1-2. DESIGN OF APPROACH BRIDGE

(1) SUPERSTRUCTURE

CONSTRUCTION OF THE BRIDGE OVER THE RIVER RUPSA (PHASE-2)
(DESIGN CALCULATION OF SUPERSTRUCTURE OF APPROACH BRIDGE)

DESIGNED BY : PRASANTA KUMAR BHOWMIK DATE : 2/14/00

GENERAL INPUT

Total Length of one Girder		29950 mm,
Distance of center of Bearing from end of Girder		450 mm,
Live Load	HS 20	
Total no. of Lane		4 Lane,
Aggregate Size of Chips		19.05 mm,
Mention Deck type to be used		2
Total no. of girder below deck		7
Put 1.0 for working stress design or 2.0 for load factor design.		2
Put 1.0 for non-composite or 2.0 for composite wearing course		2
Percentage of Jack loss (assumed)		3 %
Total load due temp. (-ve for compn, +ve for tension)		200 Kn,
Total load due to utility pipe (equally distributed in all girder)		4.0 Kn/m,
Unit Wt. of concrete for prestressed member		23.563 kN/m^3
Unit Wt. of concrete for non prestressed member		23.563 kN/m^3
Unit Wt. of Wearing Course		23.563 kN/m^3

STRESS INPUT

Put 1.0 for Post-Tensioned or 2.0 for Pre-Tensioned		1.0
28 days Concrete Strength for :		
PC Girder		35 N/mm^2
Deck Slab, Sidewalk, Railing		30 N/mm^2
At time of Transfer for :		
PC Girder		30 N/mm^2
Ultimate strength of Prestressing Tendon	fpu	1860 N/mm^2
Put 1.0 for low relaxation wire or strand, 2.0 for stress relieved wire or strand, 3.0 for smooth high strength bar or 4.0 for deformed high strength bar		2.0
Put 1.0 for wire, 2.0 for strand, 3.0 for bar		2.0
Modulus of Elasticity of Prestressing Reinforcement		193053 N/mm^2
Yield strength of Non-Prestressing Rebar		414 N/mm^2
Put 1 for plain bar or 2 for Deformed bar		2
Quantity limiting distribution of flexural reinforcemrnt, z		22766.48 kN/m

CLEAR COVERS

arapet	40 mm,	Curb	50 mm,
Pedestal of Parapet	50 mm,	Top of Deck Slab	50 mm,
Top of Sidewalk	50 mm,	Bottom of Deck Slab	40 mm,
Bottom of Sidewalk	50 mm,	Ducts	50 mm,

INPUT RELATING LOSSES

Wobble coefficient, K	0.000656 per m,
Curvature co-efficient, μ	0.25
Relative Humidity	70 %
Amount of slip	8 mm,

REDUCTION FACTORS

Phi for Moment	0.95
Phi for Shear	0.9

ALLOWABLE STRESSES

For normal R.C.C. Member

Concrete in Compression			12.000 N/mm^2
Modulus of Rupture			3.411 N/mm^2
Plain Concrete in Tension			0.716 N/mm^2
Concrete in Shear			0.432 N/mm^2
Tension in Reinforcement			165.600 N/mm^2
n	8	k	0.367
J	0.878	R	1.932 N/mm^2
Modulus of Elasticity			27572 N/mm^2
Beta1			0.8324

For Prestressed Member

Modulus of Elasticity	29781 N/mm^2
Modulus of Elasticity at time of transfer	27572 N/mm^2
Basic Creep-coefficient	2.65

A. Concrete

Stresses immediately after transfer	
i) Compression	-16.50 N/mm^2
ii) Tension with no bonded Reinforcement	1.36 N/mm^2
Tension with bonded Reinforcement	3.41 N/mm^2
Stresses at service load after losses have occured	
i) Compression	
x) under all loads except (y) & (z)	-21.00 N/mm^2
y) under Prestressed force + all permanent dead loads	-14.00 N/mm^2
z) under Live loads + 1/2 of (y)	-14.00 N/mm^2
ii) Tension	
x) with bonded Reinforcement	2.95 N/mm^2
with bonded reinforcement at severe condition	1.47 N/mm^2
y) without bonded reinforcement	0.00 N/mm^2
Modulus of Rupture	3.68 N/mm^2

MULTIPLYING FACTOR FOR DESIGN LOAD/STRESS FOR DESIGN IN LOAD FACTOR METHOD

General		1.3
β_d		1
Dead load		1
Live load,		
• for exterior beam supports S.W. L.L., traffic L.L. + Impact		1.25
• Live load + Impact, Normal condition		1.67
• Live load + Impact, Over load criterion		2.2
• Live load 1' from face of rail		1
ϕ value	For moment	0.9
	For shear	0.85

B. Prestressing Reinforcement

Yield Strength	1581 N/mm^2
<u>Stress during tensioning</u>	1423 N/mm^7
<u>Stress immediately after seating</u>	
At anchorage	1302 N/mm^2
At the end of seating loss zone	1312 N/mm^3
<u>Stress at service load after all losses</u>	1265 N/mm^5
Y_p	0.4

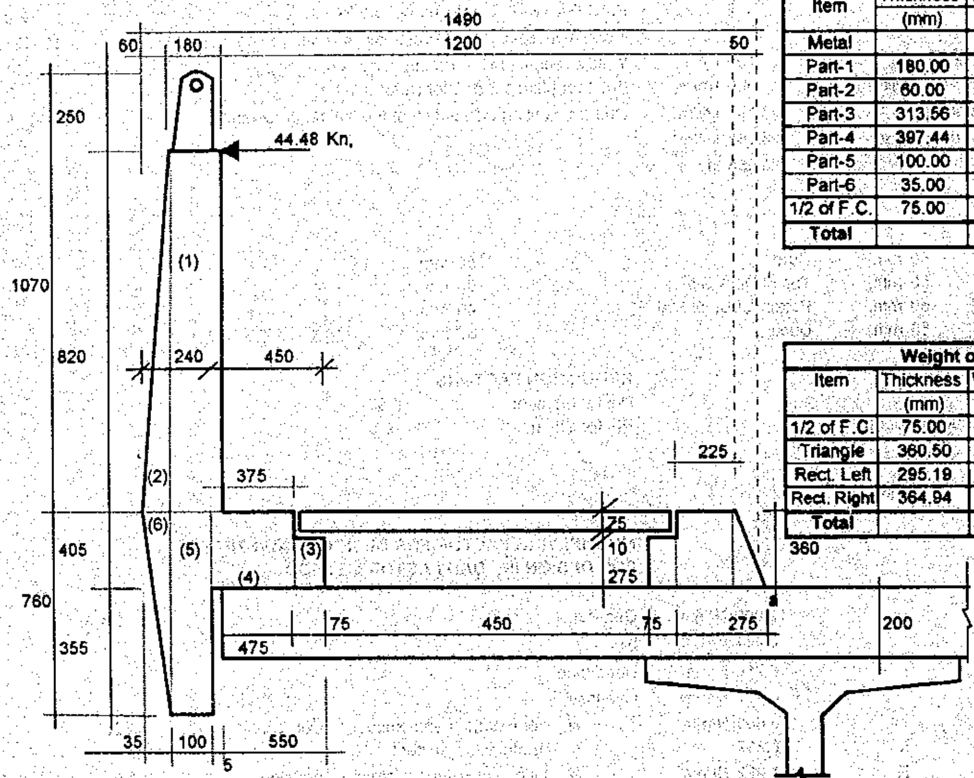
PARAPET

LEFT / SYMMETRIC PARAPET

Put 1.0 for traffic, 2.0 for combination & 3.0 for Pedestrian Railing	=	2
Width of each pre-cast segment of parapet	=	1500 mm,
Maximum c/c spacing of metalic post	=	1500 mm,
Total no. of post required	=	21 Nos.,

Unit wt. Of metalic part of post & rail	77 kN/m^3,
Bottom dimension of top post	125 mm,
Top dimension top post	100 mm,
Thickness of top post	8 mm,
Weight of metalic post per linear meter	0.012 kN/m,
Dia of metalic rail	100 mm,
Thickness rail pipe	6 mm,
Weight of rail per linear meter	0.07 kN/m,

P	44.4822 kN,
h	820 mm,
C	1



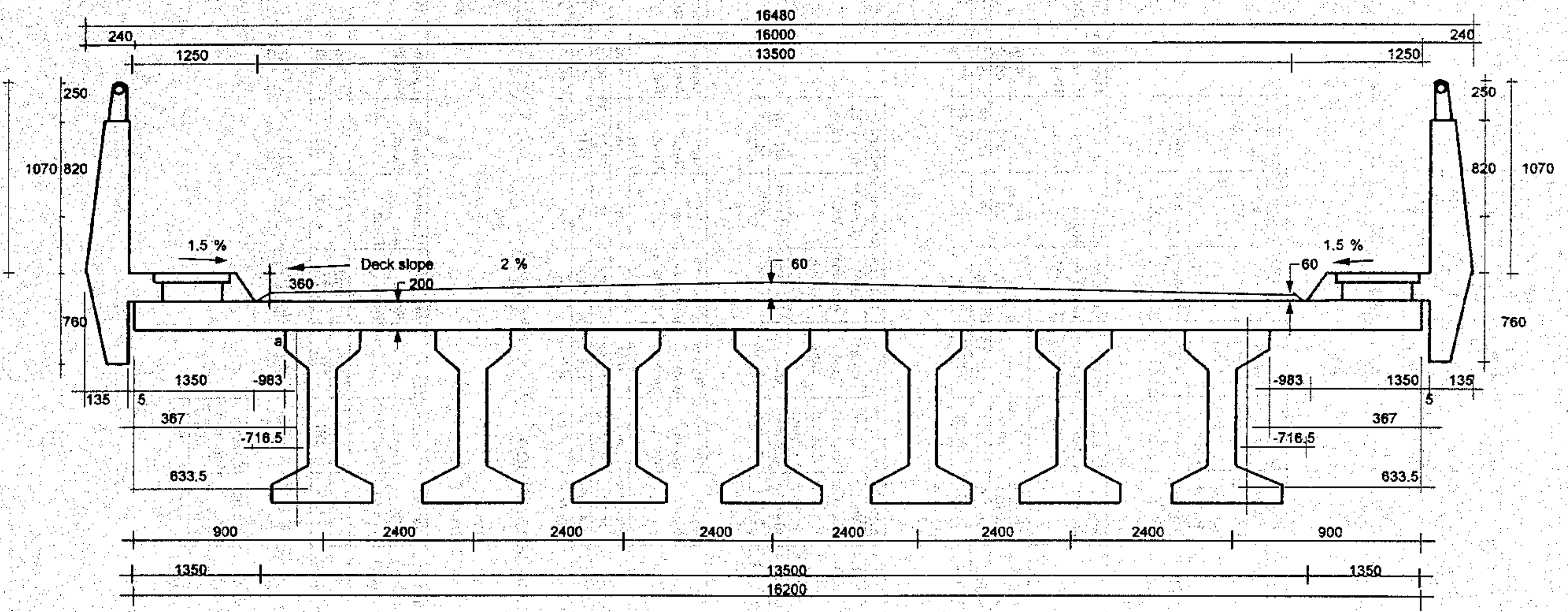
Weight of End Parapet with 1/2 F.C.			
Item	Thickness (mm)	Weight (kN/m)	Dist. of cg from a (mm)
Metal		0.083	1340
Part-1	180.00	3.478	1340
Part-2	60.00	0.580	1450
Part-3	313.56	0.554	837.5
Part-4	397.44	4.495	1115
Part-5	100.00	1.791	1405
Part-6	35.00	0.313	1467
1/2 of F.C.	75.00	0.530	838
Total		11.824	1226.972

Weight of Curb with 1/2 F.C.			
Item	Thickness (mm)	Weight (kN/m)	Dist. of cg from a (mm)
1/2 of F.C.	75.00	0.530	313
Triangle	360.50	0.212	33
Rect. Left	295.19	0.522	313
Rect. Right	364.94	1.935	163
Total		3.199	203.25

DESIGN OF DECK SLAB (For Approach Bridge)

GENERAL

Total No of girder below Deck	7 Nos.	Date :	2/14/00
Strength of Railing/Parapet against Traffic Load	100 %		
Load due to false work	1.0 kN/m2		
Put 1.0 for both side having symmetric parapet & 2.0 for different type	1.0		



LEFT / SYMMETRIC PARAPET

Put 1.0 for traffic, 2.0 for combination & 3.0 for Pedestrian Railing	=	2
Width of each pre-cast segment of parapet	=	1500 mm,
Maximum c/c spacing of metalic post	=	1500 mm,
Total no. of post required	=	21 Nos.,

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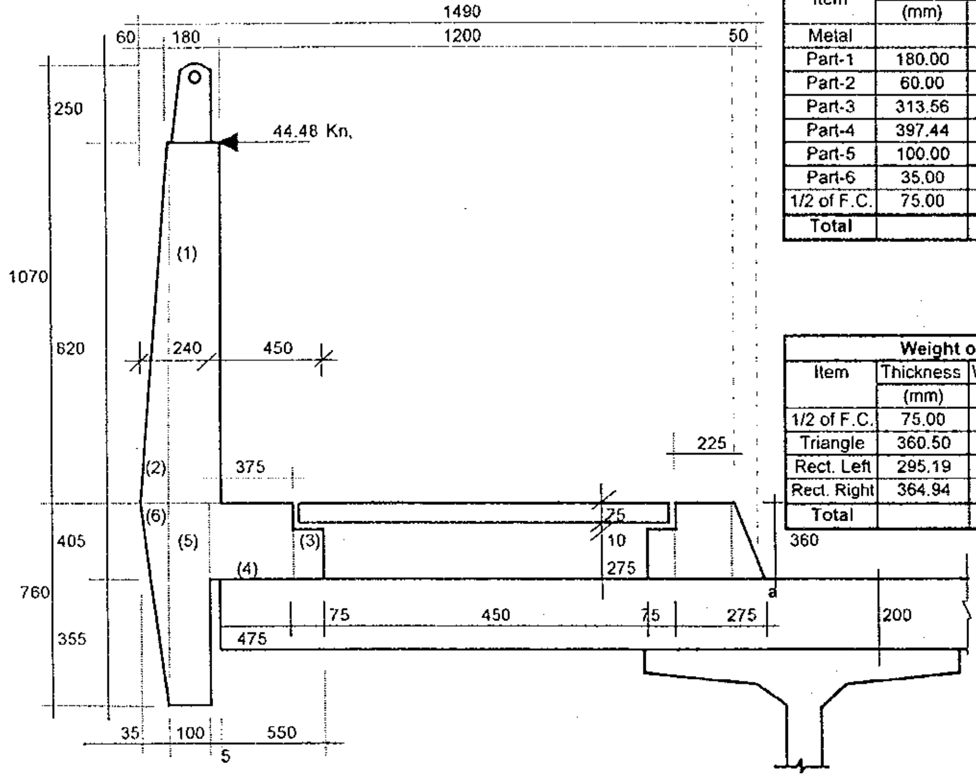
Bottom dimension of top post	125 mm,
Top dimension top post	100 mm,
Thickness of top post	8 mm,
Weight of metalic post per linear meter	0.012 kN/m,
Dia of metalic rail	100 mm,
Thickness rail pipe	6 mm,
Weight of rail per linear meter	0.07 kN/m,

P	44.4822 kN,
h	820 mm,
C	1



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Part-5	100.00	1.791	1405
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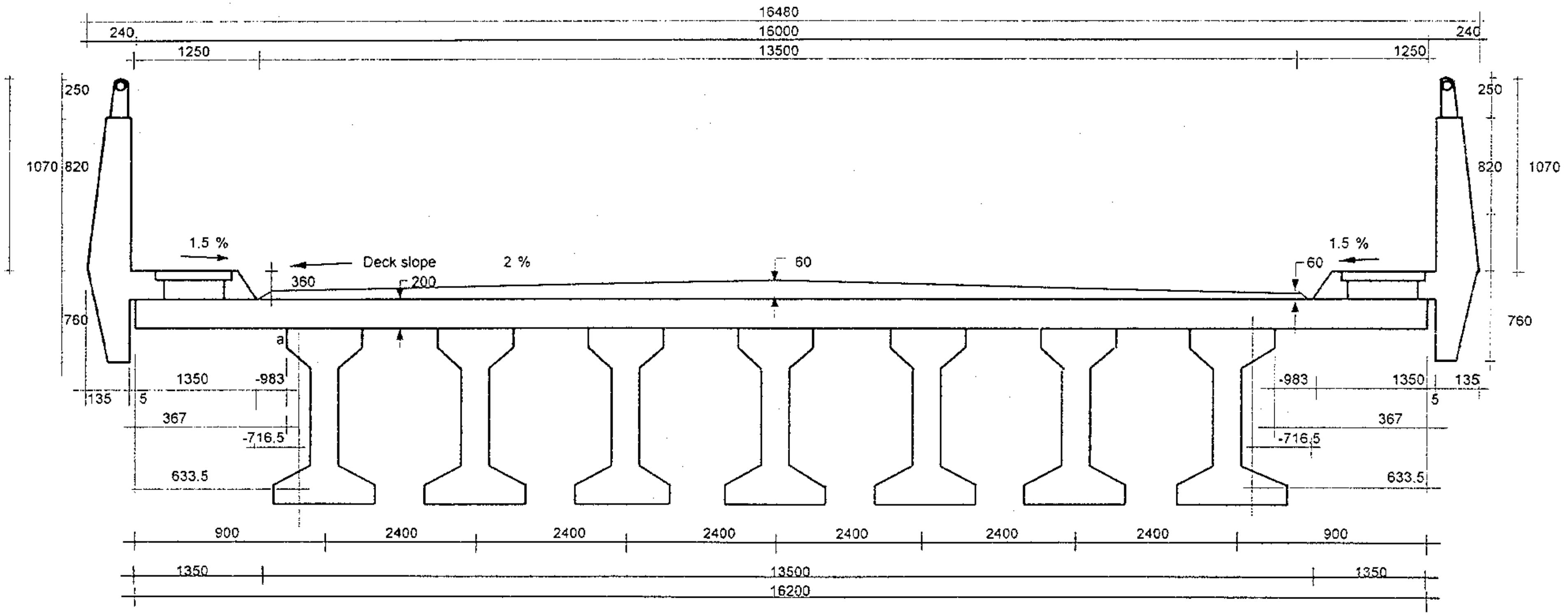


DEAD LOAD MOMENT AT SPAN END OF CANTILEVERE SLAB

FROM LEFT PARAPET SYSTEM / FROM RIGHT PARAPET SYSTEM

Components	Weight	Dist. from span end	Mom. at Span end	Components	Weight	Dist. from span end	Mom. at Span end
	kN/m	(mm)	kN-m/m		kN/m	(mm)	kN-m/m
Parapet	11.824	510.5	6.036	Parapet	11.824	510.5	6.036
Curb	3.199	-513.3	0.000	Curb	3.199	-513.3	0.000
C. Slab	2.985	316.8	0.946	C. Slab	2.985	316.8	0.946
Wearing Course 1	0.000	-358.3	0.000	Wearing Course 1	0.000	-358.3	0.000
Total	18.008		6.981	Total	18.008		6.981

DESIGN OF CANTILEVER SLAB

LEFT CANTILEVER SLAB

Moment Table

Effective Width for			Moment due to					Factored Moment Combination		
Post Load	Wheel Load over Deck Slab	Wheel Load over Sidewalk	Dead Load	Post Load	Wheel load over Deck Slab + Impact	Wheel load over Sidewalk + Impact	Wheel load at Curb	D+P (working combin-ation)	(D+WD+ WC)	((D+WS)/ 1.5)
m	m	m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m
2.0468	0.000	1.3260	6.981	27.818	0.000	15.958	0.000	34.799	9.076	29.821

Design Table

Item with design procedure	Design Moment	Effective depth available	Effective depth required	Comment regarding Thickness	Dia. of Rebar	Area of Rebar Reqd.	Minimum Area Reqd.	Spacing Reqd.	Spacing Provided	Z value for flex. cracking
	kN-m/m	mm	mm		mm	mm2	mm2	mm	mm	Mpa
Other combination, designed by method directed	29.821	140.00	66	OK	20	592.5	466.31	530.19		13874
Due to Post Load, designed by working stress method.	34.799	140.00	134.19	OK	20	1710.19	466.31	183.70	150	
Due to Post Load, designed by ultimate stress method.	69.468	140.00	100.49	OK	20	1457.588	466.31	215.53		O.K.

RIGHT CANTILEVER SLAB

Moment Table

Effective Width for			Moment due to					Factored Moment Combination		
Post Load	Wheel Load over Deck Slab	Wheel Load over Sidewalk	Dead Load	Post Load	Wheel load over Deck Slab + Impact	Wheel load over Sidewalk + Impact	Wheel load at Curb	D+P (working combin-ation)	(D+WD+ WC)	((D+WS)/ 1.5)
m	m	m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m
0.0020	0.000	1.326	6.981	27.818	0.000	15.958	0.000	34.799	9.076	29.821

Design Table

Item with design procedure	Design Moment	Effective depth available	Effective depth required	Comment regarding Thickness	Dia. of Rebar	Area of Rebar Reqd.	Minimum Area Reqd.	Spacing Reqd.	Spacing Provided	Z value for flex. cracking
	kN-m/m	mm	mm		mm	mm2	mm2	mm	mm	Mpa
Other combination, designed by method directed	29.821	140.00	65.84	OK	20	592.5442	466.31	530.19		13874
Due to Post Load, designed by working stress method.	34.799	140.00	134.19	OK	20	1710.19	466.31	183.70	150	
'e to Post Load, designed by ultimate stress method.	69.468	140.00	100.49	OK	20	1457.588	466.31	215.53		O.K.

DESIGN OF INTERIOR SLAB BY COEFFICIENT METHOD

Dead Load from Wearing Course 1.414 kN/m^2

Dead Load from self wt. of Slab 4.713 kN/m^2

Span of Slab	Moment due to Dead Load	Moment due to Live Load with Impact	Design Moment	Effective depth available	Effective depth required	Comment regarding Thickness	Dia. of Rebar	Area of Rebar Reqd.	Minimum Area Reqd.	Spacing Reqd.	Spacing Provided	Working steel stress	Z value for flex. craqcking	Comment regarding crack
m	kN-m/m	kN-m/m	kN-m/m	mm	mm		mm	mm2	mm2	mm	mm	Mpa	kN/m	
1.867	2.135	18.794	43.579	142.00	79.59	OK	16	867.7	473.0	231.7	150	125	12567	O.K.

DISTRIBUTION REINFORCEMENT

% of +ve reinforcement	Area of Rebar Required	Dia. of Rebar	Spacing Reqd.	Spacing Provided
	mm2	mm	mm	mm
67	581	12	195	150

DATA FOR PRESTRESSED GIRDER

Effective web width for Exterior Girder :

(1)	2841.0
(2)	1066.0
Final	1066.0

Effective width of flange for Exterior Girder :

(1)	3320.8
(2)	2633.0
(3)	2100.0
Final	2100.0

Fraction of Wheel Load on left exterior Girder	0.5585
Fraction of Wheel Load on right exterior Girder	0.5585
Fraction of Wheel load on left exterior Girder when Wheel over Side	1.650667
Fraction of Wheel load on right exterior Girder when Wheel over Sid	1.650667
Fraction of Wheel Load on interior Girder	1.431639

Effective web width for Interior Girder :

(1)	2841
(2)	1066
Final	1066.0

Effective width of flange for Interior Girder :

(1)	7262.5
(2)	3466
(3)	2400
Final	2400

LOAD FROM ADJUSTING CONCRETE ON GIRDER DUE TO SLAB INCLINATION

Extension of slab below due to slope of deck slab	Girder No.						
	1	2	3	4	5	6	7
	Left Exterior	1st Interior	2nd Interior	3rd Interior	4th Interior	4th Interior	Right Exterior
Rise for each girder / flange		48	21.32				
Increase of weight due to girder spacing		1.206					
Increase due to flange width		0.268					
Weight	0.268	**0.268**	**0.268**	**0.268**	**0.268**	**0.268**	0.268

SUMMERY OF LOAD ON GIRDERS

Load Type	Girder No.						
	1	2	3	4	5	6	7
	Left Exterior	1st Interior	2nd Interior	3rd Interior	4th Interior	5th Interior	Right Exterior
Non-composite dead load	9.977	11.816	11.511	11.612	11.511	11.816	9.977
Composite dead load	20.239	-1.894	4.896	2.649	4.896	-1.894	20.239

SUMMERY OF LOADS FROM DECK SYSTEM ON GIRDERS

Press 1.0 for distribution of load from deck system using moment distribution method or 2.0 for AASHTO method	**2**		
Mention Girder you Like to design now	**2**		

INTERIOR GIRDER

Non-composite dead load on girder	11.31031	kN/m,	
Composite dead load on girder + utility pipe	9.636861	kN/m,	
Effective web width	1066	mm,	
Effective flange width	2400	mm,	
Fraction of wheel load on girder	1.431639		
Fraction of wheel load on girder, when wheel over sidewalk	0		
Effective width for sidewalk	0	mm,	
Difference in elev. Between girder top and slab bottom	**11**	mm,	10.66 ← **Concentrate**
Uniform load on girder from false work	2.4	kN/m,	

EXTERIOR GIRDER

Non-composite dead load on girder	9.89652	kN/m,
Composite dead load on girder + utility pipe	9.636861	kN/m,
Effective web width	1066	mm,
Effective flange width	2100	mm,
Fraction of wheel load on girder	0.5585	
Fraction of wheel load on girder, when wheel over sidewalk	1.650667	
Effective width for sidewalk	1200	mm,
Difference in elev. Between girder top and slab bottom	11	mm,
Uniform load on girder from false work	2.1	kN/m,

DESIGN OF INTERIOR PRESTRESSED GIRDER
(Dead load from curb/rail/pedestal equally distributed on girder)

DESIGNED BY : PRASANTA KUMAR BHOWMIK

GENERAL

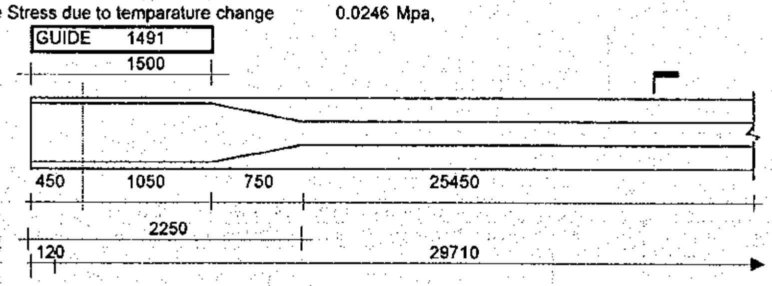
Total Length of one Girder	29950 mm,
Distance of center of Bearing from end of Girder	450 mm,
Span of Girder	29050 mm,
Distance of center of Diaphragm from end of Girder	350 mm,
C/c. spacing between girders	2400 mm,
Effective width of sidewalk	0 mm,
Fraction of wheel load on Girder	1.432
Fraction of wheel load on girder when wheel over sidewalk	0.000
Live Load Magnitude	HS 20
Put 1.0 or 2.0 for Deck type	
Mention AASHTO or WASHINGTON Pattern	AASHTO
Mention Type	6
Mention end section 1 or 2?	2
Anchorage indent	120 mm,
Tensile Stress due to temparature change	0.0246 Mpa,

DIAPHRAGM DETAILS

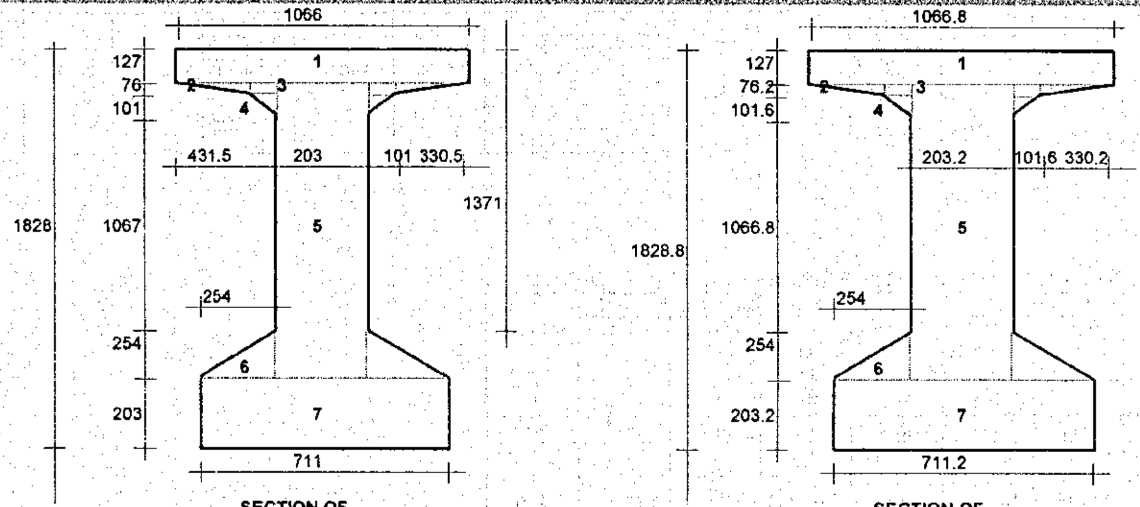
Total no. of interior diaphragm reqd	2
Total no. of interior diaphragm provided	1
Width of diaphragm	300 mm,
Depth of end diaphragm	1867 mm,
Depth of interior diaphragm	1625 mm,
Wt. of exterior diaphragm	26.78 kN,
Wt. of interior diaphragm	23.65 kN,

NOTE: In designing this girder following references are followed :
1. AASHTO - 16th edition, 1996.
2. Design of Prestressed Concrete, 2nd edition, by : Arthur H. Nilson.
3. Design of Prestressed Concrete Structure, 3rd edition, by : T. Y. Lin & Ned H. Burns.

All notations are followed from reference-2



GIRDER SECTION

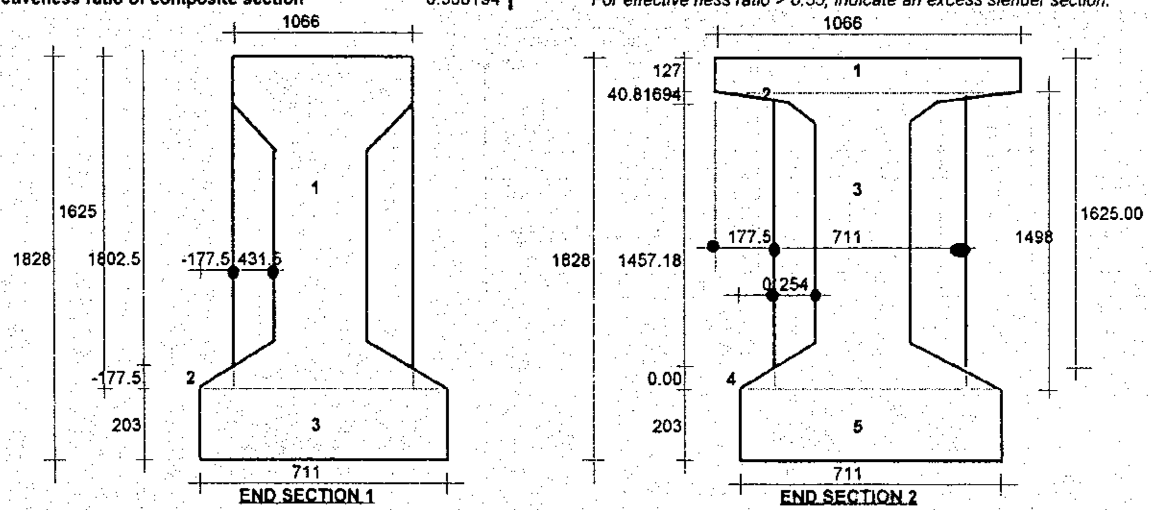


SECTION OF GIRDER IN USE

SECTION OF STANDARD GIRDER

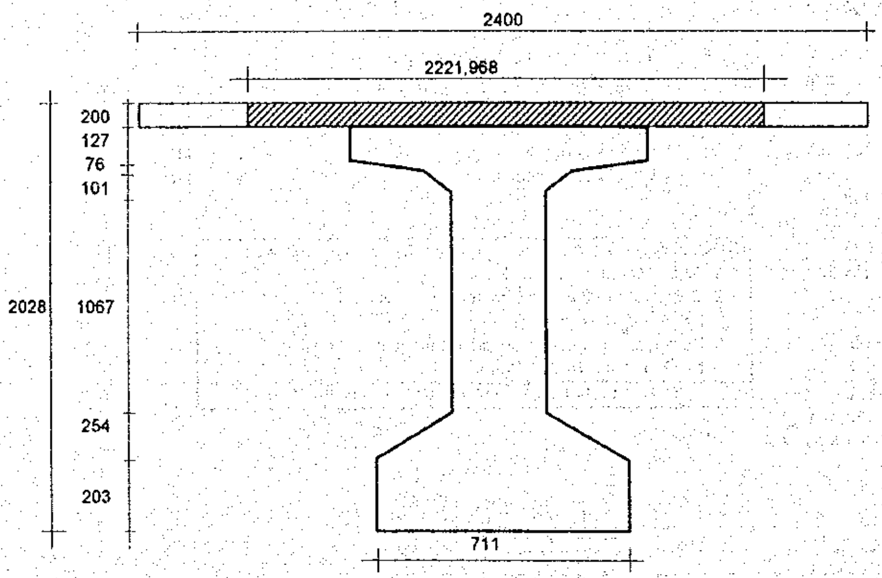
Effectiveness ratio of non-composite section	0.521613
Effectiveness ratio of composite section	0.538194

Note : *For effectiveness ratio < 0.45, indicate too heavy a section*
For effective ness ratio > 0.55, indicate an excess slender section.



END SECTION 1

END SECTION 2



COMPOSITE SECTION

PROPERTIES OF PRECAST MIDDLE SECTION

Segment	Area	Distance of CG from Top C_{1P}	Distance of CG from Bottom C_{2P}	Moment of Inertia of Components I_O	Moment of Inertia of Block Ic	Radius of Gyration r^2
	mm^2	mm	mm	mm^4	mm^4	mm^2
1	135382			1.82E+08		
2	12559			4030044		
3	7676			3694715		
4	5100.5	904.62	923.38	2890567	3.05E+11	435707.6
5	304094			5.69E+10		
6	32258			1.16E+08		
7	144333			4.96E+08		
Total	698996					

Weight per linear metre = 16.47 kN/m.

PROPERTIES OF PRECAST END SECTION

Segment	Area	Distance of CG from Top C1P	Distance of CG from Bottom C2P	Moment of Inertia of Components IO	Moment of Inertia of Block Ic	Radius of Gyration r2
	mm2	mm	mm	mm4	mm4	mm2
1	135382			1.82E+08		
2	3623			335287		
3	1065078	881.49	946.51	1.99E+11	3.98E+11	294002
4	0			0		
5	144333			4.96E+08		
Total	1352038					

Weight per linear metre = 31.86 kN/m.

PROPERTIES OF EQUIVALENT COMPOSITE SECTION AT MIDDLE

Segment	Area	Distance of CG of Composite section from Top of precast section C_{1C}	Bot. of precast section C_{2C}	Bottom of Slab C_{4C}	Top of Slab C_{3C}	Moment of Inertia of Components I_O	Moment of Inertia of Block Ic	Radius of Gyration r^2
	mm^2	mm	mm	mm	mm	mm^4	mm^4	mm^2
Precast Section	698996					3.05E+11		
Eqv. Slab	444393.6	514.1593	1313.841	525.1593	725.1593	1.48E+09	5.86E+11	512761.3
Total	1143390							

Weight per linear ft. = 26.94 kN/m.

PROPERTIES OF EQUIVALENT COMPOSITE SECTION AT END

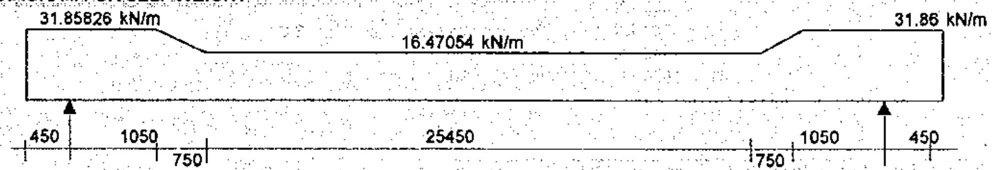
Segment	Area	Distance of CG of Composite section from Top of precast section C_{1C}	Bot. of precast section C_{2C}	Bottom of Slab C_{4C}	Top of Slab C_{3C}	Moment of Inertia of Components I_O	Moment of Inertia of Block Ic	Radius of Gyration r^2
	mm^2	mm	mm	mm	mm	mm^4	mm^4	mm^2
Precast Section	1352038					3.98E+11		
Eqv. Slab	444393.6	638.6969	1189.303	649.6969	849.6969	1.48E+09	7.28E+11	405502.1
Total	1796432							

Weight per linear ft. = 42.33 kN/m.

SUMMERY OF SECTION PROPERTIES AT DIFFERENT LOCATIONS

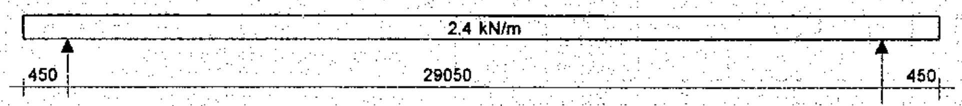
Section			1	2	3	4	5	6	7	8	9
Distance from end		(mm)	0	450.00	2250	7712.5	14975	22237.5	27700	29500	29950
Total area of duct		(mm)	13273.23	13273.23	13273.23	13273.23	13273.23	13273.23	13273.229	13273.23	13273.23
Dist.of c.g. of duct from bot.		(mm)	895.27	849.24	679.17	300.61	117.73	300.61	679.17	849.24	895.27
Duct deducted properties of noncomposite section	Acp	(mm)	1338765	1338765	685723	685723	685723	685723	685723	1338765	1338765
	C_{1P}	(mm)	880.99	880.53	899.89	892.56	889.02	892.56	899.89	880.53	880.99
	C_{2P}	(mm)	947.01	947.47	928.11	935.44	938.98	935.44	928.11	947.47	947.01
	Ic	(mm^4)	3.97E+11	3.97E+11	3.04E+11	2.99E+11	2.96E+11	2.99E+11	3.04E+11	3.97E+11	3.97E+11
	r^2	(mm^2)	296888	296819	442937	436335	431081	436335	442937	296819	296888
Duct grouted properties of noncomposite section	Acp	(mm^2)	1352038	1352038	698996	698996	698996	698996	698996	1352038	1352038
	C_{1P}	(mm)	881.49	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881.49
	C_{2P}	(mm)	946.51	946.51	923.38	923.38	923.38	923.38	923.38	946.51	946.51
	Ic	(mm^4)	3.98E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+11
	r^2	(mm^2)	294002	294002	435707.6	435707.6	435707.6	435707.6	435707.6	294002.2	294002.2
Duct grouted properties of composite section	Acc	(mm^2)	1796432	1796432	1143390	1143390	1143390	1143390	1143390	1796432	1796432
	C_{1c}	(mm)	638.70	638.70	514.16	514.16	514.16	514.2	514.2	638.7	638.7
	C_{2c}	(mm)	1189.30	1189.30	1313.84	1313.84	1313.84	1313.8	1313.8	1189.3	1189.3
	C_{4c}	(mm)	649.70	649.70	525.16	525.16	525.16	525.2	525.2	649.7	649.7
	C_{3c}	(mm)	849.70	850	725.2	725.2	725.2	725.2	725.2	849.7	849.7
	Ic	(mm^4)	7.28E+11	7.28E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	7.28E+11	7.28E+11
	r^2	(mm^2)	405502.1	405502.1	512761.3	512761.3	512761.3	512761.3	512761.3	405502.1	405502.1

LOAD DIAGRAM FOR SELF WEIGHT



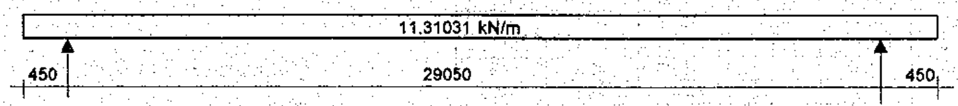
Reaction 275.4983 kN,

LOAD DIAGRAM FOR FALSE-WORK DEAD LOAD



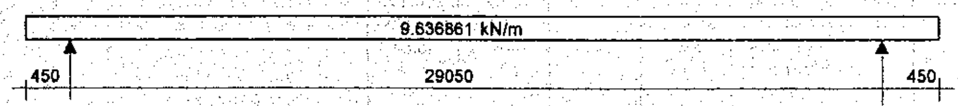
Reaction 35.94 kN,

LOAD DIAGRAM FOR NON-COMPOSITE DEAD LOAD



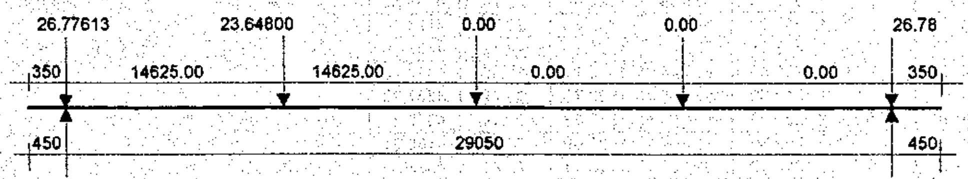
Reaction 169.3719 kN,

LOAD DIAGRAM FOR COMPOSITE DEAD LOAD



Reaction 144.312 kN,

LOAD DIAGRAM FOR CONCENTRATED LOAD FROM DIAPHRAGM



Reaction 38.60013 kN,

ANALYSIS MOR MOMENT

Section	Distance from end	Moment due to: Self wt M_O (a)	Cross Girder (Non-Composite dead load (b)	Non Composite dead load (c)	Composite dead load M_{DC} (d)	Live Load with Impact (e)	Live Load with Impact when wheel on Sidewalk (f)	Sidewalk Live Load (g)	Total (Non-Composite dead load) M_{DP} (b+c)	Falsework (Non-Composite dead load	Total Dead load Moment	Total Factored Moment
	(mm)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	450	-3.23	-2.68	-1.15	-0.98	0.00	0.00	0.00	-3.82	-0.24	-8.02	-10.43
2a	1364	222.17	8.13	144.28	122.94	223.80	0.00	0.00	152.41	30.62	497.52	1132.64
3	2250	416.70	18.61	276.24	235.37	425.29	0.00	0.00	294.85	58.62	946.91	2154.30
4	7712.5	1315.84	83.19	893.68	761.45	1331.87	0.00	0.00	976.87	189.64	3054.17	6861.91
5	14975	1750.20	169.07	1191.95	1015.60	1709.16	0.00	0.00	1361.02	252.93	4126.82	9075.44
6	22237.5	1315.84	83.19	893.68	761.45	1331.87	0.00	0.00	976.87	189.64	3054.17	6861.91
7	27700	416.70	18.61	276.24	235.37	425.29	0.00	0.00	294.85	58.62	946.91	2154.30
8	29500	-3.23	-2.68	-1.15	-0.98	0.00	0.00	0.00	-3.82	-0.24	-8.02	-10.43
9	29950	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

LOSSES

r_{ps} 144.20

rictional loss per unit length 14.20246 kN/m,

For 8mm Anchorage draw in, x(dist. Of anchorage loss zone) 21.7313 mm,

ΔP_A 617.2771 kN,

Prestress force after friction losses (before elastic shortening) :-

- Tensioning end 5333 kN,
- Seating loss zone 5641 kN, No. of cable reqd 3.64
- Midspan 5545 kN,
- Dead end 5525 kN,

SUMMERY OF INITIAL LOSSES

Section	Distance from end	Jacking Force	Losses due to friction and anchorage pull-in	Prestress Force after friction loss (before elastic shortening)	Conc. Stress at the level of steel centroid at sec.of maxm. mom. (f_{cir})	Loss due to elastic shortening	Final initial loss (after immediate losses)	Final initial prestress force (after immediate losses) (Pi)
	(mm)	(kN)	(kN)	(kN)	(mPa)	(kN)	(kN)	(kN)
1	0		617	5333	3.935	59.82	677	5273
2	450		611	5339	4.030	61.27	672	5278
2a	1364		598	5352	4.269	64.90	663	5287
3	2250		585	5365	8.189	124.50	710	5240
4	7712.5	5950	508	5442	11.551	175.61	683	5267
5	14975		405	5545	14.345	218.10	623	5327
6	22237.5		316	5634	12.049	183.19	499	5451
7	27700		393	5557	8.493	129.13	523	5427
8	29500		419	5531	4.175	63.47	482	5468
9	29950		425	5525	4.076	61.98	487	5463

SUMMERY OF TIME-DEPENDENT & FINAL LOSSES

Section	Distance from end	Conc. Stress at the level of steel cg. Due to dead loads except self wt. (f_{cds})	Loss due to creep	Loss due to shrinkage	Loss due to relaxation of pre-stressing steel	Final Time dependent loss	Total loss	Loss in percent	Final effective pre-stress force (Pe)
	(mm)	(mPa)	(kN)	(kN)	(kN)	(kN)	(kN)		(kN)
1	0	0.000	205.06		317.59	678.34	1355.44	22.78	4594.56
2	450	-0.002	210.09		317.92	683.71	1355.86	22.79	4594.14
2a	1364	0.162	217.52		318.88	692.10	1354.90	22.77	4595.10
3	2250	0.574	409.30		260.46	825.45	1535.27	25.80	4414.73
4	7712.5	3.004	510.63		243.02	909.35	1592.70	26.77	4357.30
5	14975	4.849	600.18	155.70	239.06	994.94	1617.63	27.19	4332.37
6	22237.5	3.004	536.61		292.36	984.67	1483.69	24.94	4466.31
7	27700	0.574	425.17		313.00	893.87	1416.41	23.81	4533.59
8	29500	-0.002	217.63		373.10	746.44	1228.88	20.65	4721.12
9	29950	0.000	212.44		372.82	740.96	1228.30	20.64	4721.70

EFFECT OF DIFFERENTIAL SHRINKAGE

Ultimate shrinkage co-efficient, ε_{SHU}	0.0008	
Correction by humidity, (RH = 70) , $F_{SH,H}$	0.7	
Assumed day of slab casting after girder concrete placed, t	60	days
Shrinkage of girder at t days, $\varepsilon_{SH,t}$	0.632	
Differential (remaining) shrinkage, $\varepsilon_{SH,D}$	0.000206	
Ultimate creep coefficient for girder, φ_{cgu}	2.6	
Ultimate creep coefficient for slab, φ_{csu}	2.8	
Correction by humidity for creep coefficient, (RH = 70), $F_{C,H}$	0.801	
Remaining creep of girder after t days, φ_{cg}	0.961267	
Remaining creep of slab, φ_{cs}	2.2428	

Horizontal Shear & Bending Moment on contact surface of girder & slab due to Differential Shrinkage
At all section like mid section

m	B	C	F	φ_{cg}	φ_{cs}	Girder		Slab	
						V_{SH}	M_{SH}	V_{SH}	M_{SH}
		(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
222.073	223.073	-23.746	4.730	0.961	2.243	433.145	46.107	261.969	27.886

	Girder		Slab	
	Bottom	Top	Bottom	Top
Stress at concrete due to differential shrinkage	0.708	-1.920	0.629	0.471

Horizontal Shear & Bending Moment on contact surface of girder & slab due to Differential Shrinkage
At all section like end section

m	B	C	F	φ_{cg}	φ_{cs}	Girder		Slab	
						V_{SH}	M_{SH}	V_{SH}	M_{SH}
		(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
289.845	290.845	-31.402	5.608	0.961	2.243	561.488	60.623	339.592	36.665

	Girder		Slab	
	Bottom	Top	Bottom	Top
Stress at concrete due to differential shrinkage	0.908	-1.647	0.779	0.643

DESIGN FOR MOMENT AT MID SECTION

Total Jacking tension canbe taken by supplied cable (after jack loss) 5993.99 kN.

Initial Tension				Total no. of Cable reqd.	Jacking Tension (after Jack loss)	Effective Tension at mid section	(Row) ρ	f_{ps}	a	Ultimate Design Moment	Ultimate Moment Capacity	Comment
Maximum (1)	Maximum (2)	Can be taken by supplied Cable	Actual at mid Section									
(kN)	(kN)	(kN)	(kN)		(kN)	(kN)		(N/mm²)	(mm)	(kN-m)	(kN-m)	
6552	5424	5654	5327	3.64	5950	4332	0.000947	1807.51	128.26	9075.4436	13757.43	OK

PRESTRESSING REINFORCEMENT DETAILS

Total no. of cable reqd.	Total no. of duct used	Area factor	Total no. of strand in one cable	Dia of one strand	Area of One Strand	Area of One cable	Area of Total Cable (Ap)	Minimum inside diameter of Ducts	Outer diameter of Duct Provided	C/c. Spacing of Ducts Reqd.	C/c. Spacing of Ducts Allowed mid	C/c. Spacing of Ducts Allowed end
(nos)	(nos)			(mm)	(mm2)	(mm2)	(mm2)	(mm)	(mm)	(mm)	(mm)	(mm)
3.53	4	3.666667	12	12.7	98.7	1184.40	4342.80	54.92	65	103.1	125	125

Reinforcement Index	0.057072	**OK against maxm. reinforcrment.**		
Cracking Moment	3311.776 kN-m.	**OK against minm. reinforcement.**	8036.54	2.29

STRESSES AT DIFFERENT LOCATION OF MID-SECTION OF GIRDER

	Initial Tension				Effective Tension									
	Pi (alone)		Pi+Mo (A)		P_e+M_o+M_{dp} (B)		P_e+M_o+M_{dp}+M_{dc} (C)		P_e+M_o+M_{dp}+M_{dc}+M_l +SH (D)		0.5(P_e+M_o+M_{dp}+M_{dc}) +Ml (E)		At top of Deck Slab	At bot. of D eck Slab
	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	(N/mm²)	(N/mm²)
Stress	5.13	-20.63	0.13	-16.11	-5.80	-6.56	-6.69	-4.28	-9.36	-0.51	-4.46	1.32	-2.62	-1.61
All. stress			1.36	-16.50	-14.00	1.47	-14.00	1.47	-21.00	1.47	-14.00	1.47	-12	0.72
Comment			OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

CHECK AT MID SECTION FOR EXTERIOR GIRDER WHEN WHEEL ON SIDEWALK

	Effective Tension					
	P_e+M_o+M_{dp}+M_{dc}+M_l +SH (D)		0.5(P_e+M_o+M_{dp}+M_{dc}) +Ml (E)		At top of Deck Slab	At bot. of D eck Slab
	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	(N/mm²)	(N/mm²)
Stress	0.00	0.00	0.00	0.00	0.00	0.00
All. stress	-31.50	2.21	-21.00	2.21	-18	1.07
Comment	OK	OK	OK	OK	OK	OK

Factor of safety against cracking 1.94

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CABLE LAYOUT

Total no. of Row at mid Section	Row 1	Row 2							
2									
No. of Cable in	3	1							
Area factor	3	0.666667							
Cumilitive No.	3	4	0	0	0	0	0	0	0
Maxm. no. of cable allowed	5	5	5	5	5	5	5	5	5
Distance between Row	95	125							
Cumilitive distance	95	220	0	0	0	0	0	0	0
Distance of C.G. from bot. face of girder	117.73								
Eccentricity for non composite section	805.6555								
Eccentricity for Composite section	1196.11								
Total no. of Row at end Section	**Row 1**	**Row 2**	**Row 3**	**Row 4**					
4									
No. of Cable in	1	1	1	1					
Area factor	1	1	1	0.666667					
Maxm. no. of cable allowed									
Distance between Row	418	350	350	350					
Cumilitive distance	418	768	1118	1468	0	0	0	0	0
Distance of C.G. from bot. face of girder	895.3								
Eccentricity for non composite section	51.23								
Eccentricity for Composite section	294.03								

STRESSES AT DIFFERENT LOCATION OF GIRDER

Cable No.	Mention Row No. at end section	at end section (mm)	At mid section (mm)	Ordinates from bottom at following distances from end 450 (mm)	2250 (mm)	7712.5 (mm)	14975 (mm)	22237.5 (mm)	27700 (mm)	29500 (mm)	1364 (mm)		
1	1	418	95	398.88	328.23	170.97	95.00	170.97	328.23	398.88	361.84	29959.29	**30659.29**
2	2	768	95	728.16	580.96	253.29	95.00	253.29	580.96	728.16	650.98	29990.33	**30690.33**
3	3	1118	95	1057.44	833.68	335.61	95.00	335.61	833.68	1057.44	940.13	30043.18	**30743.18**
4	4	1468	220	1394.12	1121.15	513.53	220.00	513.53	1121.15	1394.12	1251.01	30088.68	**30788.68**
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		**122881.5**
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Dist. of cg from bot.		895.27	117.73	849.24	679.17	300.61	117.73	300.61	679.17	849.24	760.08		
	C1p	881.49	904.62	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881.49		
	C2p	946.51	923.38	946.51	923.38	923.38	923.38	923.38	923.38	946.51	946.51		
	C1c	638.70	514.16	638.70	514.16	514.16	514.16	514.16	514.16	638.70	638.70		
	C2c	1189.30	1313.84	1189.30	1313.84	1313.84	1313.84	1313.84	1313.84	1189.30	1189.30		
	C3c	849.70	725.16	849.70	725.16	725.16	725.16	725.16	725.16	849.70	849.70		
	C4c	649.70	525.16	649.70	525.16	525.16	525.16	525.16	525.16	649.70	649.70		
	Acp	1352038	698996	1352038	698996	698996	698996	698996	698996	1352038	1352038		
	Acc	1796432	1143390	1796432	1143390	1143390	1143390	1143390	1143390	1796432	1796432		
	Icp	3.98E+11	3.05E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+11		
	Icc	7.28E+11	5.86E+11	7.28E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	7.28E+11	7.28E+11		
	ep	51.23	805.66	97.26	244.21	622.78	805.66	622.78	244.21	97.26	186.43		
	ec	294.03	1196.11	340.06	634.67	1013.23	1196.11	1013.23	634.67	340.06	429.23		
	r_p^2	294002	435708	294002	435708	435708	435708	435708	435708	294002	294002		

				450	2250	7712.5	14975	22237.5	27700	29500
For Load case (A) stresses at	top fibre	Allowable stresses		-2.786	-5.012	-1.632	0.125	-1.552	-5.147	-2.887
	bot fibre	Tension	**1.36**	-5.186	-10.355	-14.020	-16.107	-14.654	-10.770	-5.372
	comment	Compn.	**-16.50**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**
For Load case (B) stresses at	top fibre	Allowable stresses		-2.37	-5.50	-5.52	-5.80	-5.48	-5.46	-2.43
	bot fibre	Tension	**1.47**	-4.45	-7.22	-6.91	-6.56	-7.27	-7.48	-4.58
	comment	Compn.	**-14.00**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**
For Load case (C) stresses at	top fibre	Allowable stresses		-2.34	-5.68	-6.17	-6.66	-6.12	-5.64	-2.41
	bot fibre	Tension	**1.47**	-4.43	-6.67	-5.18	-4.26	-5.54	-6.93	-4.55
	comment	Compn.	**-14.00**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**
For Load case (D) stresses at	top fibre	Allowable stresses		-3.99	-7.80	-8.69	-9.33	-8.64	-7.76	-4.06
	bot fibre	Tension	**1.47**	-3.52	-5.19	-2.06	-0.48	-2.42	-5.45	-3.65
	comment	Compn.	**-21.00**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**
	top of slab	Allowable stresses		0.64	-0.29	-1.93	-2.65	-1.93	-0.29	0.64
	bot of slab	Tension	**0.72**	0.78	0.08	-1.11	-1.63	-1.11	0.08	0.78
	comment	Compn.	**-12.00**	**OK-NOK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-NOK**
For Load case (E) stresses at	top fibre	Allowable stresses		-1.17	-3.07	-3.97	-4.46	-3.95	-3.11	-1.20
	bot fibre	Tension	**1.47**	-2.22	-2.47	0.11	1.32	-0.07	-2.60	-2.28
	comment	Compn.	**-14.00**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**	**OK-OK**

TENSION REINFORCEMENT

Yield strength of rebar 414 Mpa, Allowable strength of rebar 165.6 Mpa,

On Girder top for Girder casting

Stress at		Depth of	Total	Rebar	Dia. of	No. of
Top fibre	Bot. fibre	ten. Area	tension	Area reqd.	Rebar	rebar reqd.
(Mpa)	(Mpa)	(mm)	(kN)	(mm^2)	(mm)	
0.125	-16.107	14.12	0.94	5.69	12	0.05

On Girder bottom for service load

Stress at		Depth of	Total	Rebar	Dia. of	No. of
Top fibre	Bot. fibre	ten. Area	tension	Area reqd.	Rebar	rebar reqd.
(Mpa)	(Mpa)	(mm)	(kN)	(mm^2)	(mm)	
-4.455	1.319	417.52	180.17	1087.98	20	3.46

DESIGN FOR SHEAR

Yield strength of rebar 276 Mpa, Allowable strength iof rebar 124.2 Mpa,

ANALYSIS FOR SHEAR

Section	Distance from end	Shear due to								
		Self wt V_O (a)	Cross Girder (Non-Composite dead load (b)	Non Composite dead load (c)	Composite dead load V_{DC} (d)	Live Load with Impact (e)	Live Load with Impact when wheel on Sidewalk (f)	Sidewalk Live Load (g)	Total Dead Load	Factored Design Shear V_U
	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2	450	261.16	11.824	164.282	139.975	253.74	0.00	0.00	577.24	1301.29
2a	1364	232.04	11.824	153.945	131.167	245.52	0.00	0.00	528.98	1220.69
3	2250	209.59	11.824	143.924	122.629	237.52	0.00	0.00	487.96	1150.00
4	7712.5	119.62	11.824	82.141	69.988	187.53	0.00	0.00	283.57	775.78
5	14975	0.00	11.824	0.000	0.000	118.87	0.00	0.00	11.82	273.44
6	22237.5	119.62	11.824	82.141	69.988	187.53	0.00	0.00	283.57	775.78
7	27700	209.59	11.824	143.924	122.629	237.52	0.00	0.00	487.96	1150.00
8	29500	261.16	11.824	164.282	139.975	253.74	0.00	0.00	577.24	1301.29

Section	Distance from end	Diameter of stirrup	No. of Leg	Effective depth for Shear (d)	f2p (stress in the conc. due to Pe)	fo (tensile stress at bot.due to self wt.)	Mcr (moment causing flexural crack)	Vci	Vp	Vcw	Vc (shear strength provided by conc.)	Spacing Reqd.
					(N/mm^2)	(N/mm^2)	(kN-m)	(kN)	(kN)	(kN)	(kN)	(mm)
2	450	12	2	1462	-4.461894	-0.0077	3114.91	479905	469.91	1282.95	1282.95	337
2a	1364	12	2	1462	-5.438445	0.5290	3299.64	3913.31	455.45	1268.55	1268.55	337
3	2250	12	2	1462	-9.584544	1.2634	3716.70	2319.64	424.01	1496.91	1496.91	337
4	7712.5	12	2	1527	-14.46102	3.9895	4425.96	743.94	335.97	1448.91	743.94	344
5	14975	12	2	1710	-16.78044	5.3064	4756.62	293.60	224.95	1467.43	293.60	364
6	22237.5	12	2	1527	-14.82281	3.9895	4545.29	758.31	344.37	1471.82	758.31	344
7	27700	12	2	1462	-9.842597	1.2634	3801.82	2365.96	435.43	1523.47	1523.47	337
8	29500	12	2	1462	-4.585219	-0.0077	3166.70	487891	482.90	1304.30	1304.30	337

HORIZONTAL SHEAR TRANSFER

Section	Distance from end	Effective depth for Shear (d)	Contact surface width	Factored Design Shear	Allowable shear strength (ϕV_{nh})	Comment	Diameter of stirrup	No. of Leg	Minimum spacing required
	(mm)	(mm)	(mm)	(kN)	(kN)		(mm)		(mm)
2	450	979	1066	1301.29	2263	O.K.	12	2	169.75
3	2250	1149	1066	1150.00	2656	O.K.	12	2	169.75
4	7712.5	1527	1066	775.78	3532	O.K.	12	2	169.75
5	14975	1710	1066	273.44	3954	O.K.	12	2	169.75
6	22237.5	1527	1066	775.78	3532	O.K.	12	2	169.75
7	27700	1149	1066	1150.00	2656	O.K.	12	2	169.75
8	29500	979	1066	1301.29	2263	O.K.	12	2	169.75

ELONGATION OF CABLE

Outside jack cable length (assumed), l_j = 700 mm,

Dead anchor width, l_a = 70 mm,

Cable stress at jack (after jack loss), f_j' = 1370 N/mm^2,

Cable stress at dead anchor, f_{LP} = 1258 N/mm^2,

Average cable length of anchor to anchor, L_p = 30004 mm,

Average elongation of cables by one side jacking, ΔL_p = 210 mm,

ANCHORAGE ZONE DESIGN

Minimum longitudinal extent of anchorage zone	=	1066 mm,
Maximum longitudinal extent of anchorage zone	=	1599 mm,
Selected longitudinal extent of anchorage zone	=	1500 mm,

- DESIGN of GENERAL ZONE

α, angle of inclination of the resultant of the tendon = 5.93 deg,

Size of bearing plate	Thickness of bearing plate	Diameter of cone	Effective bearing area (Ab)	Factored tandon load	Total factored tendon load	C/c spacing of anchorage	Longitudinal extent of local zone (lc)	Correction factor (k)	Concrete compressive stress (fca)	Allowable Concrete compressive stress	Comment
(mm)	(mm)	(mm)	(mm^2)	(kN)	(kN)	(mm)	(mm)		(Mpa)	(Mpa)	
230	35	152	49581.69	1947.273	7140	350	264.5	1.271	16.85	21	O.K.

DESIGN FOR VERTICAL DIRECTION

Cable no.	1	2	3	4	0	0	0	0	0	0	0
Eccentricity	528.51	178.51	171.49	521.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00
d_{burst}	658.44	827.68	831.07	661.83	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Reinforcement distribution distance	=	1861.895 mm, Maximum = 2742 mm,
Bursting force, T_{burst}	=	1929.157 kN,
Area of reinforcement required for bursting force	=	5482.117 mm^2,
Diameter of bursting stirrup	=	16 mm,
Total no of 2 legged stirrup required	=	13.63 nos.,

DESIGN FOR HORIZONTAL DIRECTION

Eccentricity (e)	Lateral dimension (h)	Angle of tendon (α)	Bursting distance (d_{burst})	Bursting force (T_{burst})	Area of rebar required	Dia. Of rebar	No. of rebar reqd	Rebar distribution distance
(mm)	(mm)	(deg)	(mm)	(kN)	(mm^2)	(mm)		(mm)
0	711	0	355.5	1207.574	3431.582	16	8.53	888.75

DESIGN FOR EDGE TENSION

Spalling force (T_{spal})	Area of rebar required	Dia. Of rebar	No. of rebar reqd
(kN)	(mm^2)	(mm)	
142.8	406	12	0.03

DEFLECTION CALCULATION

Deflection due to: Initial Prestress	Effective Prestress	Self wt. (at initial period)	Self wt. (after 28 days)	Non-composite dead load	Composite dead load	Instantaneous Deflection	Deflection at erection	Long-time Deflection	Live load	Live load rotation at support
(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(rad.)
45.2	36.8	-18.322	-16.96	-13.2	-5.1	26.9	47.5	-5.4	-7.3	0.001234

DESIGN OF LEFT EXTERIOR PRESTRESSED GIRDER
(Dead load from curb/rail/pedestal equally distributed on girder)

DESIGNED BY : PRASANTA KUMAR BHOWMIK

GENERAL

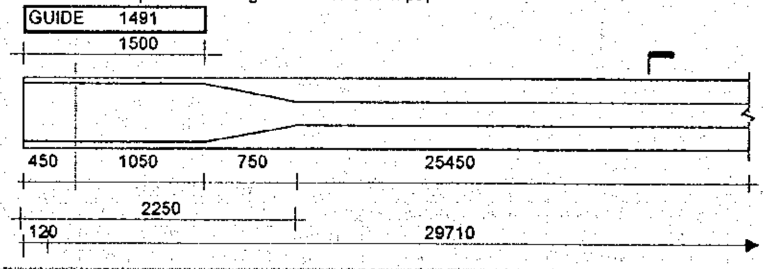
Total Length of one Girder	29950 mm,
Distance of center of Bearing from end of Girder	450 mm,
Span of Girder	29050 mm,
Distance of center of Diaphragm from end of Girder	350 mm,
C/c. spacing between girders	2400 mm,
Effective width of sidewalk	1200 mm,
Fraction of wheel load on Girder	0.559
Fraction of wheel load on girder when wheel over sidewalk	1.651
Live Load Magnitude	HS 20
Put 1.0 or 2.0 for Deck type	
Mention AASHTO or WASHINGTON Pattern	AASHTO
Mention Type	6
Mention end section 1 or 2?	2
Anchorage indent	120 mm,
Tensile Stress due to temparature change	0.0246 Mpa,

DIAPHRAGM DETAILS

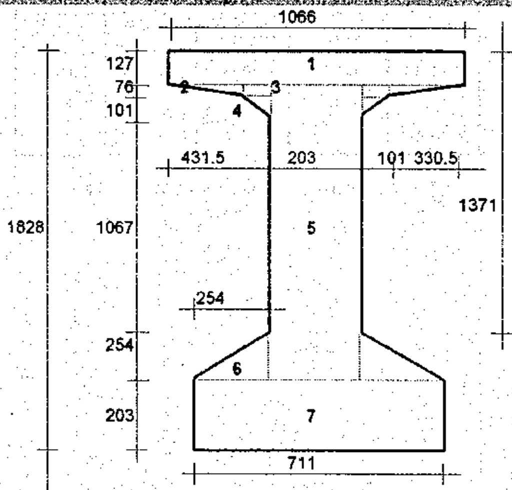
Total no. of interior diaphragm reqd	2
Total no. of interior diaphragm provided	1
Width of diaphragm	300 mm,
Depth of end diaphragm	1867 mm,
Depth of interior diaphragm	1625 mm,
Wt. of exterior diaphragm	13.39 kN,
Wt. of interior diaphragm	11.82 kN,

NOTE: In designing this girder following references are followed :
1. AASHTO - 16th edition, 1996.
2. Design of Prestressed Concrete, 2nd edition, by : Arthur H. Nilson.
3. Design of Prestressed Concrete Structure, 3rd edition, by : T. Y. Lin & Ned H. Burns.

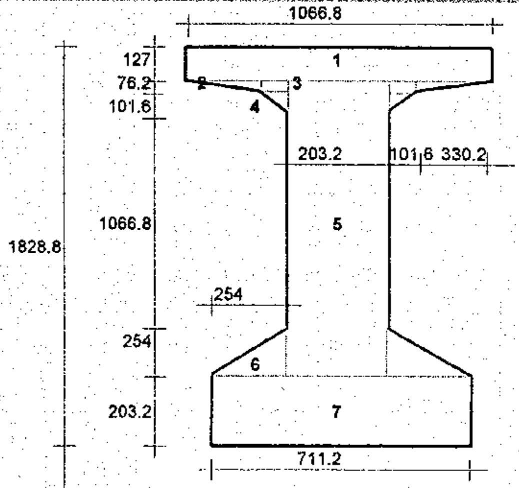
All notations are followed from reference-2



GIRDER SECTION



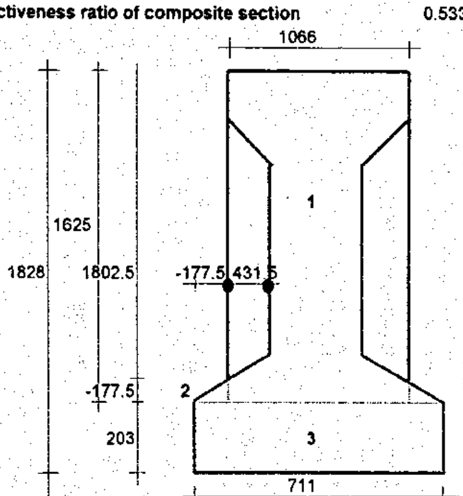
SECTION OF GIRDER IN USE



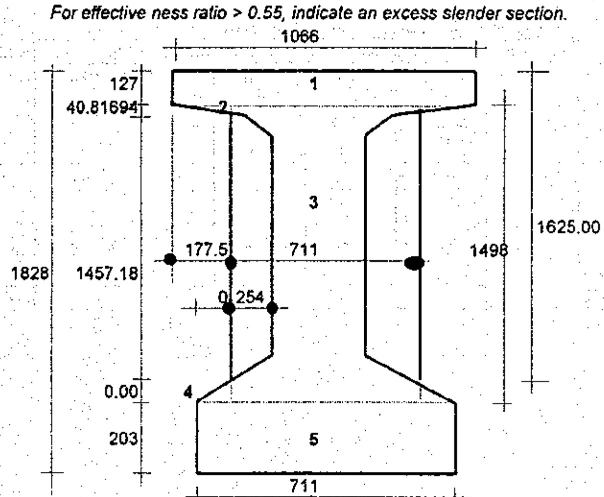
SECTION OF STANDARD GIRDER

Effectiveness ratio of non-composite section	0.521613
Effectiveness ratio of composite section	0.533982

Note : *For effectiveness ratio < 0.45, indicate too heavy a section*
For effective ness ratio > 0.55, indicate an excess slender section.

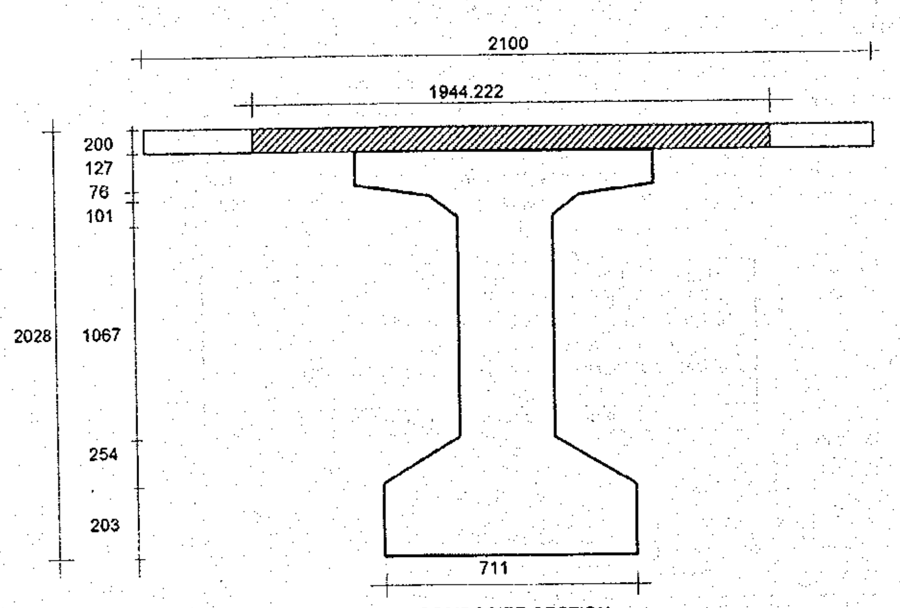


END SECTION 1



END SECTION 2

11-15



COMPOSITE SECTION

PROPERTIES OF PRECAST MIDDLE SECTION

Segment	Area	Distance of CG from		Moment of Inertia of		Radius of Gyration
		Top C_{1P}	Bottom C_{2P}	Components I_o	Block Ic	r^2
	mm^2	mm	mm	mm^4	mm^4	mm^2
1	135382			1.82E+08		
2	12559			4030044		
3	7676			3694715		
4	5100.5	904.62	923.38	2890567	3.05E+11	435707.6
5	304094			5.69E+10		
6	32258			1.16E+08		
7	144333			4.96E+08		
Total	698996					

Weight per linear metre = 16.47 kN/m.

PROPERTIES OF PRECAST END SECTION

Segment	Area	Distance of CG from		Moment of Inertia of		Radius of Gyration
		Top C1P	Bottom C2P	Components IO	Block Ic	r2
	mm2	mm	mm	mm4	mm4	mm2
1	135382			1.82E+08		
2	3623			335287		
3	1065078	881.49	946.51	1.99E+11	3.98E+11	294002
4	0			0		
5	144333			4.96E+08		
Total	1352038					

Weight per linear metre = 31.86 kN/m.

PROPERTIES OF EQUIVALENT COMPOSITE SECTION AT MIDDLE

Segment	Area	Distance of CG of Composite section from				Moment of Inertia of		Radius of Gyration
		Top of precast section C_{1C}	Bot. of precast section C_{2C}	Bottom of Slab C_{4C}	Top of Slab C_{3C}	Components I_o	Block Ic	r^2
	mm^2	mm	mm	mm	mm	mm^4	mm^4	mm^2
Precast Section	698996	545.5206	1282.479	556.5206	756.5206	3.05E+11	5.64E+11	518080.8
Eqv. Slab	388844.4					1.30E+09		
Total	1087840							

Weight per linear ft. = 25.63 kN/m.

PROPERTIES OF EQUIVALENT COMPOSITE SECTION AT END

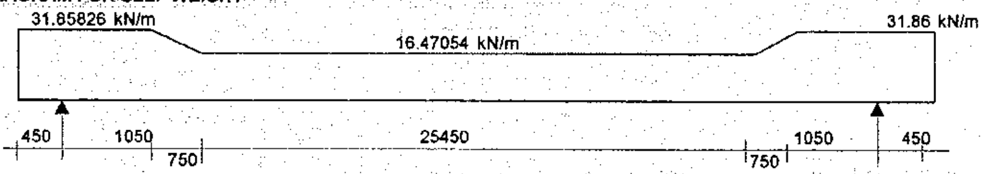
Segment	Area	Distance of CG of Composite section from				Moment of Inertia of		Radius of Gyration
		Top of precast section C_{1C}	Bot. of precast section C_{2C}	Bottom of Slab C_{4C}	Top of Slab C_{3C}	Components I_o	Block Ic	r^2
	mm^2	mm	mm	mm	mm	mm^4	mm^4	mm^2
Precast Section	1352038	662.2677	1165.732	673.2677	873.2677	3.98E+11	6.96E+11	399960.8
Eqv. Slab	388844.4					1.30E+09		
Total	1740882							

Weight per linear ft. = 41.02 kN/m.

SUMMERY OF SECTION PROPERTIES AT DIFFERENT LOCATIONS

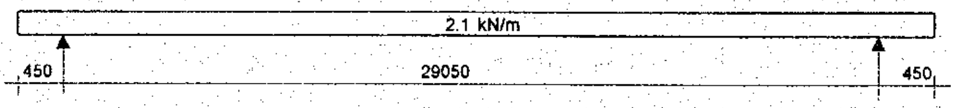
Section			1	2	3	4	5	6	7	8	9
Distance from end		(mm)	0	450.00	2250	7712.5	14975	22237.5	27700	29500	29950
Total area of duct		(mm)	13273.23	13273.23	13273.23	13273.23	13273.23	13273.23	13273.229	13273.23	13273.23
Dist.of c.g. of duct from bot.		(mm)	895.27	849.24	679.17	300.61	117.73	300.61	679.17	849.24	895.27
Duct deducted properties of noncomposite section	Acp	(mm)	1338765	1338765	685723	685723	685723	685723	685723	1338765	1338765
	C_{1P}	(mm)	880.99	880.53	899.89	892.56	889.02	892.56	899.89	880.53	880.99
	C_{2P}	(mm)	947.01	947.47	928.11	935.44	938.98	935.44	928.11	947.47	947.01
	Ic	(mm^4)	3.97E+11	3.97E+11	3.04E+11	2.99E+11	2.96E+11	2.99E+11	3.04E+11	3.97E+11	3.97E+11
	r^2	(mm^2)	296888	296819	442937	436335	431081	436335	442937	296819	296888
Duct grouted properties of noncomposite section	Acp	(mm^2)	1352038	1352038	698996	698996	698996	698996	698996	1352038	1352038
	C_{1P}	(mm)	881.49	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881.49
	C_{2P}	(mm)	946.51	946.51	923.38	923.38	923.38	923.38	923.38	946.51	946.51
	Ic	(mm^4)	3.98E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+11
	r^2	(mm^2)	294002	294002	435707.6	435707.6	435707.6	435707.6	435707.6	294002.2	294002.2
Duct grouted properties of composite section	Acc	(mm^2)	1740882	1740882	1087840	1087840	1087840	1087840	1087840	1740882	1740882
	C_{1c}	(mm)	662.27	662.27	545.52	545.52	545.52	545.5	545.5	662.3	662.3
	C_{2c}	(mm)	1165.73	1165.73	1282.48	1282.48	1282.48	1282.5	1282.5	1165.7	1165.7
	C_{4c}	(mm)	673.27	673.27	556.52	556.52	556.52	556.5	556.5	673.3	673.3
	C_{3c}	(mm)	873.27	873	756.5	756.5	756.5	756.5	756.5	873.3	873.3
	Ic	(mm^4)	6.96E+11	6.96E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	6.96E+11	6.96E+11
	r^2	(mm^2)	399960.8	399960.8	518080.8	518080.8	518080.8	518080.8	518080.8	399960.8	399960.8

LOAD DIAGRAM FOR SELF WEIGHT



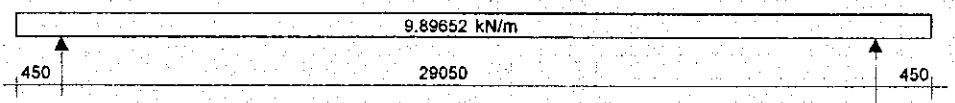
Reaction 275.4983 kN,

LOAD DIAGRAM FOR FALSE-WORK DEAD LOAD



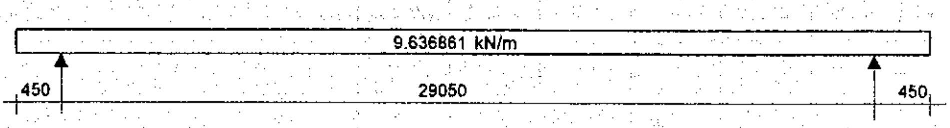
Reaction 31.4475 kN,

LOAD DIAGRAM FOR NON-COMPOSITE DEAD LOAD



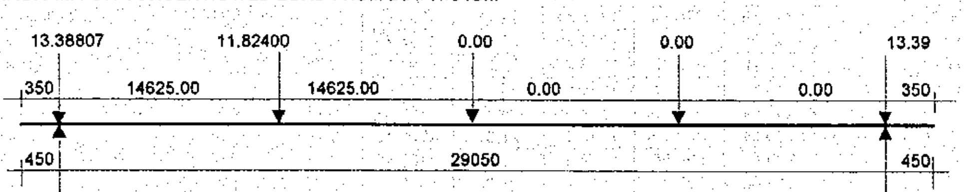
Reaction 148.2004 kN,

LOAD DIAGRAM FOR COMPOSITE DEAD LOAD



Reaction 144.312 kN,

LOAD DIAGRAM FOR CONCENTRATED LOAD FROM DIAPHRAGM



Reaction 19.30006 kN,

ANALYSIS MOR MOMENT

Section	Distance from end	Moment due to Self wt M_O (a)	Cross Girder (Non-Composite dead load (b)	Non Composite dead load (c)	Composite dead load M_{DC} (d)	Live Load with Impact (e)	Live Load with Impact when wheel on Sidewalk (f)	Sidewalk Live Load (g)	Total (Non-Composite dead load) M_{DP} (b+c)	Falsework (Non-Composite dead load	Total Dead load Moment	Total Factored Moment
	(mm)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	450	-3.23	-1.34	-1.00	-0.98	0.00	0.00	0.00	-2.34	-0.21	-6.54	-8.50
2a	1364	222.17	4.06	126.25	122.94	87.31	258.03	44.32	130.31	26.79	475.42	953.49
3	2250	416.70	9.30	241.71	235.37	165.91	490.36	84.53	251.01	51.29	903.08	1811.47
4	7712.5	1315.84	41.60	781.97	761.45	519.58	1535.64	272.70	823.57	165.93	2900.86	5767.45
5	14975	1750.20	84.53	1042.96	1015.60	666.76	1970.64	363.60	1127.49	221.31	3893.29	7623.11
6	22237.5	1315.84	41.60	781.97	761.45	519.58	1535.64	272.70	823.57	165.93	2900.86	5767.45
7	27700	416.70	9.30	241.71	235.37	165.91	490.36	84.53	251.01	51.29	903.08	1811.47
8	29500	-3.23	-1.34	-1.00	-0.98	0.00	0.00	0.00	-2.34	-0.21	-6.54	-8.50
9	29950	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

LOSSES

f_{ps} 144.20

Frictional loss per unit length 14.20246 kN/m,

For 8mm Anchorage draw in, x(dist. Of anchorage loss zone) 21.7313 mm,

ΔP_A 617.2771 kN,

Prestress force after friction losses (before elastic shortening) :-

- Tensioning end 5333 kN,
- Seating loss zone 5641 kN, No. of cable reqd 3.64
- Midspan 5545 kN,
- Dead end 5525 kN,

SUMMERY OF INITIAL LOSSES

Section	Distance from end	Jacking Force	Losses due to friction and anchorage pull-in	Prestress Force after friction loss (before elastic shortening)	Conc. Stress at the level of steel centroid at sec. of maxm. mom. (f_{ci})	Loss due to elastic shortening	Final initial loss (after immediate losses)	Final initial prestress force (after immediate losses) (Pi)
	(mm)	(kN)	(kN)	(kN)	(mPa)	(kN)	(kN)	(kN)
1	0		617	5333	3.935	59.82	677	5273
2	450		611	5339	4.030	61.27	672	5278
2a	1364		598	5352	4.269	64.90	663	5287
3	2250		585	5365	8.189	124.50	710	5240
4	7712.5	5950	508	5442	11.551	175.61	683	5267
5	14975		405	5545	14.345	218.10	623	5327
6	22237.5		316	5634	12.049	183.19	499	5451
7	27700		393	5557	8.493	129.13	523	5427
8	29500		419	5531	4.175	63.47	482	5468
9	29950		425	5525	4.076	61.98	487	5463

SUMMERY OF TIME-DEPENDENT & FINAL LOSSES

Section	Distance from end	Conc. Stress at the level of steel cg. Due to dead loads except self wt. (f_{cds})	Loss due to creep	Loss due to shrinkage	Loss due to relaxation of pre-stressing steel	Final Time dependent loss	Total loss	Loss in percent	Final effective pre-stress force (Pe)
	(mm)	(mPa)	(kN)	(kN)	(kN)	(kN)	(kN)		(kN)
1	0	0.000	205.06		317.59	678.34	1355.44	22.78	4594.56
2	450	-0.002	210.06		317.92	683.69	1355.85	22.79	4594.15
2a	1364	0.148	217.97		318.79	692.46	1355.26	22.78	4594.74
3	2250	0.521	410.92		260.13	826.75	1536.57	25.82	4413.43
4	7712.5	2.761	518.01		241.54	915.25	1598.61	26.87	4351.39
5	14975	4.429	612.94	155.70	236.51	1005.14	1627.84	27.36	4322.16
6	22237.5	2.761	543.99		290.89	990.58	1489.60	25.04	4460.40
7	27700	0.521	426.79		312.68	895.17	1417.71	23.83	4532.29
8	29500	-0.002	217.61		373.11	746.42	1228.87	20.65	4721.13
9	29950	0.000	212.44		372.82	740.96	1228.30	20.64	4721.70

EFFECT OF DIFFERENTIAL SHRINKAGE

Ultimate shrinkage co-efficient, ε_{SHU}	0.0008
Correction by humidity, (RH = 70) , $F_{SH,H}$	0.7
Assumed day of slab casting after girder concrete placed, t	60 days
Shrinkage of girder at t days, $\varepsilon_{SH,t}$	0.632
Differential (remaining) shrinkage, $\varepsilon_{SH,D}$	0.000206
Ultimate creep coefficient for girder, φ_{cgu}	2.6
Ultimate creep coefficient for slab, φ_{csu}	2.8
Correction by humidity for creep coefficient, (RH = 70), $F_{C,H}$	0.801
Remaining creep of girder after t days, φ_{cg}	0.961267
Remaining creep of slab, φ_{cs}	2.2428

Horizontal Shear & Bending Moment on contact surface of girder & slab due to Differential Shrinkage

At all section like mid section

m	B	C	F	φ_{cg}	φ_{cs}	Girder		Slab	
						V_{SH}	M_{SH}	V_{SH}	M_{SH}
		(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
253.798	254.798	-27.267	5.227	0.961	2.243	413.201	44.218	249.907	26.744

	Girder		Slab	
	Bottom	Top	Bottom	Top
Stress at concrete due to differential shrinkage	0.676	-1.833	0.674	0.524

Horizontal Shear & Bending Moment on contact surface of girder & slab due to Differential Shrinkage

At all section like end section

m	B	C	F	φ_{cg}	φ_{cs}	Girder		Slab	
						V_{SH}	M_{SH}	V_{SH}	M_{SH}
		(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
331.252	332.252	-35.998	6.257	0.961	2.243	528.518	57.263	319.651	34.633

	Girder		Slab	
	Bottom	Top	Bottom	Top
Stress at concrete due to differential shrinkage	0.855	-1.551	0.828	0.700

DESIGN FOR MOMENT AT MID SECTION

Total Jacking tension canbe taken by supplied cable (after jack loss) 5993.99 kN.

Initial Tension				Total no. of Cable reqd.	Jacking Tension (after Jack loss)	Effective Tension at mid section	(Row) ρ	f_{ps}	a	Ultimate Design Moment	Ultimate Moment Capacity	Comment
Maximum (1)	Maximum (2)	Can be taken by supplied Cable	Actual at mid Section									
(kN)	(kN)	(kN)	(kN)		(kN)	(kN)		(N/mm^2)	(mm)	(kN-m)	(kN-m)	
6552	5424	5654	5327	3.64	**5950**	4322	0.001083	1800.012	145.98	7623.112	13633.27	OK

PRESTRESSING REINFORCEMENT DETAILS

Total no. of cable reqd.	Total no. of duct used	Area factor	Total no. of strand in one cable	Dia of one strand	Area of One Strand	Area of One cable	Area of Total Cable (Ap)	Minimum inside diameter of Ducts	Outer diameter of Duct Provided	C/c Spacing of Ducts Reqd.	C/c Spacing of Ducts Allowed mid	C/c Spacing of Ducts Allowed end
(nos)	(nos)			(mm)	(mm2)	(mm2)	(mm2)	(mm)	(mm)	(mm)	(mm)	(mm)
3.53	4	3.666667	12	12.7	98.7	1184.40	4342.80	54.92	65	103.1	125	125

Reinforcement Index	0.064955	**OK against maxm. reinforcrment.**	
Cracking Moment	3379.928 kN-m,	**OK against minm. reinforcement.**	8024.59 6.20

STRESSES AT DIFFERENT LOCATION OF MID-SECTION OF GIRDER

	Initial Tension				Effective Tension									
	Pi (alone)		Pi+Mo (A)		$P_e+M_o+M_{dp}$ (B)		$P_e+M_o+M_{dp}+M_{dc}$ (C)		$P_e+M_o+M_{dp}+M_{dc}+M_l$ +SH (D)		$0.5(P_e+M_o+M_{dp}+M_{dc})$ +Ml (E)		At top of Deck Slab	At bot. of Deck Slab
	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	(N/mm^2)	(N/mm^2)
Stress	5.13	-20.63	0.13	-16.11	-5.02	-7.32	-6.00	-5.01	-7.82	-3.49	-3.31	-1.31	-1.54	-0.84
All. stress			1.36	-16.50	-14.00	1.47	-14.00	1.47	-21.00	1.47	-14.00	1.47	-12	0.72
Comment			OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

CHECK AT MID SECTION FOR EXTERIOR GIRDER WHEN WHEEL ON SIDEWALK

	Effective Tension					
	$P_e+M_o+M_{dp}+M_{dc}+M_l$ +SH (D)		$0.5(P_e+M_o+M_{dp}+M_{dc})$ +Ml (E)		At top of Deck Slab	At bot. of Deck Slab
	Top (N/mm^2)	Bottom (N/mm^2)	Top (N/mm^2)	Bottom (N/mm^2)	(N/mm^2)	(N/mm^2)
Stress	-9.09	-0.52	-4.57	1.66	-3.16	-2.03
All. stress	-31.50	2.21	-21.00	2.21	-18	1.07
Comment	OK	OK	OK	OK	OK	OK

Factor of safety against cracking 5.07

CABLE LAYOUT

Total no. of Row at mid Section 2	Row 1	Row 2							
No. of Cable in	3	1							
Area factor	3	0.666667							
Cumilitive No.	3	4	0	0	0	0	0	0	0
Maxm. no. of cable allowed	5	5	5	5	5	5	5	5	5
Distance between Row	95	125							
Cumilitive distance	95	220	0	0	0	0	0	0	0
Distance of C.G. from bot. face of girder	117.73								
Eccentricity for non composite section	805.6555								
Eccentricity for Composite section	1164.75								
Total no. of Row at end Section 4	**Row 1**	**Row 2**	**Row 3**	**Row 4**					
No. of Cable in	1	1	1	1					
Area factor	1	1	1	0.666667					
Maxm. no. of cable allowed									
Distance between Row	418	350	350	350					
Cumilitive distance	418	768	1118	1468	0	0	0	0	0
Distance of C.G. from bot. face of girder	895.3								
Eccentricity for non composite section	51.23								
Eccentricity for Composite section	270.46								

STRESSES AT DIFFERENT LOCATION OF GIRDER

Cable No.	Mention Row No. at end section	at end section (mm)	At mid section (mm)	Ordinates from bottom at following distances from end 450 (mm)	2250 (mm)	7712.5 (mm)	14975 (mm)	22237.5 (mm)	27700 (mm)	29500 (mm)	1364 (mm)		
1	1	418	95	398.88	328.23	170.97	95.00	170.97	328.23	398.88	361.84	29959.29	30659.29
2	2	768	95	728.16	580.96	253.29	95.00	253.29	580.96	728.16	650.98	29990.33	30690.33
3	3	1118	95	1057.44	833.68	335.61	95.00	335.61	833.68	1057.44	940.13	30043.18	30743.18
4	4	1468	220	1394.12	1121.15	513.53	220.00	513.53	1121.15	1394.12	1251.01	30088.68	30788.68
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		122881.5
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Dist. of cg from bot.		895.27	117.73	849.24	679.17	300.61	117.73	300.61	679.17	849.24	760.08		
C1p		881.49	904.62	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881.49		
C2p		946.51	923.38	946.51	923.38	923.38	923.38	923.38	923.38	946.51	946.51		
C1c		662.27	545.52	662.27	545.52	545.52	545.52	545.52	545.52	662.27	662.27		
C2c		1165.73	1282.48	1165.73	1282.48	1282.48	1282.48	1282.48	1282.48	1165.73	1165.73		
C3c		873.27	756.52	873.27	756.52	756.52	756.52	756.52	756.52	873.27	873.27		
C4c		673.27	556.52	673.27	556.52	556.52	556.52	556.52	556.52	673.27	673.27		
Acp		1352038	698996	1352038	698996	698996	698996	698996	698996	1352038	1352038		
Acc		1740882	1087840	1740882	1087840	1087840	1087840	1087840	1087840	1740882	1740882		
Icp		3.98E+11	3.05E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+11		
Icc		6.96E+11	5.64E+11	6.96E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	6.96E+11	6.96E+11		
ep		51.23	805.66	97.26	244.21	622.78	805.66	622.78	244.21	97.26	186.43		
ec		270.46	1164.75	316.49	603.31	981.87	1164.75	981.87	603.31	316.49	405.65		
r_p^2		294002	435708	294002	435708	435708	435708	435708	435708	294002	294002		

For Load case (A) stresses at	top fibre	Allowable stresses		-2.786	-5.012	-1.632	0.125	-1.552	-5.147	-2.887
	bot fibre	Tension	1.36	-5.186	-10.355	-14.020	-16.107	-14.654	-10.770	-5.372
	comment	Compn.	-16.50	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK
For Load case (B) stresses at	top fibre	Allowable stresses		-2.37	-5.35	-5.00	-5.02	-4.95	-5.31	-2.44
	bot fibre	Tension	1.47	-4.45	-7.38	-7.43	-7.32	-7.79	-7.64	-4.57
	comment	Compn.	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK
For Load case (C) stresses at	top fibre	Allowable stresses		-2.34	-5.55	-5.71	-5.98	-5.67	-5.51	-2.41
	bot fibre	Tension	1.47	-4.43	-6.82	-5.67	-4.98	-6.03	-7.08	-4.55
	comment	Compn.	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK
For Load case (D) stresses at	top fibre	Allowable stresses		-3.90	-7.40	-7.55	-7.80	-7.51	-7.35	-3.96
	bot fibre	Tension	1.47	-3.57	-5.92	-4.31	-3.46	-4.68	-6.18	-3.70
	comment	Compn.	-21.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK
	top of slab	Allowable stresses		0.70	0.03	-1.07	-1.57	-1.07	0.03	0.70
	bot of slab	Tension	0.72	0.83	0.31	-0.50	-0.86	-0.50	0.31	0.83
	comment	Compn.	-12.00	OK-NOK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-NOK
For Load case (E) stresses at	top fibre	Allowable stresses		-1.17	-2.80	-3.11	-3.31	-3.09	-2.84	-1.21
	bot fibre	Tension	1.47	-2.21	-3.11	-1.90	-1.31	-2.09	-3.24	-2.28
	comment	Compn.	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK

TENSION REINFORCEMENT

Yield strength of rebar 414 Mpa, Allowable strength of rebar 165.6 Mpa,

On Girder top for Girder casting

Stress at Top fibre (Mpa)	Stress at Bot. fibre (Mpa)	Depth of ten. Area (mm)	Total tension (kN)	Rebar Area reqd. (mm^2)	Dia. of Rebar (mm)	No. of rebar reqd.
0.125	-16.107	14.12	0.94	5.69	12	0.05

On Girder bottom for service load

Stress at Top fibre (Mpa)	Stress at Bot. fibre (Mpa)	Depth of ten. Area (mm)	Total tension (kN)	Rebar Area reqd. (mm^2)	Dia. of Rebar (mm)	No. of rebar reqd.
-3.306	-1.311	0.00	#VALUE!	#VALUE!	20	#VALUE!

DESIGN FOR SHEAR

Yield strength of rebar 276 Mpa, Allowable strength iof rebar 124.2 Mpa,

ANALYSIS FOR SHEAR

Section	Distance from end	Shear due to: Self wt V_D (a)	Cross Girder (Non-Composite dead load) (b)	Non Composite dead load (c)	Composite dead load V_{DC} (d)	Live Load with Impact (e)	Live Load with Impact when wheel on Sidewalk (f)	Sidewalk Live Load (g)	Total Dead Load	Factored Design Shear V_U
	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2	450	261.16	5.912	143.747	139.975	98.99	292.56	0.16	550.80	1096.37
2a	1364	232.04	5.912	134.702	131.167	95.78	283.08	0.15	503.82	1022.97
3	2250	209.59	5.912	125.933	122.629	92.66	273.85	0.14	464.06	959.29
4	7712.5	119.62	5.912	71.873	69.988	73.16	216.23	0.09	267.39	628.70
5	14975	0.00	5.912	0.000	0.000	46.37	137.06	0.04	5.91	185.86
6	22237.5	119.62	5.912	71.873	69.988	73.16	216.23	0.09	267.39	628.70
7	27700	209.59	5.912	125.933	122.629	92.66	273.85	0.14	464.06	959.29
8	29500	261.16	5.912	143.747	139.975	98.99	292.56	0.16	550.80	1096.37

Section	Distance from end	Diameter of stirrup	No. of Leg	Effective depth for Shear (d)	f2p (stress in the conc. due to Pe)	fo (tensile stress at bot.due to self wt.)	Mcr (moment causing flexural crack)	Vci	Vp	Vcw	Vc (shear strength provided by conc.)	Spacing Reqd.
					(N/mm^2)	(N/mm^2)	(kN-m)	(kN)	(kN)	(kN)	(kN)	(mm)
2	450	12	2	1462	-4.461911	-0.0077	3114.92	546451	469.92	1282.95	1282.95	337
2a	1364	12	2	1462	-5.438022	0.5290	3299.46	3900.21	455.41	1268.49	1268.49	337
3	2250	12	2	1462	-9.581729	1.2634	3715.78	2306.99	423.69	1496.62	1496.62	337
4	7712.5	12	2	1527	-14.44142	3.9895	4419.50	726.50	335.51	1447.67	726.50	344
5	14975	12	2	1710	-16.74092	5.3064	4743.58	289.94	224.42	1465.38	289.94	364
6	22237.5	12	2	1527	-14.80321	3.9895	4538.83	740.42	343.92	1470.58	740.42	344
7	27700	12	2	1462	-9.839782	1.2634	3800.89	2353.02	435.30	1523.18	1523.18	337
8	29500	12	2	1462	-4.585236	-0.0077	3166.71	555543	482.90	1304.31	1304.31	337

HORIZONTAL SHEAR TRANSFER

Section	Distance from end	Effective depth for Shear (d)	Contact surface width	Factored Design Shear	Allowable shear strength (ϕVnh)	Comment	Diameter of stirrup	No. of Leg	Minimum spacing required
	(mm)	(mm)	(mm)	(kN)	(kN)		(mm)		(mm)
2	450	979	1066	1096.37	2263	O.K.	12	2	169.75
3	2250	1149	1066	959.29	2656	O.K.	12	2	169.75
4	7712.5	1527	1066	628.70	3532	O.K.	12	2	169.75
5	14975	1710	1066	185.86	3954	O.K.	12	2	169.75
6	22237.5	1527	1066	628.70	3532	O.K.	12	2	169.75
7	27700	1149	1066	959.29	2656	O.K.	12	2	169.75
8	29500	979	1066	1096.37	2263	O.K.	12	2	169.75

ELONGATION OF CABLE

Outside jack cable length (assumed), l_j = 700 mm,

Dead anchor width, l_a = 70 mm,

Cable stress at jack (after jack loss), f_j' = 1370 N/mm^2,

Cable stress at dead anchor, f_{LP} = 1258 N/mm^2,

Average cable length of anchor to anchor, L_p = 30004 mm,

Average elongation of cables by one side jacking, ΔL_p = 210 mm,

ANCHORAGE ZONE DESIGN

Minimum longitudinal extent of anchorage zone = 1066 mm,
Maximum longitudinal extent of anchorage zone = 1599 mm,
Selected longitudinal extent of anchorage zone = 1500 mm,

- DESIGN of GENERAL ZONE

α, angle of inclination of the resultant of the tendon = 5.93 deg,

Size of bearing plate	Thickness of bearing plate	Diameter of cone	Effective bearing area (Ab)	Factored tandon load	Total factored tendon load	C/c spacing of anchorage	Longitudinal extent of local zone (lc)	Correction factor (k)	Concrete compressive stress (fca)	Allowable Concrete compressive stress	Comment
(mm)	(mm)	(mm)	(mm^2)	(kN)	(kN)	(mm)	(mm)		(Mpa)	(Mpa)	
230	35	152	49581.69	1947.273	7140	350	264.5	1.271	16.85	21	O.K.

DESIGN FOR VERTICAL DIRECTION

Cable no.	1	2	3	4	0	0	0	0	0	0	0
Eccentricity	528.51	178.51	171.49	521.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dburst	656.44	827.68	831.07	661.83	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Reinforcement distribution distance	=	1861.895 mm, Maximum = 2742 mm,
Bursting force, Tburst	=	1929.157 kN,
Area of reinforcement required for bursting force	=	5482.117 mm^2,
Diameter of bursting stirrup	=	16 mm,
Total no of 2 legged stirrup required	=	13.63 nos.,

DESIGN FOR HORIZONTAL DIRECTION

Eccentricity (e)	Lateral dimension (h)	Angle of tendon (α)	Bursting distance (dburst)	Bursting force (Tburst)	Area of rebar required	Dia. Of rebar	No. of rebar reqd	Rebar distribution distance
(mm)	(mm)	(deg)	(mm)	(kN)	(mm^2)	(mm)		(mm)
0	711	0	355.5	1207.574	3431.582	16	8.53	888.75

DESIGN FOR EDGE TENSION

Spalling force (Tspal)	Area of rebar required	Dia. Of rebar	No. of rebar reqd
(kN)	(mm^2)	(mm)	
142.8	406	12	0.03

DEFLECTION CALCULATION

Deflection due to						Instantaneous Deflection	Deflection at erection	Long-time Deflection	Live load	Live load rotation at support
Initial Prestress	Effective Prestress	Self wt. (at initial period)	Self wt. (after 28 days)	Non-composite dead load	Composite dead load					
(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(rad.)
45.2	36.7	-18.322	-16.96	-10.9	-5.3	26.9	47.5	-1.0	-3.0	0.000482

DESIGN OF REINFORCED ELASTOMERIC BEARING (AASHTO-1992, METHOD-A)

Developed & designed by : Prasanta Kumar Bhowmik.

50% SHEAR DEFORMATION DUE TO CREEP & SHRINKAGE ALLOWED

INPUT OF LOAD/STRESS/GEOMETRY

Dead Load Reaction	=	627	kN
Live Load Reaction without Impact	=	207	kN
Live Load Rotation	=	0.0013	Radian
Total length of Girder	=	30	m
Width of Girder	=	711	mm
Temperature Change	=	21	°C
Hardness of Bearing Material	=	60	Dur.
Yield Strength of Steel Laminates, f_y	=	248	MPa
Fatigue Strength of Steel Laminates, f_{sr}	=	165	MPa

INPUT OF BEARING DIMENSIONS

Width of Elastomeric Bearing $_{(Trans.)}$	=	300	mm
Length of Elastomeric Bearing $_{(Long.)}$	=	400	mm
Thickness of internal layers, h_{ri}	=	10	mm
Thickness of Cover Layer	=	8.5	mm
Total no of Internal Layer of Elastomer	=	0	O.K.
Thickness of Steel Plates	=	3	mm
Total thickness of Bearing	=	20	mm

Design Shear Force To Substructure, H = 75.7059 Kn

Creep & Shrinkage	=	7.500	mm
Temperature shortening	=	0.000	mm
Shear deformation of the Bearing, Δ_s	=	7.500	mm
Shear Modulus, G_{min}	=	0.930	MPa
Shear Modulus, G_{max}	=	1.430	MPa
Creep deflection at 25 years	=	35.000	%
Instantaneous Deflection k_b	=	0.600	
Minm. Elastomer Thickness for shear, h_{rt}	=	15.00	mm
Minimum Area Required	=	119933	mm^2
Minimum Dimension for Stability (L or W)	≥	45	mm
Minimum length against Compression	=	400	mm
Minimum Shape Factor	=	7.47	
Maxm. thick. of each elastomer layer, h_{ri}	=	11.5	mm
Total No. of Layer required, N	=	-0.20	
Thickness Reqd. for Steel Plates	=	0.71	mm
Actual Shape Factor, S	=	8.57	O.K.
Effective Compressive Modulus of the Elastomer, E_c	=	382.51	MPa
Instantaneous Compressive Deflection of Bearing Δ_c	=	0.31	mm
θL	=	0.4	mm
			O.K.

Project : .... Designed : ....

Structure : .... Date : ....

Item : ....

DESIGN OF CONNECTION

Dead Load Reaction from

- exterior girder = 275.5 + 148.20 + 117.89 + 18.95
  = 560.54 kN.
- interior girder = 275.5 + 169.37 + 117.89 + 37.90
  = 600.66 kN.

∴ Total Dead Load Reaction

= 2 × 560.54 + 5 × 600.66

= 4124.38 kN.

∴ Connection Design force

= 0.2 × 4124.38 AASHTO I-A-5.2

= 824.88 kN.

Let us provide 2×7 = 14 nos 25φ bar as connection bar

f_y = 414 MPa

f_v = 0.4 f_y (Table 10.32.1A of AASHTO)

= 165 MPa

A_v = 490.87 mm^2

∴ strength of 14-25φ bars against shear

= 14 × 490.87 × 165/1000

= 1133 kN > 824.88 kN O.K.

Sheet .... of ....

Project : ....
Structure : ....
Item : ....

Designed : ....
Date : ....

DESIGN OF END DIAPHRAGM FOR CONCENTRATED LOAD FROM PIN

Longitudinal force in each connection bar

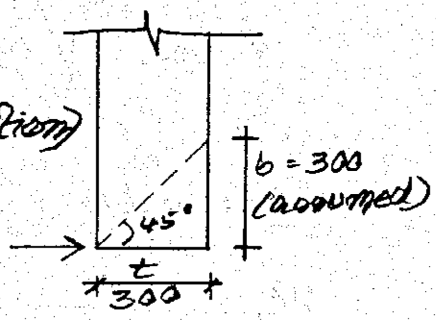
$= 824.88/14 = 58.92$ kN

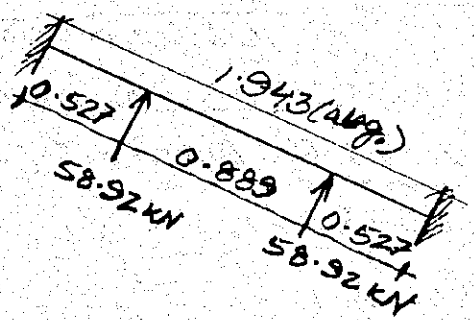
Stresses due to above load will be concentrate at bottom region of diaphragm.

For longitudinal load

$t = 300$ mm

$b = 300$ mm (assuming 45° distribution)





Fixed end moment

$$= \frac{58.92 \times 0.527 \times 1.416^2}{1.943^2} + \frac{58.92 \times 1.416 \times 0.527^2}{1.943^2}$$

$= 22.63$ kN.m.

Reaction $= 58.92$ kN

Mid span moment $= 58.92 \times 0.9715 - 22.63 - 58.92 \times 0.4445$

$= 8.42$ kN.m.

Project : .... Designed : ....
Structure : .... Date : ....
Item : ....

working stress design

$f_c' = 30\ N/mm^2$
$f_c = 12\ N/mm^2$
$f_y = 414\ N/mm^2$
$f_s = 165.6\ N/mm^2$
$R = 1.932$
$j = 0.878$
$k = 0.367$

$d_{reqd.} = \sqrt{M/Rb} = \sqrt{\dfrac{22.63 \times 1000^2}{1.932 \times 300}}$
$= 197.60\ mm$

$d_{available} = 300 - 40 - 12 - 25/2 = 235.5 > 197.60$ o.k.

$A_s = \dfrac{M}{f_s j d} = \dfrac{22.63 \times 1000^2}{165.6 \times 0.878 \times 235.5}$
$= 660.90\ mm^2$

Area of 2-22φ bars $= 760.27 > 660.90$ o.k.

$d = 300 - 40 - 12 - 22/2 = 237\ mm.$

Shear stress, $v = \dfrac{V}{bd} = \dfrac{58.92 \times 1000}{300 \times 237}$
$= 0.83\ N/mm^2$

Allowable shear stress $= 0.079\sqrt{30}$
$= 0.433\ N/mm^2 < 0.83\ N/mm^2$

stirrups reqd.

Sheet .... of............

Project : ....

Structure : ....

Item : ....

Designed : ....

Date : ....

Design for stirrup

$v - v_c = 0.83 - 0.433 = 0.397\ N/mm^2$

$0.332\sqrt{f_c'} = 1.82\ N/mm^2 > 0.397\ N/mm^2$ O.K.

$0.166\sqrt{f_c'} = 0.91\ N/mm^2 > 0.397\ N/mm^2$

Normal spacing can be maintained

Let us use 10φ bar as stirrup

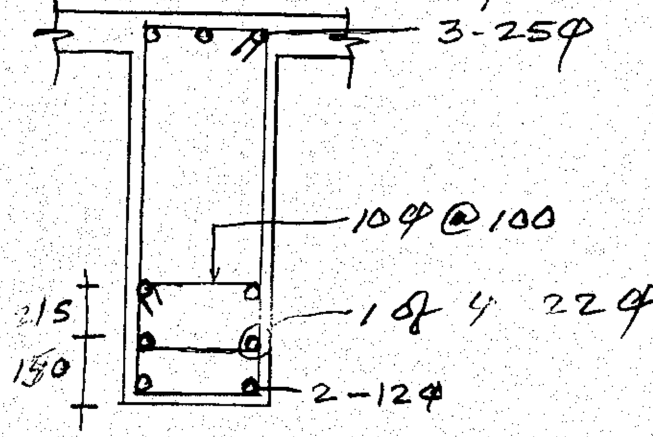
$A_v = 2 \times 78.54 = 157.08\ mm^2$

$\therefore S = \frac{A_v f_y}{(v - v_c) b_w} = \frac{157.08 \times 165.6}{0.397 \times 300} = 218\ mm\ c/c$

$S_{max} = \frac{A_v f_y}{0.34475 b_w} = \frac{157.08 \times 414}{0.34475 \times 300} = 628\ mm$

$S_{max} = \frac{d}{2} = \frac{237}{2} = 118.5\ mm$

$\therefore$ Provide 10φ stirrup 100 mm c/c.



Sheet .... of ....

Project :

Structure :

Item :

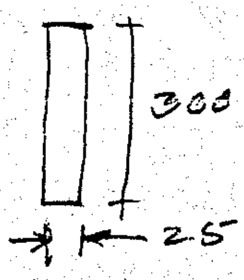
Designed :

Date :

Check For Bearing on Concrete

$h = 300\ mm$

$\phi = 25\ mm$



Bearing Pressure on concrete

$$= \frac{58.92 \times 1000}{300 \times 25}$$

$$= 7.86\ MPa$$

Allowable bearing stress

$$= 0.3 f'_c = 0.3 \times 30 = 9\ MPa > 7.86\ MPa.$$

1-2. DESIGN OF APPROACH BRIDGE

(2) SUBSTRUCTURE

A. Soil Spring

(i) Pier No. 11

Following the article 16.15 of Foundation Analysis and design written by Bowles (5th. edition) Modulus of Subgrade reaction K_s can be given as follows :

$K_s = A_s + B_s Z^n$

where $A_s = F_{w1} C_m C (c N_c + 0.5 \gamma B_p N_\gamma)$ and

$B_s = F_{w2} C_m C (\gamma Nq Z^n)$ again where

$F_{w1} = 1.3$ to 1.7 say 1.5

$F_{w2} = 2.0$ to 4.4 say 3.2

$C_m = 1 + (457 / D)^{0.75} >= 1.5$ where D is pile diameter

$C_m = 1 + (457 / 900)^{0.75} = 1.6$

Assume $C = 40$, $n = 0.6$ and $\gamma = 10$ kn/m^3

Therefore $A_s = 1.5 \times 1.6 \times 40 (c N_c + 0.5 \times 10\ B_p N_\gamma) = 90 (c Nc + 5 Bp N\gamma)$

$B_s = 3.2 \times 1.6 \times 40 \times 10 Nq Z_n = 1920 Nq Z^n$

Among all the boreholes in the viaduct area BH1VP2 is taken as the basis for estimating Spring constant because the top soil of this borehole is the softest. The Soil parameter is assumed as follows :

Top layer (Upto a depth of 11.5 I.e. RL = -8.28)

For avg. SPT = 3, $\phi = 0$, $c = 6.25 \times SPT = 18.75$,

$N_q = 1$, $N_\gamma = 0$, $N_c = 5.14$ (See table 4.4.7.1A odf AASHTO Code.)

Therefore, $Ks = 90 \times 18.75 \times 5.14 \times + 1920 \times 1 \times Z^{0.6} = 8,674 + 1920 Z^{0.6}$

Like main bridge here also, the spring stiffness is reduced by 35% to account for the group action on laterally loaded pile and hence multiplying the above equation by 0.65,

$Ks = 5,650 + 1,250 Z^{0.6}$

Bottom layer (For remaining depth of pile)

Avg. SPT = 13, $\phi = 0$, c = 6.25 x SPT = 81.25, $N_q = 1$, $N_\gamma = 0$, $N_c = 5.14$
Assuming reduction factor of 0.65 for Group action,
$Ks = 0.65\ (90 \times 81.25 \times 5.14 \times + 1920 \times 1 \times Z^{0.6}) = 24,400 + 1,250\ Z^{0.6}$

Depth Z (m)	Ks	Spring Constant KFX	KFZ
0.60	6,570	14,783	4,139
1.35	7,147	9,648	9,648
2.85	7,993	10,791	10,791
4.35	8,670	11,704	11,704
5.85	9,257	12,498	12,498
7.35	9,787	13,212	13,212
8.85	10,275	13,871	13,871

Note :
$Ks = 5,650 + 1250 \times Z^{0.6}$
KFX = Ks x 15.0 x 1.2 / 8 for Z = 0.60m (Pilecap)
KFZ = Ks x 4.2 x 1.2 / 8 for Z = 0.60m (Pilecap)
KFX = KFZ = Ks x 0.9 x 1.5 for Z > 0.60m (Piles)

Depth Z (m)	Ks	KFX / KFZ
10.35	29,480	39,798
11.85	29,910	40,378
13.35	30,318	40,930
14.85	30,709	41,457
16.35	31,084	41,963
17.85	31,445	42,451
19.35	31,795	42,923
20.85	32,133	43,380
22.35	32,463	43,825
23.85	32,783	44,257
25.35	33,096	44,679
26.85	33,401	45,091
28.35	33,699	45,494

$Ks = 24,400 + 1250 \times Z^{0.6}$

(ii) Pier No. 1 to 10

4 times of that of pier no. 11 shall be used as Spring constant with values as follows :

Depth Z (m)	Ks	Spring Constant KFX	KFZ
0.60	6,570	59,130	16,556
1.35	7,147	38,592	38,592
2.85	7,993	43,164	43,164
4.35	8,670	46,818	46,818
5.85	9,257	49,990	49,990
7.35	9,787	52,850	52,850
8.85	10,275	55,483	55,483

Note :
$Ks = 5,650 + 1250 \times Z^{0.6}$

Depth Z (m)	Ks	KFX / KFZ
10.35	29,480	159,193
11.85	29,910	161,513
13.35	30,318	163,719
14.85	30,709	165,827
16.35	31,084	167,852
17.85	31,445	169,804
19.35	31,795	171,691
20.85	32,133	173,521
22.35	32,463	175,298
23.85	32,783	177,029
25.35	33,096	178,716
26.85	33,401	180,364
28.35	33,699	181,975

$Ks = 24,400 + 1250 \times Z^{0.6}$

(iii) Abutment

6 times of that of pier no. 11 shall be used as Spring constant with values as follows :

Depth Z (m)	Ks	Spring Constant KFX	KFZ
0.50	6,475	62,934	16,316
1.25	7,079	57,341	57,341
2.75	7,944	64,343	64,343
4.25	8,628	69,888	69,888
5.75	9,220	74,685	74,685
7.25	9,753	79,000	79,000
8.75	10,243	82,970	82,970

Note :

$Ks = 5,650 + 1250 \times Z^{0.6}$

KFX = Ks x 16.2 x 1.2 / 2 for Z = 0.50m (Pilecap)

KFZ = Ks x 4.2 x 1.2 / 2 for Z = 0.50m (Pilecap)

KFX = KFZ = 6 x Ks x 0.9 x 1.5 for Z > 0.50m (Piles)

Depth Z (m)	Ks	KFX / KFZ
10.25	29,451	238,550
11.75	29,882	242,044
13.25	30,292	245,363
14.75	30,683	248,534
16.25	31,059	251,580
17.75	31,421	254,514
19.25	31,772	257,351
20.75	32,111	260,101
22.25	32,441	262,772
23.75	32,762	265,372
25.25	33,075	267,907
26.75	33,381	270,383
28.25	33,679	272,803

$Ks = 24,400 + 1250 \times Z^{0.6}$

(iv) End pier

For D > 1200mm, $C_m = 1.25$

Assume C = 40, n = 0.6 and $\gamma = 10$ kn/m^3

Therefore $A_s = 1.5 \times 1.25 \times 40\ (c\,N_c + 0.5 \times 10\ B_p N_\gamma) = 75\ (c\,Nc + 5\ Bp\ N\gamma)$

$B_s = 3.2 \times 1.25 \times 40 \times 10\ Nq\ Z_n = 1285\ Nq\ Z^n$

Considering Borehole BH1BA2, the soil parameter is assumed as follows :

Top layer (Upto a depth of 10.5 i.e. RL = -6.25)

For avg. SPT = 3, $\phi = 0$, c = 6.25 x SPT = 18.75,

$N_q = 1$, $N_\gamma = 0$, $N_c = 5.14$ (See table 4.4.7.1A odf AASHTO Code.)

Therefore, $Ks = 0.65\ (75 \times 18.75 \times 5.14 \times + 1285 \times 1 \times Z^{0.6}) = 4,700 + 835\ Z^{0.6}$

Bottom layer (For remaining depth of pile)

Avg. SPT = 20, $\phi = 0$, c = 6.25 x SPT = 125, $N_q = 1$, $N_\gamma = 0$, $N_c = 5.14$
Assuming reduction factor of 0.65 for Group action,
$Ks = 0.65\ (75 \times 125 \times 5.14 + 1285 \times 1 \times Z^{0.6}) = 31{,}300 + 835\ Z^{0.6}$

Depth Z (m)	Ks	Spring Constant KFX	KFZ
0.75	5,403	22,286	22,286
2.25	6,058	24,990	24,990
3.75	6,545	49,091	49,091
5.25	6,958	52,187	52,187
6.75	7,326	54,944	54,944
8.25	7,662	57,464	57,464

Note :
$Ks = 4{,}700 + 835 \times Z^{0.6}$
KFX = Ks x 11.0 x 1.5 / 4 for Z <= 2.25m (Pilecap)
KFZ = Ks x 11.0 x 1.5 / 4 for Z <= 2.25m (Pilecap)
KFX = KFZ = 2 x Ks x 2.5 x 1.5 for Z > 2.25m (Piles)

Depth Z (m)	Ks	KFX / KFZ
9.75	34,574	259,306
11.25	34,868	261,507
12.75	35,146	263,594
14.25	35,411	265,584
15.75	35,666	267,493
17.25	35,911	269,330
18.75	36,147	271,104
20.25	36,376	272,822
21.75	36,599	274,490
23.25	36,815	276,112
24.75	37,026	277,693
26.25	37,231	279,236
27.75	37,433	280,744

$Ks = 31{,}300 + 835 \times Z^{0.6}$

B. DEAD LOAD FROM SUPERSTRUCTURE

(I) On pier

a. Steel railing	= 50 kg/m	=	0.50 kn/m		
b. Precast Parapet	= 0.469 sqm x 23.56	=	11.05 kn/m		
c. Sidewalk Slab	= 0.056 sqm x 23.56	=	1.32 kn/m		
d. Curb	= 0.119 sqm x 23.56	=	2.80 kn/m		
e. Barrier	= 0.336 sqm x 23.56	=	7.92 kn/m		
			23.59 x 2 x 30 =		1415 kn
f. 60mm Premix	= 30 x 13.50 x 0.06 = 24.30 Cum x 23.56			=	573 kn
g. RC Deck	= 30 x 16.20 x 0.20 = 98.10 Cum x 23.56			=	2311 kn
h. Girders	= 7 x 551 kn (See Girder design)			=	3857 kn
I. Diaphragm	= (See Girder design)			=	455 kn
					8611 kn

Dividing equally on each Girder = 8611 / 7 = **1230** kn

11-32

(ii) On Abutment

From deck & Girder	= 7 x 1230 / 2	=	4305 kn
From approach Slab	=15.1 x 2.5 x 0.36 x 23.56	=	320 kn
		Acting at Node 13 =	4625 kn

Moment Mz = 4305 x 0.275 - 320 x 0.45 = 1039 knm acting at Node 13.

Back fill	= 15.1 x 1.425 x 4.0 x 18.85	=	1623 kn
Return wall	= 2 x 1.425 x 4.3 x 0.55 x 23.56	=	159 kn
	= 2 x 3.075 x 1.0 x 0.55 x 23.56	=	80 kn
Railing etc.	= 23.59 x 2 x 4.5	=	212 kn
			2074 kn

UDL acting on member 65 = 2074 / 2.1 = 990 kn/m

C. LIVE LOAD FROM SUPERSTRUCTURE

(I) On Peir

Assuming Lane load and 4 lanes and allowing 8.75 % reduction

a. Live Load	= (30 x 9.4 + 116) x 4 x 0.9125	=	1453 kn
b. Impact	= 1453 * 15.24 / (30+38)	=	326 kn
c. Sidewalk	= 2 x 30 x 1.2 x 2.875	=	207 kn
d. Utilities	= 4 kn/m x 30.0	=	120 kn
			2106 kn

Dividing equally on each Girder = 2106 / 7 = **300** kn

(ii) On Abutment

Vertical load = 7 x 300 / 2 =1050 kn acting at Node 13
Moment Mz = 1050 x 0.275 =290 knm acting at Node 13

D. WIND LOAD ON SUPERSTRUCTURE

Design Wind Velocity, $V_d = 2.5 \times V_0 \times V_{10} / V_B \log_e (Z / Z_0)$

where V_b = Base wind velocity = 161 km/hr

V_0 = Friction Velocity = 13.21 km/hr for open country

V_{10} = Basic Wind Speed = 238 km/hr.

Z_0 = Friction Length = 0.07 for open country

$V_d = 2.5 \times 13.21 \times 238 / 161 (\log Z - \log 0.07) = 130 + 48.82 \log Z$

R.L. on top of deck above the tallest peir = 17.0 - 30 x 0.03 = 16.1 MPWD
R.L. on top of parapet above the tallest peir = 16.1 + 0.82 + 0.36 = 17.28 ,,
R.L. of beam soffit = 16.1 - 2.0 = 14.1 MPWD

Level at C.G. of Wind pressure = (17.28 + 14.1) / 2 = 15.69 MPWD
Normal Ground Level = 1.5 MPWD

Z for wind on superstructure = 15.69 - 1.5 = 14.19 = 14.5 (say)
Z for wind on Live load = 16.1 + 1.83 -1.5 = 16.5 (say)

Vd = 130 + 48.82 log 14.5 = 260 km/hr for Superstr.
Vd = 130 + 48.82 log 16.5 = 267 km/hr for Live load

Wind on Superstr. $= 2.394 \times (260 / 161)^2 = 6.24$ kpa (Transverse)
$0.575 \times (260 / 161)^2 = 1.50$ kpa (Longitudinal)
Wind on Live load $= 1.460 \times (267 / 161)^2 = 4.00$ kn/m (Transverse)
$0.584 \times (267 / 161)^2 = 1.60$ kn/m (Longitudinal)

Area of Superstructure exposed to wind = 30 x 3.2 = 96 sqm

Wind on Superstr. = 96 x 6.24 = 600 kn (Transverse)
96 x 1.50 = 144 kn (Longitudinal)
Wind on Live load = 30 x 4.00 = 120 kn (Transverse)
30 x 1.60 = 48 kn (Longitudinal)

E. LONGITUDINAL FORCES

5% of L.L = 2 x (30 x 9.4 + 116) x 5% = 40 kn

F. SOIL SPRING TO SIMULATE BACKFILL PRESSURE BEHIND ABUTMENT

In case of increase in temperature, superstructure shall expand and push back the abutment wall against the soil creating a situation something in between active and passive state of earth pressure. To simulate the situation, springs are used behind the abutment wall at different levels and spring reactions are calculated by running STAAD. These reactions are then checked against the passive earth pressure at the corresponding level and if the reactions are found more than the passive pressure, the spring stiffness are reduced untill the reactions become equal to passive earth pressure. Initial Spring constants are found as follows :

Assume $F_{w1} = 1$ $C_m = 1$ $c = 0$ $N_q = 18.4$ $B = 1$
$F_{w2} = 1$ $C_m = 40$ $\phi = 30$ $N_\gamma = 18.4$

$A_s = 1 \times 40 \times 0.5 \times 18.85 \times 1 \times 15.1 = 5{,}695$
$B_s = 1 \times 40 \times 18.85 \times 18.4 \times Z^n = 13{,}875\ Z^n$

For n = 0.4, $K_s = 5{,}695 + 13{,}875\ Z^{0.4}$

Depth (m)	Estimated Ks	Spring Constant	Passive Pressure	Maximum Spring Reaction	Spring Constant after adjustment
0.50	16,210	131,303	28	229	112,000
1.00	19,570	158,517	57	458	158,517
1.50	22,013	178,306	85	687	178,306
2.00	24,003	218,729	113	1,030	218,729
2.63	26,107	264,333	148	1,502	264,333
3.25	27,927	282,765	184	1,860	282,765
3.88	29,548	299,173	219	2,218	299,173
4.50	31,018	314,059	254	2,575	314,059

G. THERMAL FORCES

Mean temperature is assumed 26°C while the maximum and minimum temperature are assumed 38°C and 7°C respectively.

H. SHRINKAGE DEFORMATION

Ref : CEB - FIP Manual (Relevant pages enclosed in Appendix A)

Assuming age of girder = 15 days at the time of prestressing and 120 days at the time of casting deck and relative humidity = 70%.

	For Girder	For Deck
X-sectional area 'F'	7000 sqcm	240 x 20 = 4800 sqcm
Perimeter 'u'	519 cm	2 x 240 - 107 = 373 cm
Eff. thickness 'd_w' =2 k_w F / u	40 cm	40 cm

(kw = 1.5 from table 1.2)

Residual Shrinkage Strain 'ε_s' = ε_{so} (K_{st} - K_{sto}) where e_{so} = 25 x 10^{-5} at 70% humidity (See Table 1.2)

	For Girder	For Deck
t	10000 days	10000 days
t_o	120 days	0 days
K_{st} (Table 1.3c)	0.79	0.79
K_{sto} (Table 1.3c)	0.20	-
ε_s	14.75 x 10^{-5}	19.75 x 10^{-5}

Average Shrinkage Strain (weighted average) = (14.75 x 7000 + 19.75 x 4800) / 11800 x 10^{-5}
= 16.8 x 10^{-5}

Shrinkage deformation 'Δ' = 16.8 x 10^{-5} x 360 x 10^3 = 60.4 mm

I. CREEP DEFORMATION

Cross sectional area of concrete Girder 'A_g' = 4.90 sqm
deck 'A_d' = 3.27 sqm
Total 'A' = 8.17 sqm

Total prestress 'P' = 4766 x 7 = 33,362 kn (after all loss)
Elastic Modulus 'E' = 27.98 x 10^3 mpa

Creep Coefficient 'φ' = $\varphi_{vo}\, k_{v(t-to)} + \varphi_{fo}\,(k_{ft} - k_{fto})$ where,

φ_{vo} = 0.4 φ_{fo} = 2.0 (from table 1.2)
k_v = 1 k_{ft} = 1.40 (for 10,000 days)
and k_{fto} = 0.7

Therefore φ = 0.4 x 1.0 + 2.0 (1.40 - 0.7) = 1.80

At service condition f_o = P / A = 33,362 / 8.17 = 4.08 mpa
At construction time f_t = P / Ag = 33,362 / 4.9 = 6.80 mpa

For Girder $= f_t - (f_t - t_o)(1 - e^{-\varphi})$

$= 6.8 - (6.8 - 4.08)(1 - e^{-1.8}) = 4.52$ mpa

For deck $= 4.08 (1 - e^{-1.8}) = 3.41$ mpa

Average prestress 'f' $= (f_g A_g + f_d A_d) / (A_g + A_d)$

$= (4.52 \times 4.9 + 3.41 \times 3.27) / 8.17 = 4.07$ mpa

Therefore 'P' = fA $= 4.07 \times 8.17 / 7 = 4570$ kn

Strain 'ε' $= 4.07 / 27.98 \times 10^3 = 14.5 \times 10^{-5}$

Creep Deformation 'Δ' $= 14.5 \times 10^{-5} \times 360 \times 10^3 = 52$mm

J. EARTH PRESSURE BEHIND ABUTMENT

Node	depth	Pressure
36	0.000	0.0
37	0.500	47.4
38	1.000	94.9
39	1.500	142.3
13	2.000	189.8
40	2.625	249.1
41	3.250	308.4
42	3.875	367.7
35	4.500	427.0

Note : Pressure = 18.85 x depth x 15.1 / 3

The viaduct is modeled in STAAD III (file name is Viaduct1.Std) as space frame to carry the loads described in section B to J. Piles are kept supported horizontally by providing the spring as described in section A. Output of this model is given in Appendix 'B'.

Model Viaduct1.Std considers the effect of thermal contraction accompanied with creep, Shrinkage and the backfill pressure behind the abutment. Another model Viaduct2.Std is created to study effect of thermal expansion on the structure and the output is given in appendix 'C'.

K. DESIGN OF PIER HEAD

From Page 11 of Appendix 'B' =
Max. (-)ive moment = -2094.65 knm for member 14 & load 7
Max. (+) ive moment = 1215.59 knm for member 16/17 & load 7
Max . Shear = 2843.43 kn for member 15 & load 7

Design of Beam (USD method)

		Design Moment =	2095 knm
f'c =	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1650 mm	ROWMIN =	0.0034
Depth =	900 mm		
Bar dia =	28 mm	d =	805 mm
Str. Dia. =	16 mm	As =	7389 sqmm
Cl. Cover =	65 mm	p =	0.0056
Ast =	12 Nos	a =	72.0 mm

Resisting Moment = 2,096.7 knm O.K.

		Design Moment =	1216 knm
f'c =	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1650 mm	ROWMIN =	0.0034
Depth =	900 mm		
Bar dia =	28 mm	d =	805 mm
Str. Dia. =	16 mm	As =	4926 sqmm
Cl. Cover =	65 mm	p =	0.0037
Ast =	8 Nos	a =	48.0 mm

Resisting Moment = 1,419.6 knm > O.K.

f'c =	30 mpa	Design Shear =	2844 kn
fy =	410 mpa	vc =	0.909 mpa
b =	1650 mm	Vc =	1,207.7 kn
d =	805 mm	Vs =	2,275.2 kn
Str. dia =	16 mm		
Spacing =	175 mm	Vu =	2,960.5 kn O.K.
No. of legs =	6 Nos		

L. DESIGN OF COLUMN

PIER NO. 6 TO 11

For forces see Page 12 & 13 and for design see Page 24 of appendix 'B'

PIER NO. 1 TO 5

From Page 25 of appendix B, with 1% reinforcement, column can take as much as 5627 knm (0.9 x 6252.53) of moment, even if there is no axial forces. This is sufficient for column of pier no. 1 to 5. (see forces for column in page 13 of appendix 'B')

M. DESIGN OF PILECAP

Short direction :

From Page 14 of Appendix B and for members 51-54,

Design Shear = (5799.73+5938.55+5918.46+5730.01)/15 = 1559 knm/m

Design Moment = 1559 x (1.35 - 1.5 / 2) = 935 knm/m at the face of column

Design of Beam (USD method)

		Design Moment =	935 knm
f'c =	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1000 mm	ROWMIN =	0.0034
Depth =	1200 mm		
Bar dia =	25 mm	d =	1038 mm
Str. Dia. =	0 mm	As =	3927 sqmm
Cl. Cover =	150 mm	p =	0.0038
Ast =	8 Nos	a =	63.1 mm

Resisting Moment = 1,457.7 knm O.K.

Beam shear is not critical because, pile lies entirely within critical area. However corner pile is critical for punching shear having a force of 5730.01 kn (See member 480, load 10 at page 22 of appendix 'B').

Shear area = 2 x (1036/2 + 800 + 800) x 1036 =4,388,496 sqmm

Shear stress = 5730.01 x 1000 / 4,388,496 = 1.305 mpa

Allowable Shear = 0.332 x $f'c^{0.5}$ = 1.54 mpa > 1.305 O. K.

Long direction :

From Page 16 of Appendix B and for members 76 & load 8,
Design Moment = 1622.01 /4.2 =386 knm/m
Design Shear = negligible

Design of Beam (USD method)

		Design Moment =	386 knm
f'c =	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1000 mm	ROWMIN =	0.0034
Depth =	1200 mm		
Bar dia =	25 mm	d =	1038 mm
Str. Dia. =	0 mm	As =	1963 sqmm
Cl. Cover =	150 mm	p =	0.0019
Ast =	4 Nos	a =	31.6 mm

Resisting Moment = 740.3 knm O.K.

N. DESIGN OF STEM OF ABUTMENT

From Page 17 of Appendix B and for members 73,
Design Moment = 24151.42 /16.2 = 1491 knm/m for load 12
Design Shear = 9679.87 / 16.2 = 598 kn /m for load 11

Design of Beam (USD method)

		Design Moment =	1491 knm
f'c =	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1000 mm	ROWMIN =	0.0034
Depth =	1350 mm		
Bar dia =	25 mm	d =	1273 mm
Str. Dia. =	0 mm	As =	3274 sqmm
Cl. Cover =	65 mm	p =	0.0026
Ast =	6.67 Nos	a =	52.6 mm

Resisting Moment = 1,505.6 knm O.K.

f'c =	30 mpa	Design Shear =	598 kn
fy =	410 mpa	vc =	0.909 mpa
b =	1000 mm	Vc =	983.4 kn O.K.
d =	1273 mm		

From Page 10 of Appendix C and for members 73 & load 10,
Design Moment = 1967.01 / 16.2 = 122 knm/m in water face
Use nominal reinforcement.

O. DESIGN OF PILECAP OF ABUTMENT

From Page 19 of Appendix B and for members 49 & load 12,
Design Moment = 18210.41 x (1.35 - 0.75) /16.2 = 675 knm/m at wall face
Design Shear = 18210 / 16.2 = 1124 kn /m

Design of Beam (USD method)

			Design Moment =	675	knm
f'c =	30	mpa	Beta1 =	0.8324	
fy =	410	mpa	ROWMX =	0.0231	
b =	1000	mm	ROWMIN =	0.0034	
Depth =	1000	mm			
Bar dia =	25	mm	d =	838	mm
Str. Dia. =	0	mm	As =	3274	sqmm
Cl. Cover =	150	mm	p =	0.0039	
Ast =	6.67	Nos	a =	52.6	mm

Resisting Moment = 980.0 knm O.K.

Beam shear is not critical because, pile lies entirely within critical area.
However corner pile is critical for punching shear having a force of 3035 kn
I.e. 18210.39 / 6 (See member 380, load 12 at page 22 of appendix 'B').

Shear area = 2 x (836/2 + 800 + 700) x 836 =3,206,896 sqmm
Shear stress = 3035 x 1000 / 3,206,896 = 0.946 mpa
Allowable Shear = 0.332 x $f'c^{0.5}$ = 1.54 mpa > 0.946 O. K.

P. DESIGN OF RETURN WALL

Depth	Length	UDL	Moment	Mom/m
0.25	4.50	1.57	15.90	53.60
0.75	4.00	4.71	37.70	73.04
1.25	3.00	7.85	35.34	57.34
1.75	2.00	11.00	21.99	37.90
2.25	1.50	14.14	15.90	35.34
2.75	1.50	17.28	19.44	42.41
3.25	1.50	20.42	22.97	49.48
3.75	1.50	23.56	26.51	

Design Moment = 73.04 x 1.3 = 95 knm/m

Design of Beam (USD method)

			Design Moment =	95	knm
f'c =	30	mpa	Beta1 =	0.8324	
fy =	410	mpa	ROWMX =	0.0231	
b =	1000	mm	ROWMIN =	0.0034	
Depth =	550	mm			
Bar dia =	16	mm	d =	477	mm
Str. Dia. =	0	mm	As =	1146	sqmm
Cl. Cover =	65	mm	p =	0.0024	
Ast =	5.70	Nos	a =	18.4	mm

Resisting Moment = 197.8 knm O.K.

Q. DESIGN OF PILES

Structural :

See appendix 'D'

Geotechnical :

Please refer page no. 9 & 10 of appendix B for pile reaction and appendix E for pile capacity. Please note further that reaction of piles under abutment shall have to be divided by 6

	West abut.	WP1-WP11	EP1-EP7	EP8-EP11	East abut.
Reference bore hole :	BH1 VA1	BH1 VP1	BH1 EA1	BH1 VP2	BH1 EA1
Pile reaction for load 13 (kn) :	820	2206	2206	2206	820
Pile reaction for load 14 (kn) :	2522	4801	4801	4801	2522
Uplift for load 14 (kn) :	1118	1306	1306	1306	1118
Ultimate capacity of pile (kn) :	5805	8621	9542	8888	9542
Depth of pile tip from G.L. (m) :	30.00	48.00	58.50	46.50	58.50
Top of bore hole R.L. (MPWD) :	2.03	2.16	3.31	3.22	3.31
Pile tip R. L. (MPWD) :	-27.97	-45.84	-55.19	-43.28	-55.19
Length of pile (m) :	29.81	46.34	55.69	43.78	55.69
Weight of pile (kn) :	447.09	695.10	835.35	656.70	835.35
Net uplift deducting pile weight :	670.91	610.90	470.65	649.30	282.65
Skin friction (kn) :	3780.00	7184.00	7320.00	6559.00	7320.00
FS against load 13 :	7.08	3.91	4.33	4.03	11.64
FS against load 14 :	2.30	1.80	1.99	1.85	3.78
FS against uplift :	5.63	11.76	15.55	10.10	25.90

Minimum bearing capacity : Abutment = 5805 / 2.5 = 2322 kn
Pier = 8621 / 2.5 = 3448 kn