permissible Stresses				
8	σ_{cbc}		8.33333333	8.3333
9	σ_{co}		6.25	6.25
Check for Minimum steel area mm^2				
10	Conc.Area Required for directstress	(1)/(9)	749392.459	613183
11	0.8% of area required		5995.13967	4905.5
12	0.3% of A_g		28581.5557	28582
13	Governing steel	mm^2	28581.5557	28582
14	Provided Steel area	mm^2	30687.077	30687
Check for safety of section				
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$		0.16843022 < 1	0.0802 < 1
Check for Cracked /Uncracked section				
	$\sigma_{co,cal} - \sigma_{cbc,cal}$		-0.2888011	0.2438
	Permissible Basic tensile stress in concrete		-0.61	-0.61
	Section to be designed as		**Uncracked**	**Uncracked**

S.No	Item	DRY Case (Max Longitudinal moment)	
	Loads and Moments	With L.L	Spandislodged
1	P	525.19 t	449.99 t
2	M_L	78.98 t-m	0.00 t-m
3	M_T	132.36 t-m	0.00 t-m
Actual(calculated) Stresses			
4	$\sigma_{co,cal}$ P/A_{tfm}	0.535723049	0.459010645
5	$\sigma_{cbc,cal}$ M_L/Z_{xx}	0.589559756	0
6	$\sigma_{cbc,cal}$ M_T/Z_{yy}	0.069271979	0
7	$\sigma_{cbc,cal}$ = 5 + 6	0.658831735	0
Permissible Stresses			
8	σ_{cbc}	8.3333333	8.3333333
9	σ_{co}	6.25	6.25
Check for Minimum steel area mm^2			
10	Conc.Area Required for directstress (1)/(9)	840302.51	719976.11
11	0.8% of area required	6722.4201	5759.8089
12	0.3% of A_g	28581.556	28581.556
13	Governing steel mm^2	28581.556	28581.556
14	Provided Steel area mm^2	30687.077	30687.077
Check for safety of section			
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.1647755 < 1	0.0734417 < < 1
Check for Cracked /Uncracked section			
	$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-0.123109	0.4590106
	Permissible Basic tensile stress in concrete	-0.61	-0.61
	Section to be designed as	**Uncracked**	**Uncracked**

S.No	Item		H.F.L Case (Max Longitudinal)	
	Loads and Moments		With L.L	Spandislodged
1	P		458.44 t	383.24 t
2	M_L		98.20 t-m	19.22 t-m
3	M_T		139.36 t-m	7.00 t-m
Actual(calculated) Stresses				
4	$\sigma_{co,cal}$	P/A_{tfm}	0.467638513	0.390926109
5	$\sigma_{cbc,cal}$	M_L/Z_{xx}	0.73306372	0.143503963
6	$\sigma_{cbc,cal}$	M_T/Z_{yy}	0.072933819	0.00366184
7	$\sigma_{cbc,cal}$	= 5 + 6	0.805997538	0.147165803
Permissible Stresses				
8	σ_{cbc}		8.3333333	8.3333333
9	σ_{co}		6.25	6.25
Check for Minimum steel area mm^2				
10	Conc.Area Required for directstress (1)/(9)		733509.26	613182.86
11	0.8% of area required		5868.0741	4905.4629
12	0.3% of A_g		28581.556	28581.556
13	Governing steel	mm^2	28581.556	28581.556
14	Provided Steel area	mm^2	30687.077	30687.077
Check for safety of section				
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$		0.1715419 < 1	0.0802081 < 1
Check for Cracked /Uncracked section				
	$\sigma_{co,cal} - \sigma_{cbc,cal}$		-0.34	0.2437603
	Permissible Basic tensile stress in concrete		-0.61	-0.61
	Section to be designed as		**Uncracked**	**Uncracked**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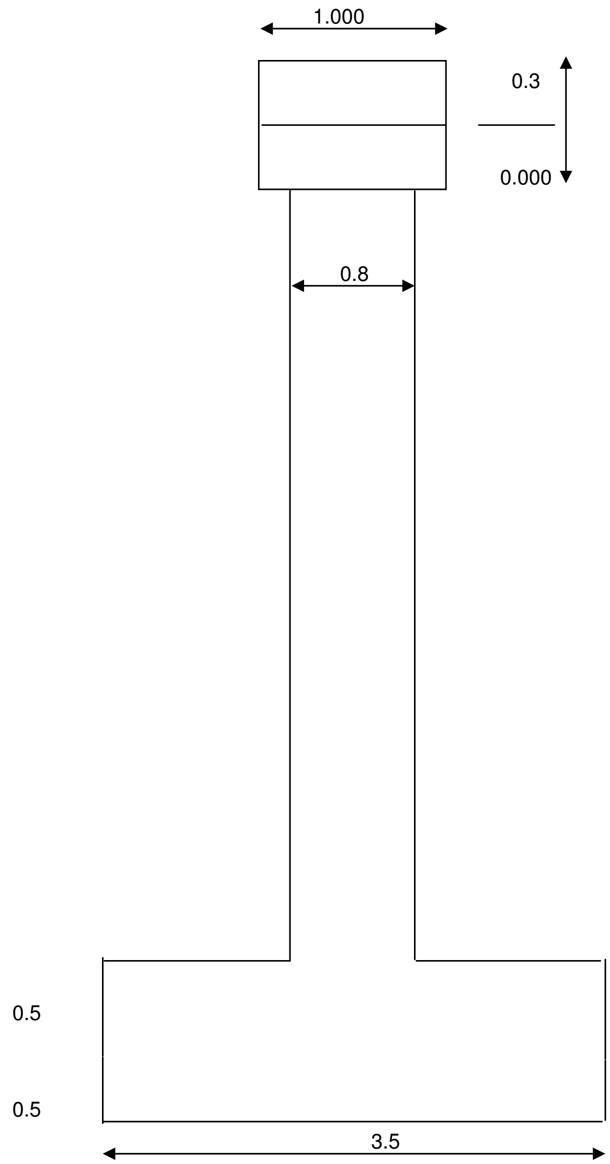
240.928
0.3
240.628
H.F.L 240.009
240.628
12.000
11.200
G.L 236.941
12.300
234.441
0.15
1
233.441
12.300


1.000
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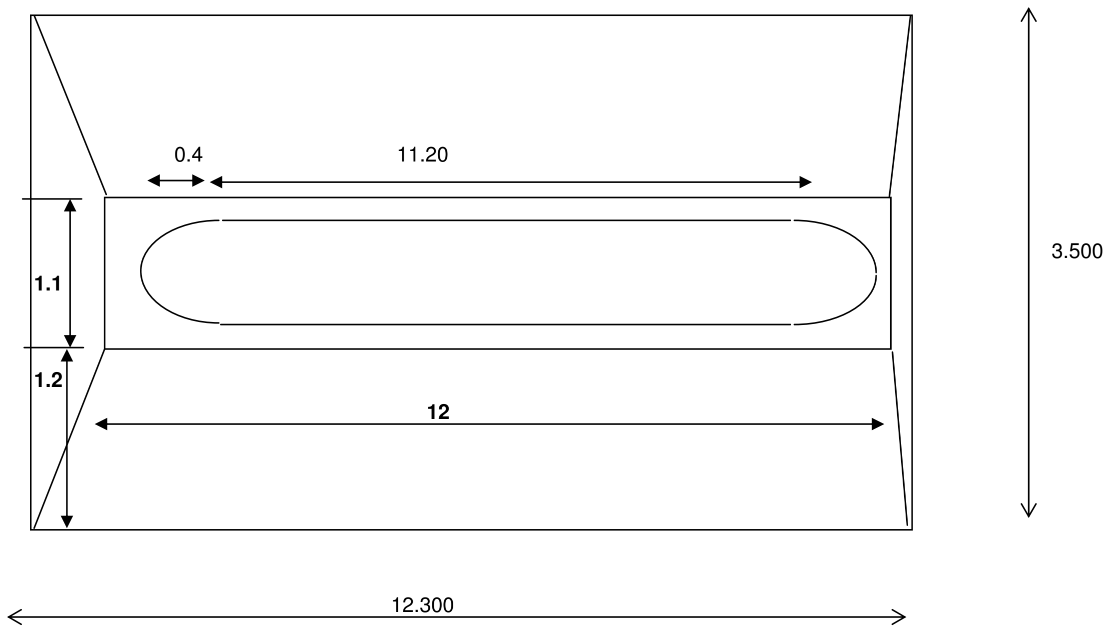
3.5



Dimensions of Substructure & Foundation

1	Pedestal											
			Length			=	0.7					
			Width			=	0.55		Volume	=	0	
			Height			=	0					
2	Pier cap											
a	Top uniform portion											
			Width			=	1.00					
			Depth			=	0.3		Volume	=	3.69	m^3
			Length			=	12.30					
b	Bottom trapezoidal portion											
			Width			=	1					
			Depth			=	0		Volume	=	0	m^3
			Length			=	12.3					
	Area at level	240.628 m	=	12.3	x 1	=	12.3	m^2				
	Area at level	240.628 m	=	1	x 12	=	12	m^2		=	**3.69**	m^3

3	Pier shaft							
	Dimension of pier shaft in longitudinal direction		=	11.20				
	Dimension of pier shaft in Transverse direction	R	=	0.00		Area	=	9.46 m^2
		C	=	0.80				
	Height of pier shaft		=	6.19				
	Height above Ground level		=	3.687	m	Volume	=	34.8888
	Height below Ground level		=	2.500	m	Volume	=	23.6566
							=	**58.5454** m^3
	H.F.L Case:							
	Height above H.F.L		=	0.000	m	Volume	=	0
	Height below H.F.L		=	5.57	m	Volume	=	52.6881
							=	**52.6881** m^3
4	Pedestal at footing top	Width	=	12.3				

	Length	=	12.30		Volume	=	0 m^3
	Height	=	0				

5 Footing

at foundation level

Width	=	12.300	m		
Length	=	3.50	m		6.765
Thickness at Root	=	1	m		21.525
Thickness at Tip	=	0.5	m		7.38

Volume = 35.67 m^3

Volume of Overburdden earth below ground level

Total volume 12.300 x 3.500 x 3.500 = 150.675 m^3

Net volume below Ground level = 91.3484 m^3

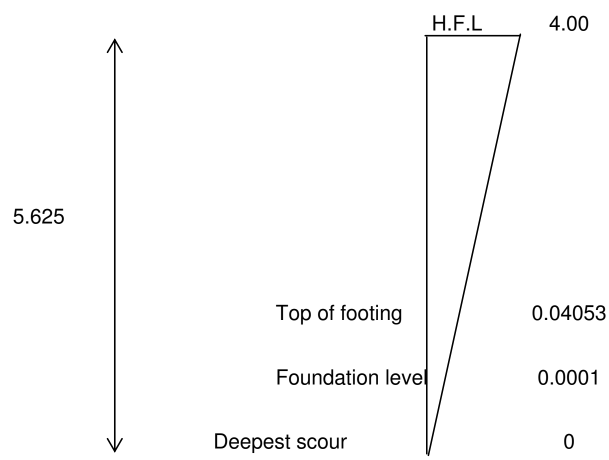
Sectional Properties of Footing

A	=	12.300 x 3.500	=	43.050 m^2
Z_L	=	3.500 x 25.215	=	88.253 m^3
Z_T	=	12.300 x 2.04167	=	25.113 m^3

Forces and Moments due to Watercurrent Force

Intensity of water current pressure = 52 k V^2 kg/m^2

Assuming 20 degree variation in water current direction



Water current Pressure in Transverse direction

	R.L	k	Pressure = 52 k V^2 COS 20
H.F.L	240.009	0.66	129.001003
Pier Shaft bottom	234.441	0.66	1.30721016
Top of footing	234.441	1.5	2.97093219
Bottom of footing	233.441	1.5	0.0073296

Water current Force in Transverse direction

Component	Length	Height	Force	HT of force	Moment	
	m	m	t			
Pier	11.20	5.568	4.06311	4.73119	19.2234	t-m
Top of footing	12.300	1.000	0.01832	0.66585	0.0122	t-m

Water Current Pressure in Longtudinal direction

	R.L	k	Pressure = 52 k V^2 SIN 20
H.F.L	240.009	0.66	46.9525253
Pier Shaft bottom	234.441	0.66	0.47578559
Top of footing	234.441	1.5	1.08133089
Bottom of footing	233.441	1.5	0.00266776

Water current Force in Longtudinal direction

Component	Length m	Height m	Force t	HT of force	Moment
Pier	11.20	5.568	1.47885	4.73119	6.99673 t-m
Top of footing	12.300	1.000	0.00667	0.66585	0.00444 t-m

		F	M
Summary	at Founding level	F	M
	Longitudinal direction	4.08143	19.2355642 t-m
	Transverse direction	1.48552	7.00117281 t-m

		F	M
Summary	at PierShaft bottom level	F	M
	Longitudina direction	4.06311	19.2233683 t-m
	Transverse direction	1.47885	6.99673388 t-m

DESIGN OF PIER CAP

The pier cap is supported by pier shaft on all sides. Therefore pier cap has been designed as per clause 716.2.1 ,I.R.C:78-1987

The pier cap shall be reinforced with a total minimum of 1% steel in both directions as per clause 716.2.1 ,I.R.C:78-1987

Length of pier cap	=	12	m
width of pier cap	=	1	
Depth of pier cap	=	0.3	

Rein forcement in the direction of length of pier

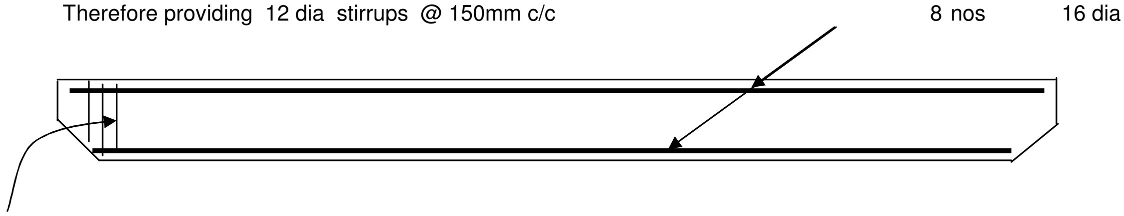
Area of steel required (mm$^{2)}$	=	1500	mm^2	in each face
Providing steel by distributing equally at top& bottom	=	8	16	dia

Area of steel provided (mm^2)	=	1608.495	mm^2

Therefore providing 16dia 8 nos

Reinforcement in the direction of width of pier

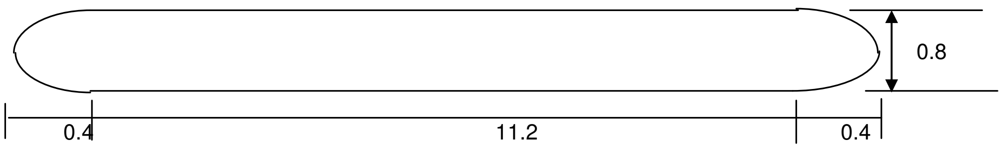
Therefore providing 12 dia stirrups @ 150mm c/c

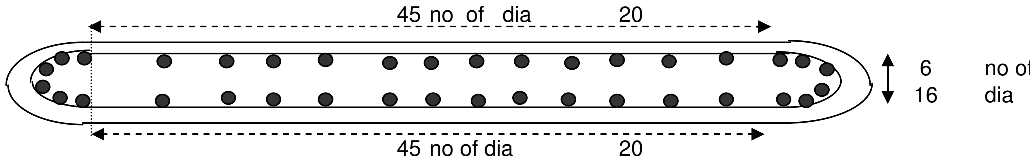


REINFORCEMENT DETAILS OF PIER SHAFT

Design of pier wall

The pier wall on a conservative side has been analysed as a rectangular section (neglecting the semicircular cut & ease water on both end) subjected to combined axial load and biaxial moments with the help of computer programme. The grade of concrete for pier shall be M30.The section of pier is checked conservatively at top of foundation. The minimum area of steel provided is 0.3% of gross crosssectional area as per IRC:21-2000





Minimum Longitudinal Reinforcement required in pier shaft:

0.3% of the gross cross sectional area of pier shaft	=	28800	mm^2
Provided reinforcement	=	30687.08	mm^2

Transverse Reinforcement provided in pier shaft:
(refer clause 306.3 IRC:21-2000)

Thickness of pier (least lateral dimension)	=	800	mm
Dia of largest longitudinal bar	=	20	mm
dia of lateral ties required	=	5	
	but >=	8	mm

The pitch of transverse reinforcement shall not exceed the lesser of the following

1	Least lateral dimension of pier	=	800
2	twelve times dia of smallest longitudinal reinforcement in pier shaft	=	240
3		=	300

Therefore allowable pitch of lateral ties = 240

Provide transverse reinforcement
8 dia of lateral ties 150 mm c/c

SLENDERNESS EFFECT OF PIER WALL

Calculation of slenderness effect of pierwall:

height of pier shaft = h		=	6.187	
effective length of pier wall under service condition = 1.2 h (refer clause 306.1 IRC:21-2000)		=	7.4244	
Thickness of pier wall		=	0.8	
Radius of gyration = r	d/sqrt(12)	=	0.23094	
Slenderness ratio of pier wall 1.2*h/r		=	32.1486	< 50 **shortcolumn effect predominant**

Since the pier shaft is acting as a short column, The reduction coefficientis 1
I.e β 1

Reduced value of permissible stresses for steel and concrete are:

Grade of concrete = M30 30

concrete = 1020 t/m^2
steel = 20400 t/m^2

FOR ONE SPAN DISLODGED CONDITION :

Effective length of pier wall under service condition is	1.75*h	=	10.82725	
Slenderness ratio		=	46.88337	< 50 **shortcolumn effect predominant**

Since the pier shaft is acting as a short column, The reduction coefficientis 1
I.e β 1

β = 1.5 - l/100r

= 1.031166

Reduced value of permissible stresses for steel and concrete are:

Grade of concrete = M30 30

concrete = 1051.79 t/m^2
steel = 21035.79 t/m^2

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     8. 6 11.2 0 0;7 21.2 0 0;8 21.58 0 0
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    10. 1 1 2 7
    11. MEMBER PROPERTY CANADIAN
    12. 1 TO 7 PRI YD 1.0 ZD 1.0
    13. CONSTANT
    14. E CONCRETE ALL
    15. DENSITY CONCRETE ALL
    16. POISSON CONCRETE ALL
    17. SUPPORT
    18. 2 3 6 7   PINNED
    19. MEMBER RELEASE
    20. 4 START FX FY FZ MX MY MZ
    21. 4 END  FX FY FZ MX MY MZ
    22. DEFINE MOVING LOAD
    23. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    24. LOAD GENERATION 175
    25. TYPE 1 -13.4 0. 0. XINC .2
    26. PERFORM ANALYSIS
    27. ***MAX REA
    28. LOAD LIST 78
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      2   78      0.00     16.89      0.00      0.00      0.00      0.00
      3   78      0.00     32.11      0.00      0.00      0.00      0.00
      6   78      0.00     39.42      0.00      0.00      0.00      0.00
      7   78      0.00     11.58      0.00      0.00      0.00      0.00
    30. ***MAX MOM
    31. LOAD LIST 54
    32. PRINT SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z
      2   54      0.00     38.68      0.00      0.00      0.00      0.00
      3   54      0.00     53.32      0.00      0.00      0.00      0.00
      6   54      0.00      0.00      0.00      0.00      0.00      0.00
      7   54      0.00      0.00      0.00      0.00      0.00      0.00

    33. FINISH
```

```
INPUT FILE: CLASS A.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.02 0 0;2 0.4 0 0;3 10.4 0 0;4 10.78 0 0;5 10.82 0 0
     8. 6 11.2 0 0;7 21.2 0 0;8 21.58 0 0
     9. MEMBER INCIDENCES
    10. 1 1 2 7
    11. MEMBER PROPERTY CANADIAN
    12. 1 TO 7 PRI YD 1.0 ZD 1.0
    13. CONSTANT
    14. E CONCRETE ALL
    15. DENSITY CONCRETE ALL
    16. POISSON CONCRETE ALL
    17. SUPPORT
    18. 2 3 6 7   PINNED
    19. MEMBER RELEASE
    20. 4 START FX FY FZ MX MY MZ
    21. 4 END  FX FY FZ MX MY MZ
    22. DEFINE MOVING LOAD
    23. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    24. LOAD GENERATION 202
    25. TYPE 1 -18.8 0. 0. XINC .2
    26. PERFORM ANALYSIS
    27. ***MAX REA
    28. LOAD LIST 121
    29. PRINT SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE

 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z
     2  121      0.00      3.20      0.00      0.00      0.00      0.00
     3  121      0.00     25.00      0.00      0.00      0.00      0.00
     6  121      0.00      6.53      0.00      0.00      0.00      0.00
     7  121      0.00     13.87      0.00      0.00      0.00      0.00
       30. ***MAX MOMENT
    31. LOAD LIST 128
    32. PRINT SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE

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

     2  128      0.00      1.75      0.00      0.00      0.00      0.00
     3  128      0.00      3.64      0.00      0.00      0.00      0.00
     6  128      0.00     26.60      0.00      0.00      0.00      0.00
     7  128      0.00      9.80      0.00      0.00      0.00      0.00


    33. FINISH
```

DESIGN OF SUPERSTRUCTURE

For Design of superstructure of solid slab of 10.0 m refer MOST STANDARD Drawing titled “STANDARD PLANS FOR 3.0 m TO 10.0 M SPAN REINFORCED CEMENT CONCRETE (Solid slab superstructure With & without footpaths)) FOR HIGHWAYS” Drg. No. SD/114.

BRIDGE AT CH:58+900

DESIGN OF SUBSTRUCTURE

DESIGN DATA

Formation Level	=	245.451	m
Ground Level	=	239.785	m
Lowest Water Level	=	235.785	m
Highest Flood Level	=	242.295	m
Founding Level	=	235.785	m
Thickness of bearing & pedestal	=	0.300	m
Width of abutment	=	12.000	m
Bouyancy factor	=	1.0	
Gross Safe Bearing Capacity	=	32.000	t/sqm
Dry density of earth	=	1.800	t/cum
Submerged density of earth	=	1.0	t/cum
Saturated density of earth	=	2.000	t/cum
Coefficient of base friction	=	0.5	
Span (c/c of exp. joint)	=	21.600	m
Overall Width of deck slab	=	12.000	m
Width of carriageway	=	11.000	m
Width of crash barrier	=	0.500	m
Depth of Superstructure	=	2.100	m
Tkickness of wearing coat	=	0.056	m
Unit wt of concrete	=	2.400	t/m^3
no. of elastomeric bearing	=	4	
size of elastomer brgs.(500 x 250	x	50	mm)
Grade of Concrete	-	M 25	25
Grade of Reinforcement	-	415 (HYSD)	
Live Load	-	One Lane of 70R Wheeled + Class A	
	-	3 lanes of Class A	
Permissible Compressive stress in Concrete	-	850	t/m^2
Permissible Tensile stress in Steel	-	20400	t/m^2
Modular ratio, m	-	10	
factor, k	-	0.294	
Lever arm factor, j	-	0.902	
Moment of Resistance	-	113	t/m^2
Thickness of returnwall		0.5	m

COEFFICIENT OF ACTIVE EARTH PRESSURE

AS PER COULOMB'S THEORY, COEFFICIENT OF ACTIVE EARTH PRESSURE IS

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

WHERE

ϕ = ANGLE OF INTERNAL FRICTION OF EARTH

α = ANGLE OF INCLINATION OF BACK OF WALL

δ = ANGLE OF INTERNAL FRICTION BETWEEN WALL & EARTH

ι = ANGLE OF INCLINATION OF BACKFILL

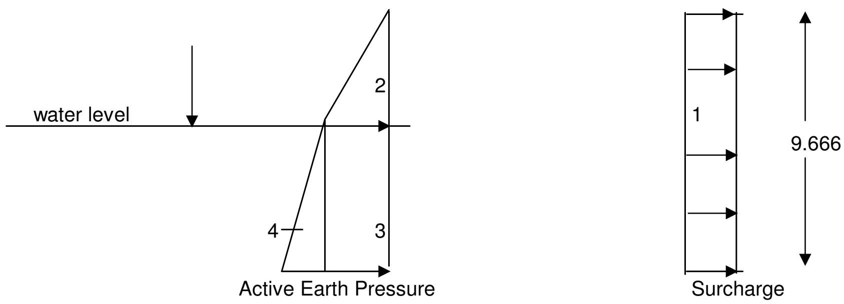
HERE	ϕ =	30 °	=	0.524	Radian
	α =	90 °	=	1.571	Radian
	δ =	20 °	=	0.349	Radian
	ι =	0 °	=	0	Radian
			K_a =	0.2973	

Therefore, Horizontal coefficient of Active earth pressure = **$K_a COS\phi$ = K_{ha} = 0.2794**

HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 9.666 m
For DESIGN purpose, the height of abutment is considered as, say, = **9.670 m**

CALCULATION OF ACTIVE EARTH PRESSURE



DRY. condition

a) Service Condition

Element No.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	77.78	4.833	375.89
2	Dry Earth	0.5	9.666	4.86	281.92	4.060	1144.50
3		1.0	0.000	4.86	0.00	0.000	0.00
4	SubmgEarth	0.5	0.000	0.00	0.00	0.000	0.00
TOTAL					**359.69**		**1520.38**

b) Span Dislodge Condition

Net force = 359.69 - 77.78 = **281.92** t
Net moment = 1520.38 - 375.89 = **1144.50** tm

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.67	77.78	4.833	375.89
2	Dry Earth	0.5	3.156	1.76	33.39	7.836	261.65
3		1.0	6.510	1.76	137.76	3.255	448.42
4	SubmgEarth	0.5	6.510	1.82	71.04	2.170	154.16
TOTAL					**319.97**		**1240.12**

b) Span Dislodge Condition

Net force = 319.97 - 77.78 = **242.20** t
Net moment= 1240.12 - 375.89 = **864.23** tm

Forces & moments due to Abutment (Concrete) components

DRY Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment about toe
1	Toe Slab	1.0	2.200	12.00	0.500	2.40	31.68	1.100	34.85
2		0.5	2.200	12.00	0.700	2.40	22.18	1.467	32.52
3	Heel Slab	1.0	3.600	12.00	0.500	2.40	51.84	5.200	269.57
4		0.5	3.600	12.00	0.700	2.40	36.29	4.600	166.92
5	Stem Wall	1.0	1.200	12.00	6.910	2.40	238.81	2.800	668.67
6		0.5	0.000	12.00	6.910	2.40	0.00	2.200	0.00
7	stem rect	1.0	1.200	12.00	1.200	2.40	41.47	2.800	116.12
10	Cap	1.0	1.200	12.00	0.300	2.40	10.37	2.800	29.03
11	Dirt Wall	1.0	0.300	12.00	2.106	2.40	18.20	3.250	59.14
8	Retutnwall	0.5	3.600	1.00	0.700	2.40	3.02	5.800	17.54
8a		1.0	3.600	1.00	9.166	2.40	79.19	5.200	411.81
TOTAL							**533.05**		**1806.17**

H.F.L. Case :

Element No.	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment ,@ Toe
1	Toe Slab	1.0	2.200	12.00	0.500	1.40	18.48	1.100	20.33
2		0.5	2.200	12.00	0.700	1.40	12.94	1.467	18.97
3	Heel Slab	1.0	3.600	12.00	0.500	1.40	30.24	5.200	157.25
4		0.5	3.600	12.00	0.700	1.40	21.17	4.600	97.37
5	Stem Wall	1.0	1.200	12.00	5.310	1.40	107.05	2.800	299.74
6		0.5	0.000	12.00	5.310	1.40	0.00	2.200	0.00
5	Stem Wall	1.0	1.200	12.00	0.400	2.40	13.82	2.800	38.71
6		0.5	0.000	12.00	0.400	2.40	0.00	2.200	0.00
7	stem rect	1.0	1.200	12.00	1.200	1.40	24.19	2.800	67.74
10	Cap	1.0	1.200	12.00	0.30	2.40	10.37	2.800	29.03
11	Dirt Wall	1.0	0.300	12.00	2.11	2.40	18.20	3.250	59.14
8	Returnwall	0.5	3.600	1.00	0.70	1.40	1.76	5.800	10.23
8c		1.0	3.600	1.00	5.31	1.40	26.76	5.200	139.16
8d		1.0	3.600	1.00	3.16	2.40	27.27	5.200	141.79
TOTAL							**312.25**		**1079.46**

Forces & moments due to Earth and LL surcharge

DRY Case : **Self weight of Earth**

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@ Toe
8	DRY EARTH	0.5	3.600	11.00	0.700	1.8	24.95	5.800	144.70
8a		1.0	3.600	11.00	8.466	1.800	603.46	5.200	3137.97
TOTAL							**628.40**		**3282.67**

H.F.L Case :

Element No	Component	Area Factor	Length (m)	Width (m)	Height (m)	Density (t/m^3)	Weight (t)	C.G.from toe (m)	Moment `@Toe
8	SATURATED SOIL	0.5	3.600	11.000	0.7	1.000	13.86	5.800	80.39
8c		1.0	3.600	11.000	5.31	1.000	210.28	5.2	1093.44
8d	DRY	1.0	3.600	11.000	3.156	1.800	224.96	5.200	1169.79
TOTAL							**449.10**		**2343.61**

L.L.SURCHARGE

Element No	Component	Area Factor	Length	Width	Height	Density	Weight	C.G.from toe	Moment `@ Toe
12	DRY EARTH	0.00	3.600	11.00	1.20	1.80	**0.00**	5.200	**0.00**

SUMMARY OF FORCES AND MOMENTS:

LOAD CASE	Case. L.W.L.		Case. H.F.L.		
	Service Cond.	Span dislodged	Service Cond.	Span dislodged	
Vertical load from superstructure including LL	**334.81**	0.00	**334.81**	0.00	
Vertical load from substructure (b)	1161.45	1161.45	761.34	761.34	
Total Vertical Load V = (a) + (b)	1496.27	1161.45	1096.16	761.34	
Total Horizontal Force H =	**377.69**	281.92	**337.97**	242.20	
Moment @ toe due to (a)	**937.48**	0.00	**937.48**	0.00	
Moment @ toe due to (b)	5088.84	5088.84	3423.07	3423.07	
Total Moment @ toe (M)	6026.32	5088.84	4360.55	3423.07	
Dist. of C.G. of V from toe Z = M/V	4.028	4.38	3.98	4.50	
eccentricity (e = Z -b/2)	0.528	0.881	0.478	0.996	
Relieving Moment @ c/l base (M1)	**789.39**	**1023.76**	**524.00**	**758.37**	
overturning moment due to					
Horz. braking force	**141.48**	0.00	**141.48**	0.00	
Earth Pressure	1520.38	1144.50	1240.12	864.23	
Total overturning Moment (M2)	**1661.86**	**1144.50**	**1381.60**	**864.23**	
Net moment (M2-M1) = M_L	**872.47**	**120.74**	**857.59**	**105.86**	
Factor of Safety					
Against overturning (M / M2)	**3.63**	**4.45**	**3.16**	**3.96**	Safe against overturning
Against sliding (µ x V / H)	**1.981**	**2.060**	**1.622**	**1.572**	Safe against sliding

I.R.C 78-2000:cl 706.3.4

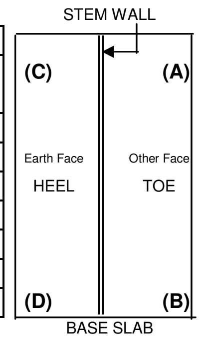
Area of base (A) = 7.000 x 12.00 = 84.00 m^2

Z_L = 98.00 m^3

Z_T = 168.00 m^3

CHECK FOR BASE PRESSURE:

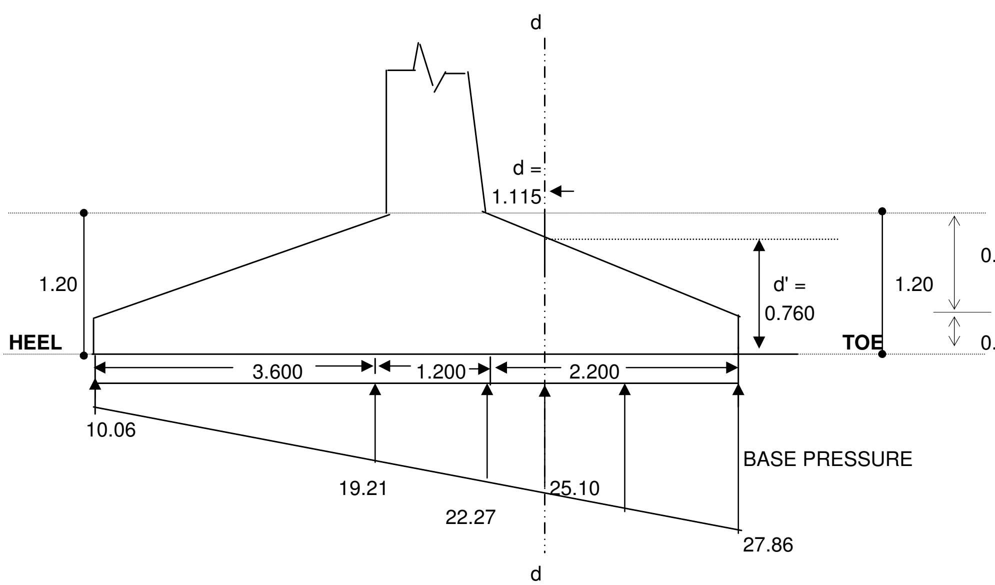
Base Pressure		LWL CASE		HFL CASE	
		Service Cond.	Span dislodged	Service Cond.	Span dislodged
	P/A	17.81	13.83	13.05	9.06
	M_L/Z_L	8.90	1.23	8.75	1.08
	M_T/Z_T	1.15	0.00	1.15	0.00
(A)	$(P/A + M_L/Z_L + M_T/Z_T)$	**27.86**	**15.06**	**22.95**	**10.14**
(B)	$(P/A + M_L/Z_L - M_T/Z_T)$	25.57	15.06	20.65	10.14
(C)	$(P/A - M_L/Z_L + M_T/Z_T)$	10.06	12.59	5.45	7.98
(D)	$(P/A - M_L/Z_L - M_T/Z_T)$	**7.762**	**12.59**	**3.15**	**7.98**



Max.Base Pressure = **27.86** **t/m^2 <** **32.00** **Hence O.K.**

Min. Base Pressure = **3.15** **t/m^2 >** **0** **Hence O.K.**

DESIGN OF TOE SLAB



BENDING MOMENT AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward Loads	1	1.0	2.640	1.100	2.904
	2	0.5	1.848	0.733	1.355
Upward Base pressure	Rect.	1.0	-48.988	1.100	-53.886
	Trian.	0.5	-6.156	1.467	-9.028
TOTAL			-50.655		-58.655

Bending Moment at face of stem 58.655 tm/m

Effective depth required 0.721 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 2859 mm^2

Minimum steel required 0.15% 1673 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 0.06% 669 mm^2/m
mainsteel 2859 mm^2 12 ϕ , @ 150 C/C
753.9822

Hence provide , 25 ϕ , @ 150 C/C
0 ϕ , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.
Minimum steel reinfocement as per above clause 250 mm^2/m
provide 12 ϕ , @ 150 C/C
753.9822

Check for Shear

SHEAR FORCE AT "d" FROM FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	Moment
Downward loads	1	1.0	1.302	0.543	0.706
	2	0.5	0.911	0.362	0.330
Upward base pressure	Rect.	1.0	-27.237	0.543	-14.776
	Trian.	0.5	-1.497	0.723	-1.083
TOTAL			-26.521		-14.823

Effective depth (d') at distence d 0.760 m

Shear force at critical section 26.5 t

Bending Moment at critical section 14.82 tm

tan β = 0.35

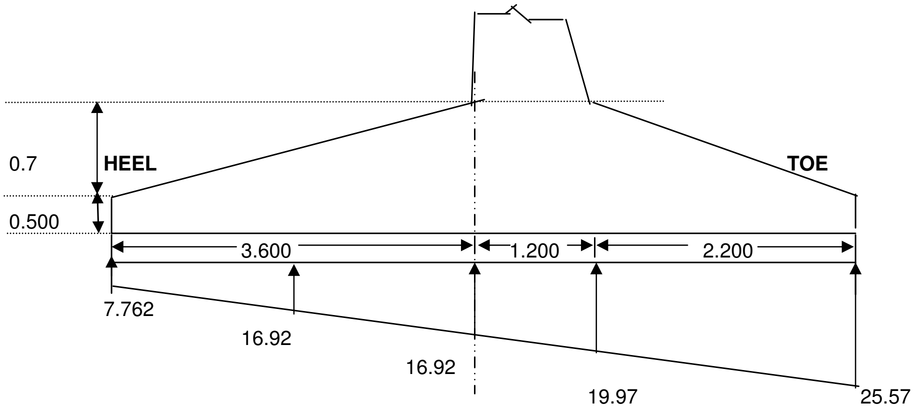
Net shear force S-M*tanβ/d' **19.78** t

Hence, Shear stress 26.02 t/m^2

% of reinforcement 0.43

Permissible shear stress 29.33 t/m^2 **Hence O.K.**

DESIGN OF HEEL SLAB



BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

Loadings	Element	Area fact.	force	L.A.	moment
Downward Loads.	3	1	4.320	1.800	7.776
	4	0.5	3.024	1.200	3.629
	8	0.5	4.536	2.400	10.886
	8a	1	54.860	1.800	98.747
		0.5	4.536	2.400	5.443
		1	6.852	1.800	12.333
Upward base pressure	Rect.	1	-27.944	1.800	-50.300
	Trian.	0.5	-16.483	1.200	-19.779
TOTAL			33.700		68.736

Bending Moment at face of stem **68.736** tm

Effective depth required 0.781 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3352.42 mm^2
Minimum steel 0.15% 1672.50 mm^2 I.R.C 78-2000 Clause:707.2.7

Distribution steel 0.06% 669.00 mm^2/m
mainsteel 3352.42 mm^2

12 ϕ , @ 150 C/C

Hence provide , 25 ϕ , @ 125 C/C 753.9822

0 ϕ , @ 125 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinfocement as per above clause 250 mm^2/m

provide 12 ϕ , @ 150 C/C

753.9822

Check for Shear (Critical section at face of stem)

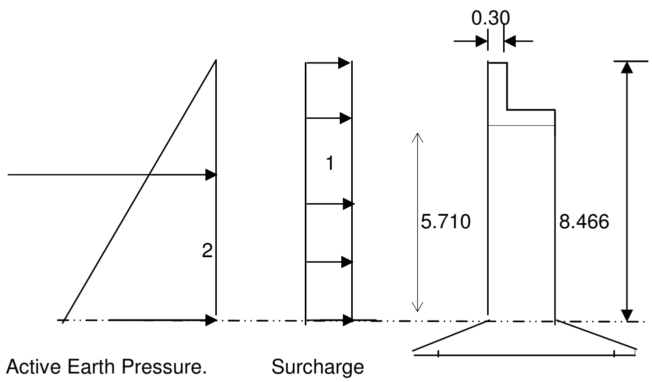
Shear force at face of stem 33.70 t

tan β 0.194
Bending moment at face of stem 68.736 tm
Net shear force S-M*tanβ /d 21.71 t

Hence, Shear stress 19.47 t/m^2
% of reinforcement 0.30

Permissible shear stress 25.10 t/m^2 **Hence O.K.**

DESIGN OF STEM WALL



SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT

R.L. OF SECTION = 236.985 m

DRY Condition

a) Service Condition

Element No.	Area factor	Height of E.P. diagram	Earth Pressure	Force	L.A.	Moment tm
1	1	8.116	0.603	58.8	4.058	238.50
2	0.5	8.116	4.081	198.8	3.409	677.49
HORIZONTAL FORCE				18.00	14.600	262.80
TOTAL				275.52		**1178.79**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 197.338 + 18.20 + 10.37 + 334.81
= 560.716 t
Longitudinal Moment = 1178.787 t-m
Transverse Moment = 192.801 t-m

b) Span Dislodged case

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 197.338 + 18.20 + 10.37 + 0.00
= 225.901 t
Longitudinal Moment = 915.987 t-m
Transverse Moment = 0.000 t-m

H.F.L. condition

a) Service Condition

Element no.	Component	Area factor	Height (m)	Pressure (t/m^2)	Force (t)	C.G. from base (m)	Moment (tm)
1	LL Surcharge	1.0	1.200	0.603	61.31	4.058	248.79
2	Dry Earth	0.5	2.750	1.54	25.35	6.227	157.87
3		1.0	5.310	1.54	97.91	2.655	259.96
4	SubmgEarth	0.5	5.310	1.48	47.27	1.770	83.66
TOTAL					**231.84**		**750.275**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 120.874 + 18.20 + 10.37 + 334.81
= 484.252 t
Longitudinal Moment = 1013.075 t-m
Transverse Moment = 192.801 t-m

b) Span Dislodged case

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 120.874 + 18.20 + 10.37 + 0.00
= 149.437 t
Longitudinal Moment = 750.275 t-m
Transverse Moment = 0.000 t-m

Cross Sectional area 1.2 m^2
For horizontal reinforcement area of steel required for the stem at the section/metre = 194.6531 mm^2

Providing **12** ,@ 581.02 c/c say, **150** C/C as horizontal reinforcement

Movement of Deck:

Total Length of Bridge = 21.6 m

= 0.0005 x 10800 = 5.4 mm

Longitudinal Force :

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

Span dislodged condition

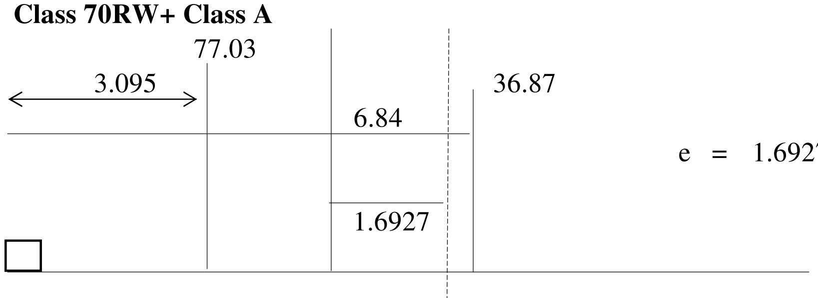
Longitudinal horizontal force = $\frac{0.000}{2}$ + 6.358 = 6.358 t Say, 7.0 t

Summary of Longitudinal Forces :

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

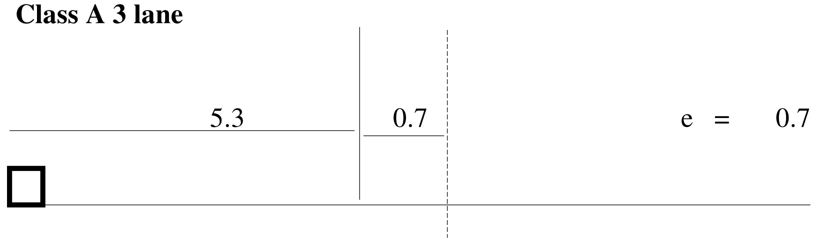
Transverse eccentricity

Class 70RW+ Class A



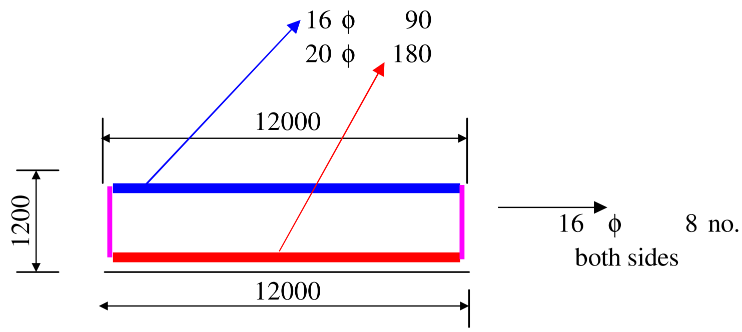
e = 1.6927

Class A 3 lane



Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

	P	M_L	M_T
Dead Load Reaction:			
	184.71	t	
SIDL			
	36.20	t	
L.L Max Reaction Case			
70R + Class A	113.9	t	192.80
Class A 3 Lane	110.61	t	77.43
	334.81		192.80



PLAN : ABUTMENT SHAFT

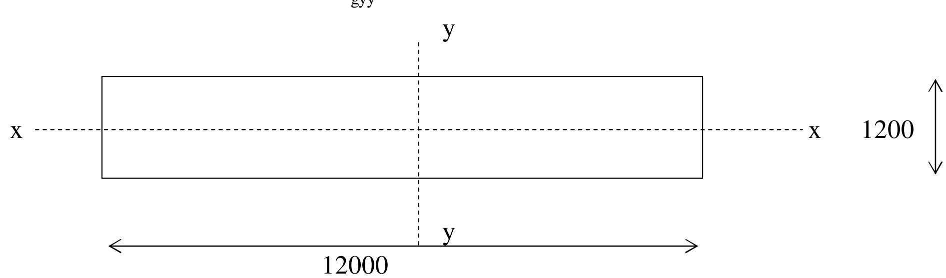
% Reinforcement provided = 77861.232 mm^2

Summary of Loads at Abutment Shaft bottom :

1 DRY condition	P	M_L	M_T
With L.L	560.72	1178.79	192.80

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}	=	1.728E+12	mm^4
Gross M.O.I of section	I_{gyy}	=	1.728E+14	mm^4



Abutment section

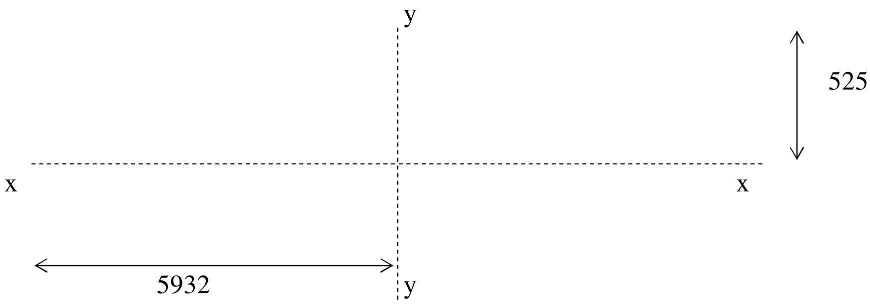
Transformed sectional properties of section:

Adopting

Modular ratio	m	=	10	
Cover			70	68
Dia of Bars		=	20	16
No of bars in tension face (longer)		=	180	
No of bars in compression face		=		90
No of bars in shorter direction		=		8
Total bars in section		=	286	

Steel Area A_s = 77861 mm^2

% of Steel = 0.5407 %



A_{sx} = 56549 mm^2

A_{sy} = 1608.5 mm^2

Area of concrete $A_c = A_g - A_s$ = 14322139 mm^2

C.G of Steel placed on longer face = 530 mm

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

Transformed Area of Section A_{tfm} = 15100751 mm^2

Transformed M.I_{txx} = $I_{gxx} + 2\,[m - 1]\,A_{sx}\,ax^2$

= 2.01392E+12 mm^4

$Z_{xx} = \dfrac{M.Itxx}{d/2}$ = 3.357E+09 mm^3

Transformed M.I_{tyy} = $I_{gyy} + 2\,[m - 1]\,A_{sy}\,ay^2$

= 1.73819E+14 mm^4

$Z_{yy} = \dfrac{M.I_{tyy}}{d/2}$ = 2.897E+10 mm^3

Permissible stresses

Minimum Gross Moment of inertia I_{min} = 1.728E+12 mm^4

Area of section = 14400000 mm^2

r = 346.41016 mm

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

Abutment shaft height L = 9.426 m

Effective length L_{eff} = 11.311 m

Slenderness ratio = 32.653 < **50**

Type of member = 1

1 Short Column

Stress reduction coefficient (IRC:21-2000 cl: 306.4.2,3) 2 Long Column

β = 1

Permissible stresses : *concrete*

σ_{cbc} = 8.3333 N/mm^2

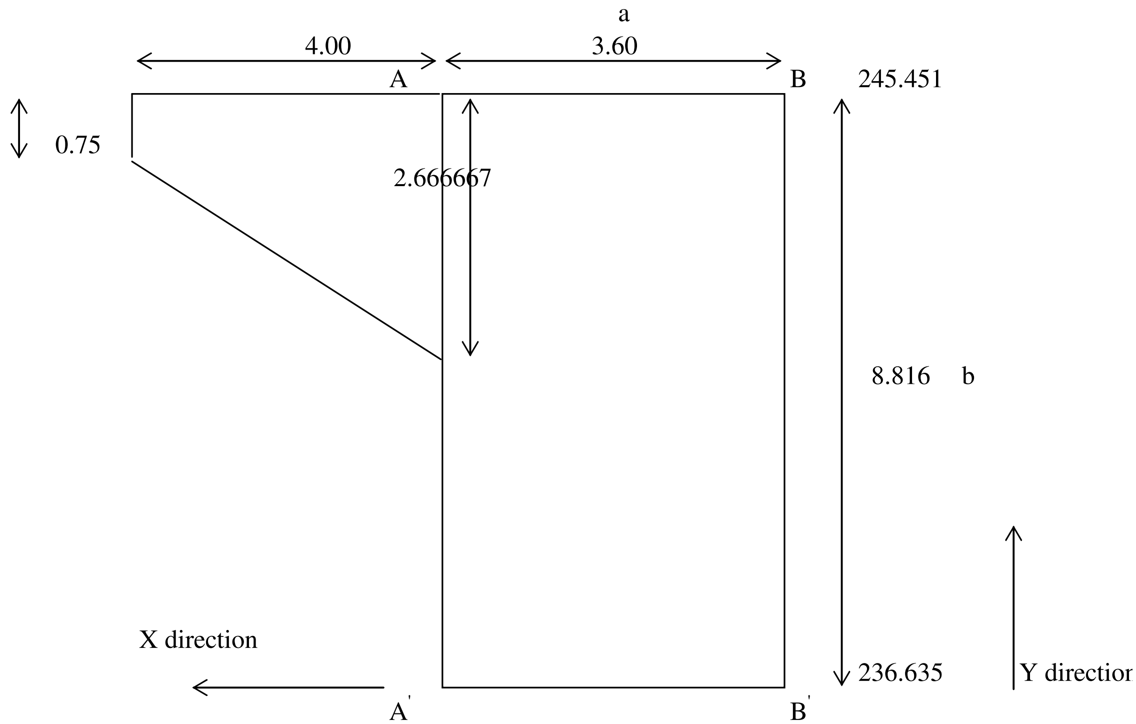
σ_{co} = 6.25 N/mm^2

Tensile stress = 0.67 N/mm^2

Permissible stresses : *Steel*

σ_{st} = 200 N/mm^2

S.No	Item	DRY Case
	Loads and Moments	With L.L
1	P	560.72 t
2	M_L	1178.79 t-m
3	M_T	192.80 t-m
Actual(calculated) Stresses		
4	$\sigma_{co,cal}$ P/A_{tfm}	0.37131642
5	$\sigma_{cbc,cal}$ M_L/Z_{xx}	3.511914483
6	$\sigma_{cbc,cal}$ M_T/Z_{yy}	0.066552525
7	$\sigma_{cbc,cal}$ = 5 + 6	3.578467008
Permissible Stresses		
8	σ_{cbc}	8.3333333
9	σ_{co}	6.25
Check for Minimum steel area mm^2		
10	Conc.Area Required for directstress	
	(1)/(9)	897145.09
11	0.8% of area required	7177.1607
12	0.3% of A_g	43200
13	Governing steel mm^2	43200
14	Provided Steel area mm^2	77861.232
Check for safety of section		
	$\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}}$	0.4888267 < 1
Check for Cracked /Uncracked section		
	$\sigma_{co,cal}$ - $\sigma_{cbc,cal}$	-3.207151
	Permissible Basic tensile stress in concrete	-0.67
	Section to be designed as	**Cracked**



Width of Solid return wall (a)		=	3.60	
Width of Cantilever return wall		=	4.00	
Avg Height of Solid return wall (b)		=	8.816	
Height of Cantilever return at Tip		=	0.75	
Height of Cantilever return at Root		=	2.666667	
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	=	0.333333	
	j	=	0.888889	
	R	=	151.1111	t/m^2

Case (1) For uniformly distributed load over entire plate

a/b	=	0.4083485	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 = 0.4083485 β_1 = 0.427167

β_2 = 0.460428

Live Load Surcharge:

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

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

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

$$\sigma_{bmax} = \frac{0.427167 \times 0.603504 \times 77.72}{0.25} = 80.14584 \text{ t/m}^2$$

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 $= 80.145837 \times 0.041667 = 3.33941$ t-m/m

$$\sigma_{amax} = \frac{0.4604283 \times 0.603504 \times 77.72}{0.25} = 86.38639 \text{ t/m}^2$$

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 $= 86.386391 \times 0.041667 = 3.599433$ t-m/m

Case (2) For Triangular loading due to earth pressure

a/b	=	0.4083485	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 = 0.4083485 β_1 = 0.242947

β_2 = 0.161873

Earth pressure:

$q = 0.2794 \times 1.8 \times 8.816 = 4.433743$ t/m^2

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

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

$$\sigma_{bmax} = \frac{0.2429474 \times 4.433743 \times 77.72}{0.25} = 334.8774 \text{ t/m}^2$$

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 = 334.8774 x 0.041667 = **13.95323** t-m/m

$\sigma_{amax} = \dfrac{0.161873 \times 4.433743 \times 77.72}{0.25} = 223.1249\ t/m^2$

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 = 223.12485 x 0.041667 = **9.296869** t-m/m

Total Moment in Solid Return /m height = **12.8963** t-m/m
Along X-direction

Total Moment in Solid Return /m width = **17.29263** 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 2.666667 x 0.179776 = 72.44307 t-m

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

A_{st}/m = 688.231 mm^2/m

Provide 12 mm dia @ 150 mm c/c providing 753.9822 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 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 5.60
+ 0.5 x 0.2794 x 1.8 x 0.5625 x 4.00 x 5.60
+ 0.5 x 0.2794 x 1.8 x 0.444444 x X^2 x dx x (7.60 - X)
+ 0.2794 x 1.8 x 1.95 x 0.666667 x X x dx x (7.60 - X)

= 10.13887 + 3.168396 + 0.11176 x 98.13333 + 0.653796 x 39.46667

= **50.07779** t-m

Moment in Solid Return /m height = 12.8963 + $\frac{50.07779}{8.816}$ = 18.57663 t-m/m

Moment in Solid Return /m width = = 17.29263 t-m/m

Design of face B-B'

Moment in Solid Return /m height = **18.57663** 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 x 0.1764 = 26.656 t-m

A_{st} = $\frac{18.57663 \times 10^6}{20400 \times 0.888889 \times 0.42}$ = 2439.159 mm^2

A_{st}/m = 2439.159 mm^2/m

Provide 20 mm dia @ 120 mm c/c providing 2617.994 mm^2 on earth face.
Provide 12 mm dia @ 120 mm c/c providing 942.4778 mm^2 on other face.
Along Horizantal direction.

Design of face A'-B'

Moment in Solid Return /m width = **17.29263** 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 x 0.1936 = 29.25511 t-m

A_{st} = $\frac{17.29263 \times 10^6}{20400 \times 0.888889 \times 0.44}$ = 2167.359 mm^2

A_{st}/m = 2167.359 mm^2/m

Provide 20 mm dia @ 130 mm c/c providing 2416.61 mm^2 on earth face.
Provide 12 mm dia @ 130 mm c/c providing 869.9795 mm^2 on other face.
Along Vertical direction.

1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure	K_a	=	0.279384
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		=	0.5
Avg. cover to reinforcement. (FOR 2-3 LAYERS)		=	0.150 m

H (From formation level) = 8.466 m

DESIGN FOR BENDING MOMENT

S.NO.	UNITS	1
H (FROM TOP)	m	8.466
WIDTH OF RETURN AT THIS LEVEL	m	3.600
FORCE DUE TO EARTH PRESSURE	t	101.610
MOMENT DUE TO EARTH PRESSURE	tm	361.297
FORCE DUE TO L.L. SURCHARGE	t	28.805
MOMENT DUE TO L.L. SURCHARGE	tm	121.932
TOTAL MOMENT	tm	483.229
DESIGN MOMENT	tm	483.229
REQUIRED EFFECTIVE DEPTH	m	2.828
EFF. DEPTH AVAILABLE	m	3.450
AREA OF STEEL REQUIRED	cm^2	77.233
DIAMETER OF BAR PROVIDED	mm	32
TOTAL NO. OF BARS	no.	12
AREA OF STEEL PROVIDED	cm^2	96.51
		O.K.
		25.0
CHECK FOR SHEAR STRESS		
SHEAR FORCE	t	135.415
SHEAR STRESS	t/m^2	78.502
Ast/bd x 100	%	0.448
PERMISSIBLE SHEAR STRESS	MPa	0.29
PERMISSIBLE SHEAR STRESS	t/m^2	29.91
Asv/sv	cm^2/m	6.904

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	=	300	mm				
Assuming a cap thickness of	=	300	mm				
Volume of Abutment cap	=	0.3	x	1.20	x	12	
	=	4.32	m^3				
Quantity of steel	=	1 %	of volume				
	=	$\frac{1}{100}$	x	4.32			
	=	0.0432	m^3				
Quantity of steel to be provided at top	=	0.0216	m^3				
Quantity of steel to be provided at bottom	=	0.0216	m^3				

Top & bottom face:

Quantity of steel to be provided in Longitudinal di	=	0.0108	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

$$\frac{0.0108}{11.9} = 907.563 \text{ mm}^2$$

Provide **9** nos of bars **12** mm dia at top & bottom face.

= 1017.88 mm^2

Transverse steel:

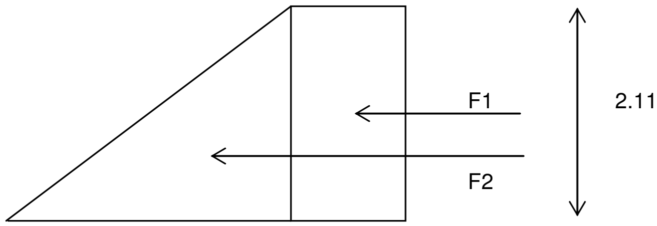
Quantity of steel to be provided in Longitudinal di	=	0.0108	m^3
Assuming a clear cover of	=	50	mm
Assuming a dia of bar	=	12	mm
Length of bar 1.20 - 0.100	=	1.1	m
Volume of each stirrup	=	0.00012	m^3
no of stirrups required for m/length	=	8	nos
Required Spacing	=	$\frac{1000}{8}$	
	=	125	mm

Provide 12 mm dia bar 125 mm c/c stirrups throught in length of abutment cap.

904.779 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	2.11	=	16.94639	t
Intensity for triangular portion	=	0.2794	x	2.00	x	2.11	=	1.176833	t/m^2
F2	=	1.176833	x	12.00	x	2.11	=	14.87046	t
M1	=	16.94639	x	1.05			=	17.84455	t-m
M2	=	14.87046	x	0.88452			=	13.15322	t-m
M1+ M2							=	**30.99777**	t-m
Total moment at base of dirt wall /m length							=	2.583147	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 $= \dfrac{2.583147}{1.51 \times 1000}$ = 130.7935 mm

Ast req $= \dfrac{2.583147}{200 \times 0.889 \times 244}$ = 595.4258 mm^2/m

Minimum steel $= \dfrac{0.12}{100} \times 300 \times 1000$ = 360 mm^2/m

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	2.583147	=	0.774944	t-m/m

Ast req $= \dfrac{0.774944}{200 \times 0.889 \times 238}$ = 183.1309 mm^2/m

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.

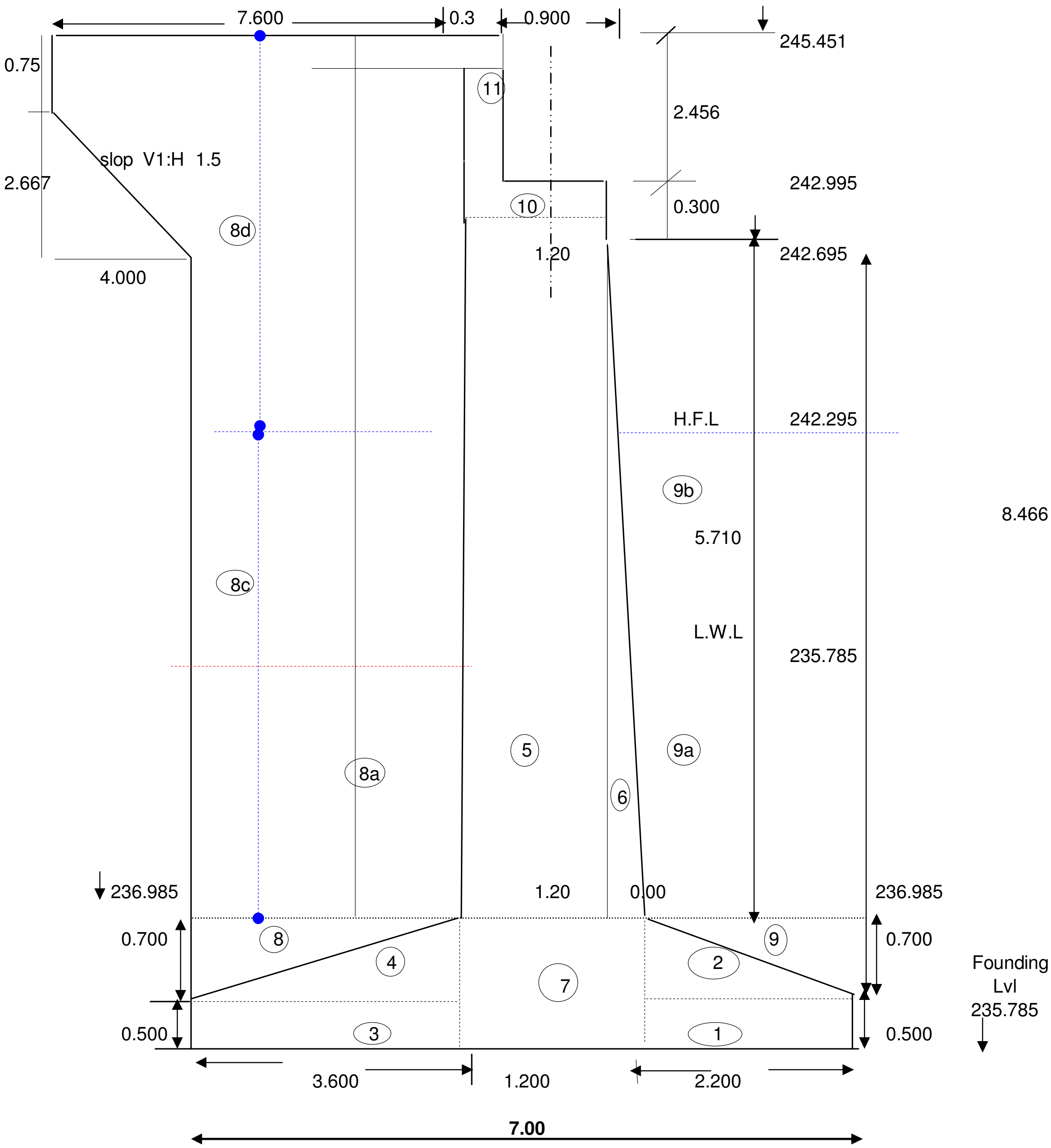
392.6991 mm^2/m

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


7.600
0.3
0.900
245.451
0.75
11
2.456
slop V1:H 1.5
2.667
242.995
10
0.300
8d
1.20
242.695
4.000
H.F.L
242.295
9b
8.466
5.710
8c
L.W.L
235.785
5
9a
8a
6
236.985
1.20
0.00
236.985
0.700
8
9
0.700
4
2
Founding
Lvl
7
235.785
3
1
0.500
0.500
3.600
1.200
2.200
7.00

```
INPUT FILE: 70RW.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.00 0 0;2 0.3 0 0;3 21.3 0 0;4 21.6 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2   3   PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 8.0 2*12 4*17.0  DIS 3.96 1.52 2.13 1.37 3.05 1.37
    20. LOAD GENERATION 175
    21. TYPE 1 -13.4 0. 0. XINC .2
    22. PERFORM ANALYSIS
    23. LOAD LIST 109
    24. PRINT SUPPORT REACTION
        SUPPORT  REACTION

   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


      2  109      0.00     22.97      0.00      0.00      0.00      0.00
      3  109      0.00     77.03      0.00      0.00      0.00      0.00


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


    25. FINISH


             *********** END OF THE STAAD.Pro RUN ***********
```

```
INPUT FILE: CLASS A.STD
     1. STAAD SPACE
     2. INPUT WIDTH 72
     3. UNIT METER MTON
     4. PAGE LENGTH 1000
     5. UNIT METER MTON
     6. JOINT COORDINATES
     7. 1 0.00 0 0;2 0.3 0 0;3 21.3 0 0;4 21.6 0 0
     8. MEMBER INCIDENCES
     9. 1 1 2 3
    10. MEMBER PROPERTY CANADIAN
    11. 1 TO 3 PRI YD 1.0 ZD 1.0
    12. CONSTANT
    13. E CONCRETE ALL
    14. DENSITY CONCRETE ALL
    15. POISSON CONCRETE ALL
    16. SUPPORT
    17. 2   3   PINNED
    18. DEFINE MOVING LOAD
    19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8  DIS 1.1 3.2 1.2 4.3 3 3 3
    20. TYPE 2 LOAD 4*6.8 2*11.4 2*2.7  DIS 3 3 3 4.3 1.2 3.2 1.1
    21. *LOAD GENERATION 202
    22. *TYPE 1 -18.8 0. 0. XINC .2
    23. LOAD GENERATION 202
    24. TYPE 2 -18.8 0. 0. XINC .2
    25. PERFORM ANALYSIS
    26. LOAD LIST 130
    27. PRINT SUPPORT REACTION
        SUPPORT  REACTION

   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
   -----------------
 JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


      2  130      0.00     13.13      0.00      0.00      0.00      0.00
      3  130      0.00     36.87      0.00      0.00      0.00      0.00


      28. FINISH
```

```
Abutment SHAFT ..DRY NORMAL


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

along width-compression face- no of bar:  90   tension face- no of bar: 180
Dia (mm)                                 16                              20
Cover (cm)                              7.50                            10.5
along depth-compression face- no of bar:   8   tension face- no of bar:   8
Dia (mm)                                 16                              16
Cover  (cm)                             7.50                             7.5

Modular Ratio : Compression  =      10.0
Modular Ratio : Tension      =      10.0
Allowable Concrete Stress    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   560.716 T
Mxx                          =  1178.787 Tm
Myy                          =   192.801 Tm


Intercept of Neutral axis : X axis : = 214.704 m
                          : y axis : =    .342 m

          Concrete Stress Governs Design


Stress in Concrete due to Loads =   71.67 Kg/cm^2
Stress in Steel    due to Loads = 1614.66 Kg/cm^2
Percentage of Steel             =     .54 %

Abutment SHAFT ..DRY SPAN DISLODGED

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   225.901 T
Mxx                          =   915.987 Tm
Myy                          =      .000 Tm


Intercept of Neutral axis : X axis : = ******* m
                          : y axis : =    .302 m

             Steel Stress Governs Design

Stress in Concrete due to Loads =   53.55 Kg/cm^2
Stress in Steel    due to Loads = 1404.83 Kg/cm^2
```

```
Percentage of Steel             =     .54 %


Abutment SHAFT ..HFL NORMAL

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   484.252 T
Mxx                          =  1013.075 Tm
Myy                          =   192.801 Tm


Intercept of Neutral axis : X axis : = 185.513 m
                          : y axis : =    .344 m

          Concrete Stress Governs Design



Stress in Concrete due to Loads =   61.88 Kg/cm^2
Stress in Steel    due to Loads = 1388.68 Kg/cm^2
Percentage of Steel             =     .54 %


Abutment SHAFT ..HFL SPAN DISLODGED

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    =     85.00 Kg/cm^2
Allowable Steel Stress       =   2040.00 Kg/cm^2

Axial Load                   =   149.437 T
Mxx                          =   750.275 Tm
Myy                          =      .000 Tm


Intercept of Neutral axis : X axis : = ******* m
                          : y axis : =    .296 m

             Steel Stress Governs Design



Stress in Concrete due to Loads =   43.71 Kg/cm^2
Stress in Steel    due to Loads = 1179.01 Kg/cm^2
Percentage of Steel             =     .54 %
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DESIGN OF SUPERSTRUCTURE

For Design of Superstructure of RCC Girder 21.0 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.