GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF LOCAL GOVERNMENT, RURAL DEVELOPMENT & CO-OPERATIVES

LOCAL GOVERNMENT ENGINEERING DEPARTMENT

MANUAL ON PRESTRESSED CONCRETE BRIDGES
PART C - DESIGN EXAMPLES

PREPARED BY:

DESIGN PLANNING & MANAGEMENT CONSULTANTS LTD.

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Manual on PC Bridges

Part C - Design Examples

CONTENTS

CHAPTER 1

INTRODUCTION

The Part C of the Manual on Prestressed Concrete Bridges, 1996 has additionally been developed to provide an in-depth calculation procedure for analysing and designing a PC bridge. The design procedure and Standard Code practice followed in designing different components of a PC bridge are the same as those mentioned in the relevant flow charts and Code requirement in the Part B of this Manual.

The reference article numbers in Part B of this Manual containing the flow charts for the step-by-step design procedure are as follows:

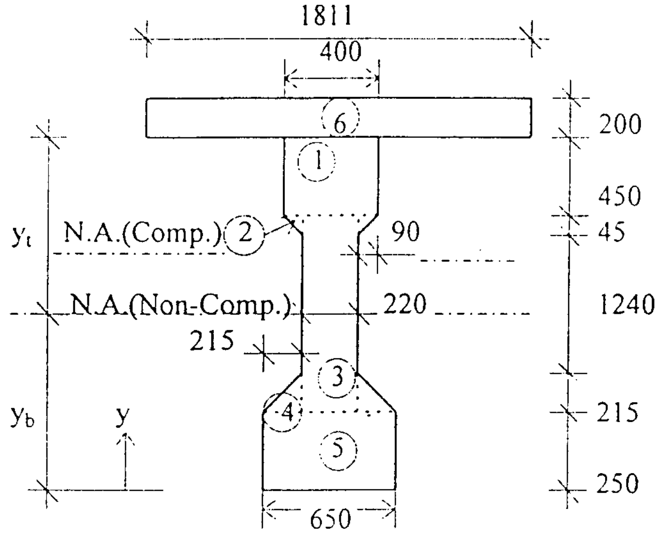
-	PC Girder	:	Art. 8.5
-	Deck Slab	:	Art. 9.4
-	Abutment-Wing Wall	:	Art. 10.4
-	Pile Cap	:	Art. 11.3
-	Cast-in-Situ Bored Pile	:	Art. 11.2

A typical design example comprising 30.0 m span length, 8.0 m abutment height with foundation Type-F and deck Type-IIA has been worked out manually in different Chapters of this part of the Manual. However, the effective and efficient tools for designing a PC bridge are the computer software and the User Guide included in Chapter 17, Part B of this Manual. The results of the hand calculated design example will be the same as that of the computer operated design software output for the same design parameters except some round-up errors.

CHAPTER 2

SUPERSTRUCTURE

2.1 Structural Design of PC Girder



Non Composite Section :

y_b = 1006.83 mm = 1.00683 m
y_t = 2200 - 1006.83 = 1193.17 mm ≅ 1.19317 m

$\Sigma I = \Sigma \bar{I} + \Sigma A \bar{y}^2$ = 3.848×10^{11} mm^4 = 0.3848 m^4

Section Modules, Z_b = 0.3848 m^4/1.00683 m = 0.3822 m^3

Z_t = 0.3848 m^4/1.19317 m = 0.3225 m^3

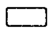
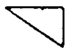
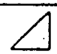
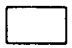
Composite Section :

Transformed effective flange width = $(2.218 \times 21152 \times 10^3)/(25906 \times 10^3)$ = 1.8109 m

Distance of N.A. from girder bottom, $Y_{bc} = (1.5608 \times 10^9)/(1.085 \times 10^6 \times 10^3)$ = 1.438 m

Distance of N.A. from deck top, Y_{ts} = 2.2+0.2-1.438 = 0.962 m

Distance of N.A. from girder top, Y_{tc} = 2.2-1.438 = 0.762 m

Comp.	Shape	b (mm)	d (mm)	A (mm^2)	y (mm)	Ay (mm^3)	$\bar{y}_{NA}$ (mm)	$\bar{I}$ (mm^4)	$\bar{y}$ (mm)	A $\bar{y}^2$ (mm^4)
1.		400	450	$180x10^3$	1975	$355.5x10^6$		$3.0375x10^9$	968.17	$1.687x10^{11}$
2.		90	45	$2x2.025x10^3$	1735	$7.026x10^6$		$x227.81x10^3$	728.17	$2.147x10^9$
3.		220	1500	$330x10^3$	1000	$330x10^6$	1006.83	$6.1875x10^{10}$	6.83	$15.394x10^6$
4.		215	215	$2x23.113x10^3$	321.67	$14.869x10^6$		$x59.354x10^6$	685.16	$2.17x10^{10}$
5.		650	250	$162.5x10^3$	125	$20.313x10^6$		$846.354x10^6$	881.83	$1.2636x10^{11}$
Non-composite ΣA =				$722.775x10^3$	ΣAY=	$727.705x10^6$	$\Sigma \bar{I}$ =	$6.588x10^{10}$	$\Sigma A \bar{y}^2$ =	$3.1892x10^{11}$
6.		1811	200	$362.2x10^3$	2300	$833.06x10^6$		$1.2073x10^9$		
Composite ΣA=				$1.085X10^6$	ΣAY =	$1.5608X10^9$	$\Sigma \bar{I}$ =	$6.709X10^{10}$		

Moment of Inertia of Composite Section

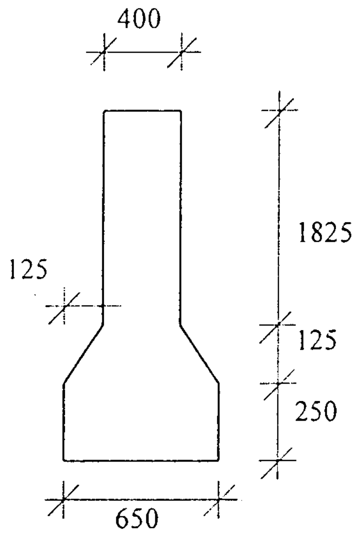
$$
\begin{aligned}
I_{comp} &= \sum \bar{I} + \sum A\ \bar{y}^2 \\
&= 6.709\text{x}10^{10} + (180\text{x}10^3\text{x}537^2 + 4.05\text{x}10^3\text{x}297^2 + 330\text{x}10^3\text{x}438^2 + \\
&\quad 46.225\text{x}10^3\text{x}1116.33^2 + 162.5\text{x}10^3\text{x}1313^2 + 362.2\text{x}10^3\text{x}862^2) \\
&= 7.895\text{x}10^{11}\,mm^4 \\
&= 0.7895\ m^4
\end{aligned}
$$

Section modulus at deck top, Z_{ts} = 0.7896/0.962 = 0.821 m^3

Section modulus at girder top, Z_{tc} = 0.7896/0.762 = 1.0362 m^3

Section modulus at girder bottom , Z_{bc} = 0.7896/1.438 = 0.549 m^3

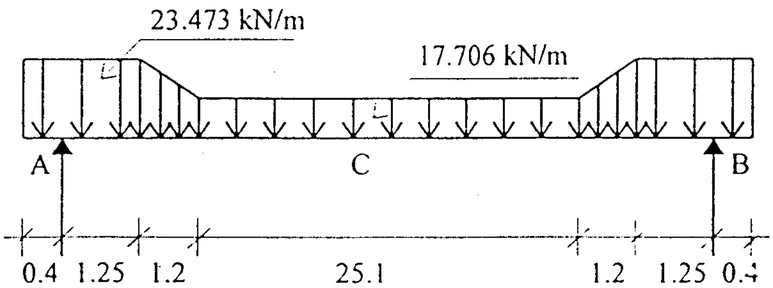
End Section of PC Girder



X-sectional area = 0.4 x 1.95 + 0.125 x 0.125 + 0.25 x 0.65
= 0.9581m^2

Dead Load Moments

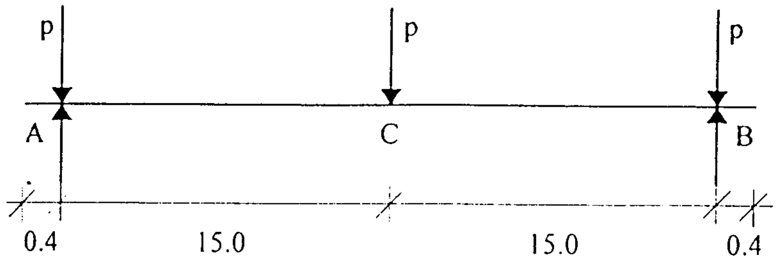
a) **Moment due to Self Weight of Girder**



R_A = 38.73 +24.707+222.21 = 285.647 kN

$M_{c,g}$ = 285.647 x 15 - 38.73 x 14.575 - 243.46 x 6.875 - 3.46 x 13.35 = 2000.24 kN-m

b) Moment due to Cross Girder



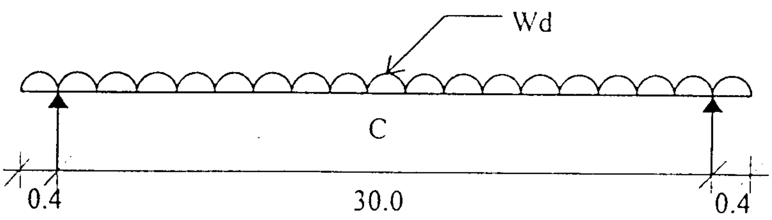
Distance between girders = 3.17 m c/c

Load from cross girder

p = (((3.17 - 0.22)/2) x 1.95 - 0.45 x 0.09 - 0.5 x 0.09 x 0.045 - 0.5 x 0.215 x 0.215) x 0.25 x 24
= 16.864 kN

$M_{c,xg}$ = 16.864 x 30/4 = 126.478 kN-m

c) Moment due to Deck Slab



Wd = (3.17/2 + 0.58 + 0.388) x 0.2 x 24 = 12.252 kN/m

(M_c) deck = 12.252 x 30 ^{2}/8 = 1378.35 kN-m

d) Moment due to WC, Side Walk and Railing

Load from railing	= 0.734 kN/m
Load from side walk	= 6.06 kN/m
Load from WC	
(3.17/2 + 0.58) x 0.04 x 23.0	= 1.992 kN/m
Total, w	= 8.786 kN/m

$(M_c)_{WC+SW+R}$ = (8.786 X 30^2)/8
= 988.425 kN-m

Live Load Moment

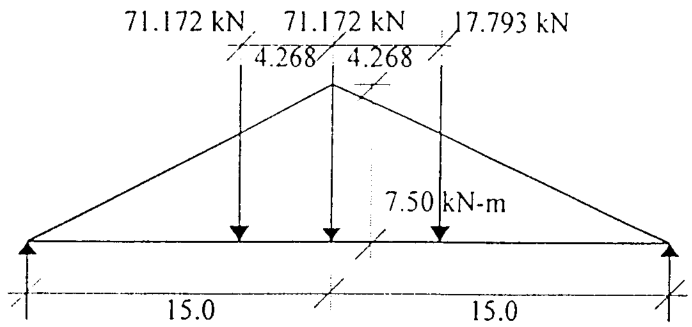
a) Moment due to Foot Path Live Load

$(M_c)_{FPLL}$ = (2.88 x 0.4) x 30^2/8
= 129.60 kN-m

b) Moment due to LL

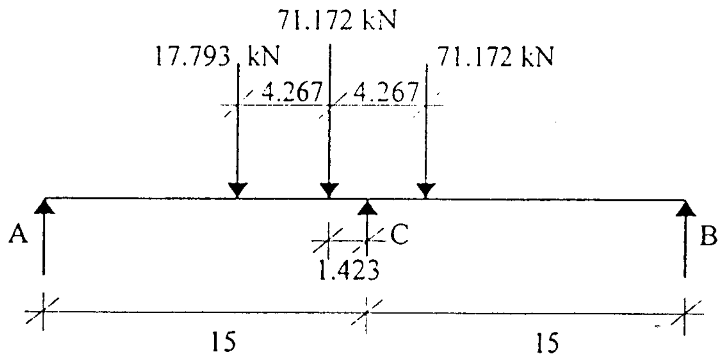
Fraction of wheel load applicable on girder,
$f = (3.17 - (0.61 - 0.58))/3.17$
$= 0.991$

i) Maximum moment due to HS20 wheel load



$M_c = (7.5/15) \times (71.172 \times 15 + 71.172 \times 10.732 + 17.793 \times 10.732) \times 0.991$
$= 1002.076$ kN-m

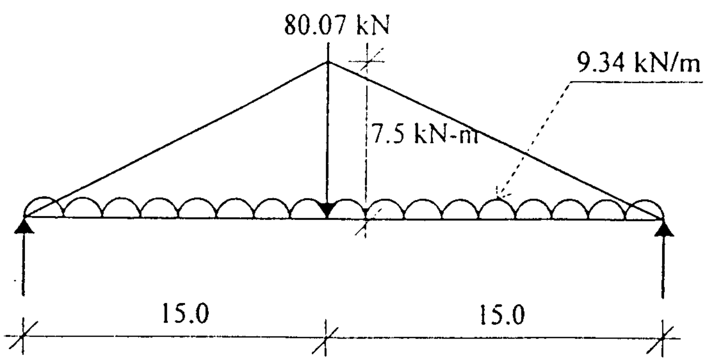
ii) Absolute maximum moment



$R_A \times 30 - 17.793 \times 20.69 - 71.172 \times 16.423 - 71.172 \times 12.156 = 0$
$\therefore R_A = 80.072$ kN

$M_c = (80.072 \times 15 - 17.793 \times 5.69 - 71.172 \times 1.423) \times 0.991$
$= 989.573$ kN-m

iii) Maximum moment due to equivalent lane load



$M_c = 0.5 \times (0.5 \times 7.5 \times 30 \times 9.34 + 7.5 \times 80.07)$
$= 825.637$ kN-m

$\therefore$ Maximum live load moment = 1002.076 kN-m

$$\text{Impact fraction, } I = \frac{15.24}{L + 38} \ngtr 0.30$$

$$= \frac{15.24}{30 + 38} = 0.224$$

Maximum LL moment with impact = 1002.076 x 1.224
= 1226.541 kN-m

Loss of Prestress

a) Calculation of Losses due Friction and Wedge Pull-in

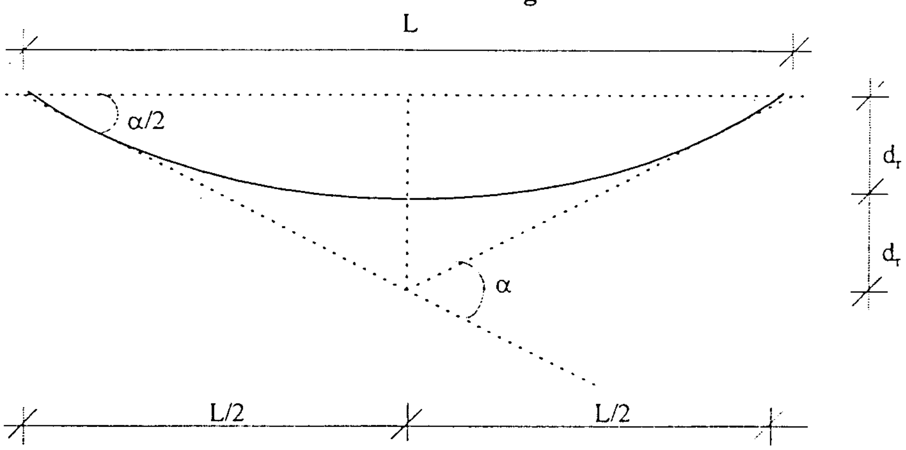


Fig. Typical Cable Profile

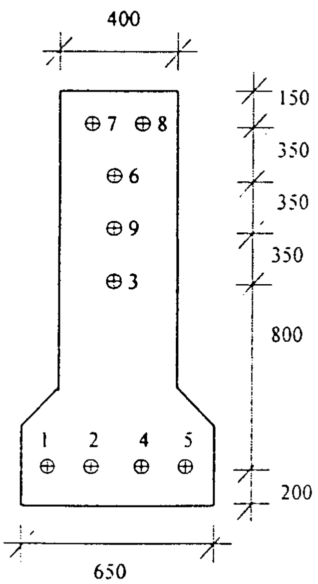


Fig. End Section of PC Girder

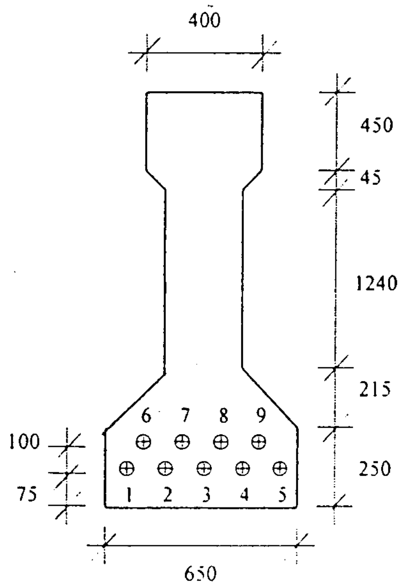


Fig. Mid Section of PC Girder

Cable sag d_r for cable #1, $(d_r)_1$ = 200 - 75 = 125

Cable sag d_r for cable #s 2,4&5
$(d_r)_2$, $(d_r)_4$ & $(d_r)_5$ = 200 - 75 = 125
Cable sag d_r for cable #3, $(d_r)_3$ = 1000 - 75 = 925

Cable sag d_r for cable #6, $(d_r)_6$ = 1700 - 175 = 1525

Cable sag d_r for cable #s 7&8
$(d_r)_7$ & $(d_r)_8$ = 2050 - 175 = 1875

Cable sag d_r for cable #9, $(d_r)_9$ = 1350 - 175 = 1175

The following equations are used for calculating loss of prestress due to friction and wedge pull-in.

i) Equation for calculating radius of curvature of each cable

$$r = L^2/(8d_r)$$

ii) Equation for calculating cable force at a distance x from jacking end

$$T_x = T_1 \, EXP(-(\mu x/r + Kx))$$

iii) Loss of prestress force due to friction per unit length

$$p = T_1 \times (1 - EXP(-(\mu/r + K))$$

iv) Length of cable subjected to prestress loss due to wedge pull in

$$X_A = ((\Delta_{wp} E_{ps} A_{ps})/p)^{1/2}$$

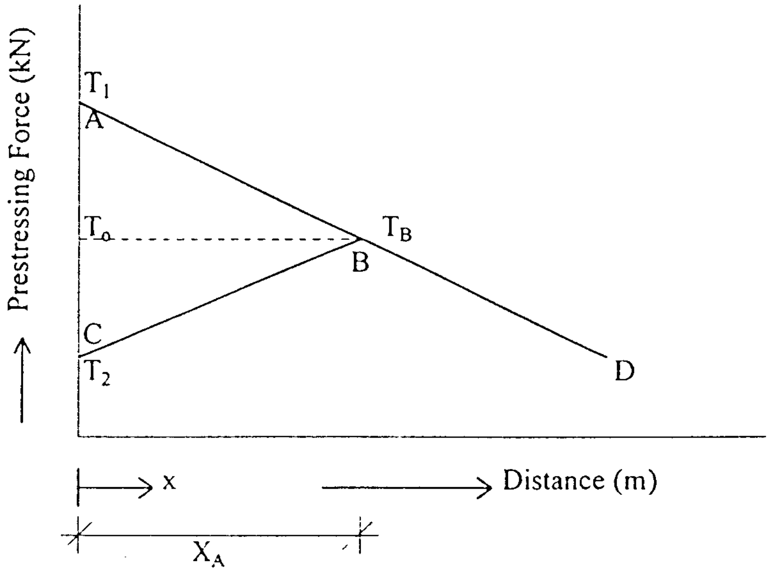


Fig. Loss of Prestress due to Wedge Pull-in

Note: When distance x is less than X_A the magnitude of prestress force after friction and wedge pull-in losses will be the ordinate of the line CB and when x is greater than X_A prestress force in the cable will be the ordinate of line BD.

Data: $\mu = 0.3$ $K = 0.007$ $\Delta_{wp} = 0.008$ m
$E_{ps} = 195 \times 10^6$ kN/m^2 $A_{ps} = 462 \times 10^{-6}$ m^2 L = 30.0 m

Cable No.	Cable Sag, dr m	Radius of Curvature r m	Initial prestress Force, T1 kN	Loss of Prestress per unit Length, p kN/m	Distance of wedge pull-in loss, X_A m	Cable force at X_A, T_B=T_0 kN	Cable force at x = 15 m kN	Loss of prestress %
1	0.125	900.00	555.00	4.055	13.332	500.94	497.187	10.416
2	0.125	900.00	555.00	4.055	13.332	500.94	497.187	10.416
3	0.925	121.62	555.00	5.229	11.740	493.61	481.529	13.238
4	0.125	900.00	555.00	4.055	13.332	500.94	497.187	10.416
5	0.125	900.00	555.00	4.055	13.332	500.94	497.187	10.416
6	1.525	73.77	555.00	6.108	10.863	488.65	470.110	15.295
7	1.875	60.00	555.00	6.620	10.434	485.93	463.575	16.473
8	1.875	60.00	555.00	6.620	10.434	485.93	463.575	16.473
9	1.175	95.74	555.00	5.596	11.348	491.49	476.737	14.101
						$\Sigma T_0 =$ 4449.37 kN		Σ 117.244

Average loss of prestress = 117.244 / 9
= 13.027 %

b) Calculation of Loss of Prestress due to Elastic Shortening

X- sectional area of girder at mid span = 0.7228 m^2
Moment of inertia of girder section at mid span = 0.3848 m^4
Distance of C.G. of 9 cables from neutral axis = 1.0068 - 0.11944 = 0.8874 m

Stress at C.G. of cables due to prestressing force and self weight of girder,

f_{cir} = 555x9/0.7228 + 555x9x0.8874^2 / 0.3848 - 2000.24x0.8874 / 0.3848
= 6910.63 + 10222.08 - 4612.82
= 12519.89 kN/m^2

Elastic shortening loss, ES = 0.5 x Es/Ec x f_{cir}
= 0.5 x (195x10^6) / (21.9x10^6) x 12519.89
= 55739.236 kN/m^2

So, elastic shortening loss = 55739.236x100 / (555/0.000462)
= 4.64 %

c) Creep Loss

As per AASHTO '92, loss of prestress due to creep, CR = 12 f_{cir} - 7 f_{cds}

where, f_{cds} = stress at the C.G. of cables due to all dead loads except those present during prestressing.

Total moment due to cross girder, deck slab and superimposed (WC+SW+Railing) dead loads
= 126.478+1378.35+988.425
= 2493.253 kN-m

Moment of inertia of the composite section, I_{comp} = 0.7895 m^4

Distance of C.G. of cables from N.A. = 1.4385 - 0.11944
= 1.3191 m

Effective prestress force = T1(1- Friction loss - Elastic shortening loss)
= 555 (1-0.13027-0.0464)
= 456.95 kN

Stress at C.G. of cables due to prestressing force and self weight of PC girder,
f_{cir} = 9x456.95/0.7228 + 9x456.95x0.8874^2 /0.3848 - 2000.24x0.8874/0.3848
= 5689.75 + 8416.18 - 4612.82
= 9493.11 kN/m^2

Stress at C.G. of cables due to dead loads except those present at the time of prestressing,
f_{cds} = 2493.253 x 1.3191/0.7895
= 4165.738 kN/m^2

Creep loss = 12x9493.11 - 7x4165.738
= 84757.155 kN/m^2

Effective prestress loss in % = 0.7x84757.155/(555/0.000462) x (494.374/555)x100
= 4.399 %

d) Shrinkage Loss

As per AASHTO '92

Shrinkage loss, SH = 0.8 (17000 -150RH) psi
= 0.8 (17000 - 150RH)/145 N/mm^2 ,
where, RH = Relative humidity in % = 60 %

Shrinkage loss = 0.8 (17000 - 150x60)/145
= 44.138 N/mm^2

So, Shrinkage loss in % = 44.138/(555x1000/462) x (494.374/555) x 100
= 3.273 %

e) Relaxation Loss (100 hrs.)

Loss of prestress due to relaxation of prestressing steel in 100 hrs. = 3.5 %

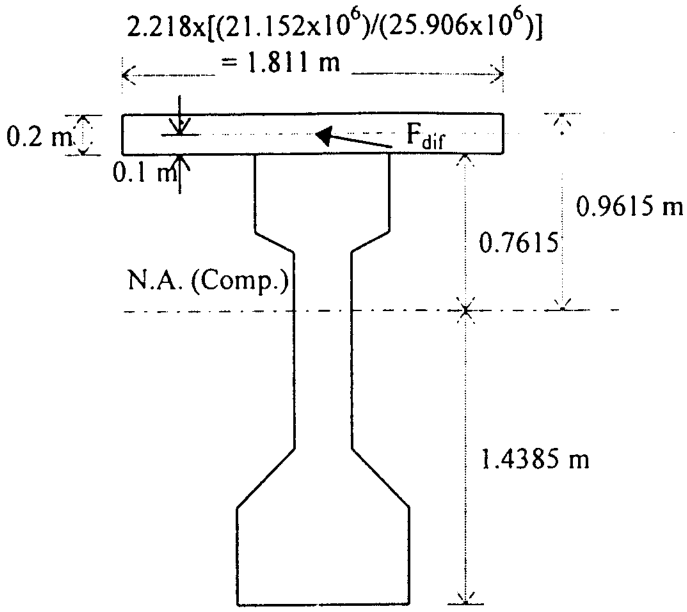
Average prestress force in the cables after wedge pull-in loss, $T_{o(avg.)}$ = 4449.37/9
= 494.374 kN

Effective prestress loss = (494.374/555) x 3.5 %
= 3.1177 %

f) Final Relaxation Loss (1000 hrs.)

Loss of prestress due to relaxation of prestressing steel in 1000 hrs. = 5 - 3.1177
= 1.8823 %

g) Loss of Prestress due to Creep Modified Differential Shrinkage



Assuming that time difference between deck slab and PC girder concreting will be 60 days.

So, creep modified differential shrinkage, A_{sh} = 100×10^{-6}

Force due to creep modified differential shrinkage, F_{dif} = $100\times10^{-6}\times25.906\times10^{6}\times1.811\times0.2$
= 938.315 kN

Stress at deck top = [938.315/1.085 + 938.315x0.8615x0.9615/0.7895]x1/1000
-[$100\times10^{-6}\times25.906\times10^{6}$]/1000
= -0.7416 N/mm^2

Stress at girder top = [938.315/1.085 + 938.315x0.8615x0.7615/0.7895]x1/1000
= 1.644 N/mm^2

Stress at girder bottom = 938.315/1.085 - 938.315x0.8615x1.4385/0.7895
= -0.608 N/mm^2

Schedule of Stresses

Section properties of non-composite section :

Gross area, ANC = 0.7228 m^2
Section modulus at top, Z_t = 0.3225 m^3
Section modulus at bottom, Z_b = 0.3822 m^3

Section properties of composite section :

Section modulus of deck top, Z_{TS} = 0.8212 m^3
Section modulus at girder top, Z_{TC} = 1.0362 m^3
Section modulus at girder bottom, Z_{BC} = 0.5489 m^3

SL. No.	Description of Items	Axial Force (kN)	Bending Moment (kN-m)	Stress at Girder bottom (N/mm^2)	Stress at Girder top (N/mm^2)	Stress at Deck top (N/mm^2)
01.	Moment due to Self Weight of Girder		2000.24	-5.233	+6.202	
02.	a) Axial Force due to Prestressing (555x9)	4995.00		+6.9106	+6.9106	
	b) Moment due to Prestressing (555x9x0.8874)		4432.563	+11.597	-13.744	
03.	Loss of Prestress due to Friction = 13.027%			-2.4109	+0.8902	
04.	Loss of Prestress due to Elastic Shortening(4.64%)			-0.8587	+0.317	
Sub-Total 1: Allowable Compression, $0.55f'_{ci} = 13.2\ N/mm^2$ Allowable Tension, $0.498\sqrt{f'_{ci}} = 2.439\ N/mm^2$				+10.005 < 13.2 OK	+0.5758 < 13.2 OK	
05.	Loss of Prestress due to Relaxation loss 3.1177%			-0.577	+0.213	
06.	Loss of Prestress due to Shrinkage 3.273%			-0.6057	+0.2236	
Sub-Total 2:				+8.822 < 13.2 OK	+1.0124 < 13.2 OK	
07.	Moment due to X-Girder		126.478	-0.331	+0.392	
08.	Moment due to Deck		1378.35	-3.606	+4.274	
09.	Moment due to WC, SW and Railing		988.425	-1.800	+0.954	+1.204
10.	Moment due to LL and FPLL		1356.141	-2.471	+1.309	+1.651
11.	Final Relaxation Loss (1.8823%)			-0.348	+0.128	
12.	Stress due to Creep modified diff. Shrinkage			-0.608	+1.644	-0.7416
13.	Creep Loss 4.399%			-0.814	+0.300	
Total Stress at Service Load Condition: Allowable Compression, $0.4f'_c = 12\ N/mm^2$ Allowable Tension, $0.249\sqrt{f'_c} = 1.36\ N/mm^2$				-1.156 < 1.36	+10.01 < 12	+2.1134 < 12

(Note: -ve means tension & +ve means compression)

Check for Ultimate Moment Capacity of PC Girder

i) Factored external moment:

Total dead load moment, TDLM = 2000.24 + 126.478 + 1378.35 + 988.425
= 4493.493 kN-m

Max. moment for LL, MLL = 1002.076 kN-m
Moment for FPLL, MFPLL = 129.60 kN-m

Factored moment for DL and LL :

MU_1 = 1.30 x (1.0 x TDLM + 1.25 x (MLL x (1+IMPACT) + MFPLL))
= 1.30 x (1.0 x 4493.493 + 1.25 x (1002.076 x (1+0.2241)+129.60))
= 8045.433 kN-m

MU_2 = 1.30 x (1.0 x TDLM + 1.67 x (MLL + MFPLL))
= 1.30 x (1.0 x 4493.493 + 1.67 x (1002.076 +129.60))
= 8298.41 kN-m

∴ Max Factored moment, MU = 8298.41 kN-m

ii) Design Strength (As per AASHTO'92)

$f^*_{SU} = f'_S[1 - (r^*/\beta_1)(\rho^* f'_S/f'_c)]$
$= f'_S \times k$

For stress-relieved prestressing steel and concrete strength
$f'c = 30\ N/mm^2$, value of $k \cong 0.9$

∴ $f^*_{SU} = 0.9 \times f'_s = 0.9 \times 1620000 = 1458000\ kN/m^2$

Depth of stress block, $a = \dfrac{A_{PS} \times f^*_{SU}}{0.85\ f'_c b}$

$$\therefore a = \frac{9 \times 0.000462 \times 1458000}{0.85 \times 30000 \times 1.8109}$$

= 0.131 m < 0.20 m (Slab thickness)

So, Design strength ϕMn is calculated as

$\phi Mn = \phi\ [A^*_S f^*_{SU} d\ (1-0.6 \times p^* f^*_{SU}/f'_c)]$

where, 'd = effective depth from girder top to c.g. of prestressing steel
= 2.0805 m

$$p^* = \text{ratio of prestressing steel} = \frac{9 \times 0.000462}{1.8109 \times 2.0805}$$

$$= 1.1036 \times 10^{-3}$$

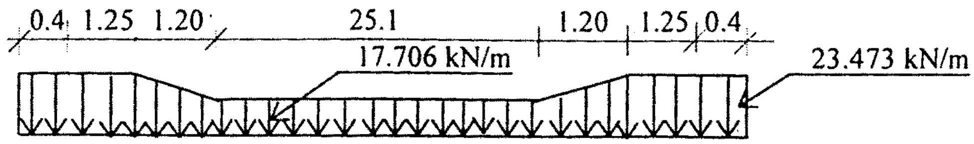
$\phi = 0.90$

$\phi Mn = 0.9 \times (9x0.000462x1458000x2.0805(1\text{-}0.6x1.1036x10^{-3}x(1458000/30000))$
$= 10986.17$ kN-m > 8298.41 kN-m

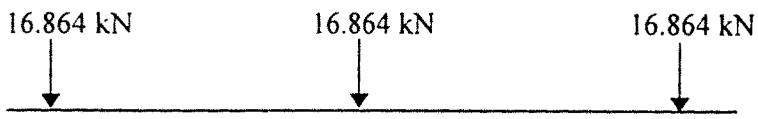
So, ultimate moment capacity of the PC girder is **OK**.

Design of Shear Reinforcement

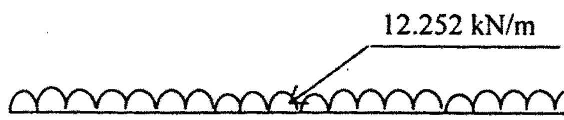
a) Dead Load Shear



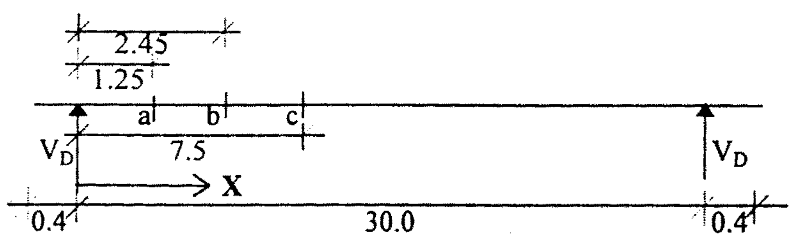
Self Weight of PC Girder



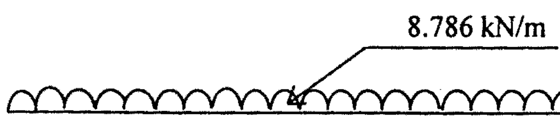
Load from X-Girder



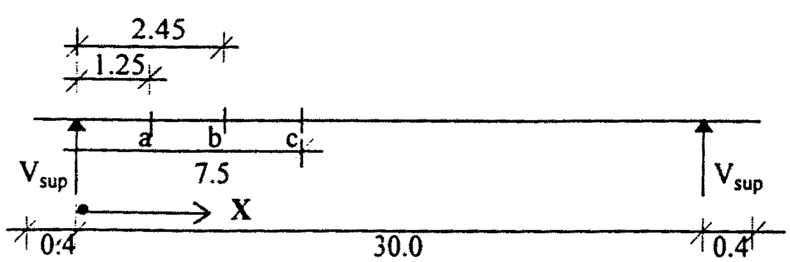
Load from Deck Slab



Dead Load Diagram



Superimposed Dead Load



Superimposed Dead Load Diagram

$(V_D)_a$ = (285.647-23.473x1.65)+8.432+(188.681-12.252x1.65) = 423.814 kN

$(V_D)_b$ = 285.647-23.473x1.65-(23.473+17.706)/2x1.2+8.432+(188.681-12.252x2.85)
= 384.404 kN

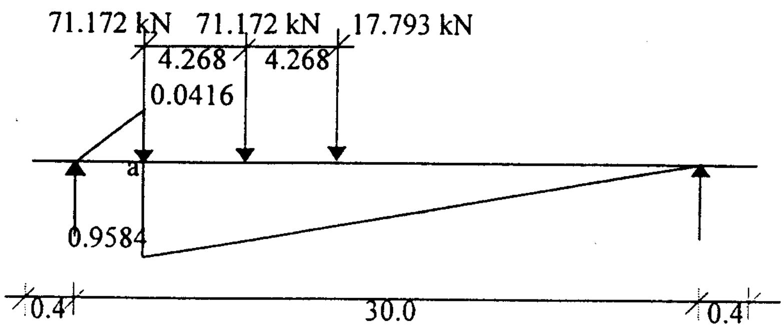
$(V_D)_c$ = (285.647-63.438-17.706x5.05)+8.432+(188.681-12.2524x7.9) = 233.12 kN

$(V_{sup})_a$ = 135.304-8.786x1.65 = 120.808 kN

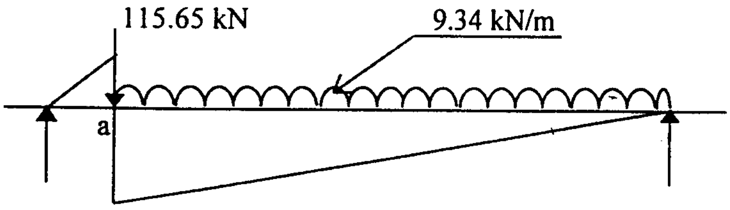
$(V_{sup})_b$ = 135.304-8.786x2.85 = 110.264 kN

$(V_{sup})_c$ = 135.304-8.786x7.9 = 65.89 kN

b) Live Load Shear



(a) Wheel Load



(b) UDL

IL Diagram For Shear at 'a'

At Service Condition

Live Load Shear at 'a'

For wheel load

$(VLL)_W$ = 0.9584/28.75(71.172x28.75+71.172x24.482+17.793x20.214)
= 138.286 kN

For lane load

$(VLL)_L$ = ½ x(115.65x0.9584+0.5x0.9584x28.75x9.34) = 119.76 kN

∴ Maximum live load shear at a, $(VLL)a$ = 138.286 kN

Live load shear for FPLL = 17.28-1.152x1.25 = 15.84 kN

Live Load Shear at 'b'

Influence Line Diagram is similar to Sec. 'a' as shown above

For wheel load

$(VLL)_W$ = 0.918/27.55(71.172x27.55+71.172x23.282+17.793x19.014) = 131.823 kN

For lane load

$(VLL)_L$ = ½ x(115.65x0.918+0.5x0.918x27.55x9.34) = 112.137 kN

Max. LL shear at 'b', $(VLL)_b$ = 131.823 kN

LL shear for FPLL at 'b' = 17.28-1.152x2.45 = 14.457 kN

Live Load Shear at 'c'

IL diagram is similar to sec.'a' as shown above

For Wheel Load

$(VLL)_W$ = 0.75/22.5x(71.172x22.5+71.172x18.232+17.793x13.964) = 104.91 kN

For Lane Load

$(VLL)_L$ = ½ x(115.65x0.75+0.5x0.75x22.5x9.34) = 82.77 kN

∴ Max LL shear at 'c', $(VLL)c$ = 104.91 kN

LL shear for FPLL at 'c' = 17.28-1.152x7.5 = 8.64 kN

At LFD

Factored Shear at 'a'

VU1 = 1.3x((423.814+120.808)x1+1.25x((1+0.224)x138.286+15.84)) = 1008.80 kN

VU2 = 1.3x((423.814+120.808)+1.67x(138.286+15.84)) = 1042.62 kN

∴ Max. factored shear at 'a', $(Vu)a$ = 1042.62 kN

Factored Shear at 'b'

VU1 = 1.3x((384.404+110.264)x1+1.25x(1.224x131.823+14.457)) = 928.75 kN

VU2 = 1.3x((384.404+110.264)x1+1.67(131.823+14.457)) = 960.64 kN

∴ Max. factored shear at 'b', $(Vu)b = 960.64$ kN

Factored Shear at 'c'

$VU1 = 1.3x((233.12+65.89)+1.25x(1.224x104.91+8.64)) = 611.42$ kN

$VU2 = 1.3x((233.12+65.895)+1.67x(104.91+8.64)) = 635.23$ kN

∴ Max. factored shear at 'c', $(Vu)c = 635.23$ kN

c) **Flexural Shear Resistance**

$V_{ci} = 1.575\sqrt{f'_c}\, b'd + V_i x M_{cr}/M_{max}$ kN/m^2 should not be less than $4.464\sqrt{f'_c}\, b'd$

$M_{cr} = (15.75\sqrt{f'_c} + f_{pc} - f_d) x I_{com}/Y_{bc}$,

where,
$f'_c = 30000$ kN/m^2
V_d = Unfactored dead load shear
V_i = Factored shear for superimposed loads
M_{max} = max. moment at the section
f_{pc} = compressive stress at bottom due to effective prestress force
f_d = stress at bottom due to unfactored dead loads

Calculation of V_{ci} at 'a' :

$b' = 0.4$ m, $d = 1.76$ m, $V_d = 423.814$ kN
$V_{i1} = 1.3(120.808x1+1.25(1.224x138.286+15.84) = 457.84$ kN
$V_{i2} = 1.3(120.808x1+1.67(138.286+15.84) = 491.66$ kN

∴ $V_i = 491.66$ kN

Total unfactored dead load moment at 'a'

$M_d = 285.647x1.25-23.473x1.65^2/2 +8.432x1.25+323.985x1.25-(12.252+8.786)x1.65^2/2$
$= 711.989$ kN-m

$f_d = 711.989/0.6575 = 1082.87$ kN/m^2
Loss of prestress at 'a' = 36.87%

Effective prestress $= 9x555x(1-0.3687) = 3153.34$ kN

$f_{pc} = 3153.34/1.3553+3153.34x0.005/0.6575 = 2326.67$ kN/m^2

$MU1 = 1.3(157.17x1+1.25(1.224x171.29+20.61)) = 578.51$ kN-m
$MU2 = 1.3(157.17x1+1.67(171.29+20.61)) = 620.93$ kN-m

∴$M_{max} = 620.93$ kN-m

$M_{cr} = (15.75\sqrt{f'_c} + f_{pc} - f_d) \times I_{comp}/Y_{bc}$
$= (15.75\sqrt{30000} + 2326.67 - 1082.87) \times 0.8855/1.3468$
$= 2611.38$ kN-m

$V_{ci} = 1.575\sqrt{f'_c}b'd + V_d + V_iM_{cr}/M_{max}$
$= 1.575\sqrt{30000} \times 0.4 \times 1.76 + 423.814 + 491.66 \times 2611.38/620.93$
$= 2683.58$ kN > $(4.464\sqrt{30000} \times 0.4 \times 1.76) = 544.32$ kN

Calculation of V_{ci} at 'b'

$V_d = 384.404$ kN
$V_{i1} = 1.3(110.264x1 + 1.25(1.224x131.823 + 14.457)) = 429.03$ kN
$V_{i2} = 1.3(110.264x1 + 1.67(131.823 + 14.457)) = 460.92$ kN

$\therefore V_i = 460.92$ kN

Total unfactored dead load moment at 'b'

$M_d = 285.647x2.45 - 23.473x1.65x2.025 - (23.473 - 17.706)x1.2^2/3 - 17.706x\ 1.2^2/2 + 8.432x2.45$
$+ 323.985x2.45 - (12.252 + 8.786)x2.85^2/2$
$= 1334.87$ kN-m

$f_d = 1334.87/0.5489 = 2431.90$ kN/m^2

Loss of prestress at 'b' = 36.627%
Effective prestress = 9x555x(1- 0.36627) = 3165.48 kN
$f_{pc} = 3165.48/1.085 + 3165x0.669/0.5489 = 6774.99$ kN/m^2

Mu1 = 1.3(295.81x1 + 1.25(1.224x320.17 + 38.786)) = 1084.39 kN-m
Mu2 = 1.3(295.81 + 1.67(320.17 + 38.786)) = 1163.85 kN-m

$\therefore M_{max} = 1163.85$ kN-m

$M_{cr} = (15.75\sqrt{30000} + 6774.99 - 2431.90)x0.7895/1.4385$
$= 3880.85$ kN-m

$V_{ci} = 1.575\sqrt{30000}x0.4x1.76 + 384.404 + 460.92x3880.85/1163.85$
$= 2113.39$ kN > 544.32 kN

Calculation of V_{ci} at 'c'

$V_d = 233.12$ kN
$V_{i1} = 1.3(65.89x1 + 1.25(1.224x104.91 + 8.64)) = 308.36$ kN
$V_{i2} = 1.3(65.89x1 + 1.67(104.91 + 8.64)) = 332.17$ kN

$\therefore V_i = 332.17$ kN

Total unfactored dead load momnet at 'c'

$M_d = 285.647x7.5-23.473x1.65x7.705-(23.473-17.706)x1.2/2x5.85-17.706x6.25^2/2$
$+8.432x7.5+323.985x7.5-(12.252+8.786)x7.9^2/2$
$= 3314.51$ kN-m

$f_d = 3314.51/0.5489 = 6038.46$ kN/m^2

Loss of prestress at 'c'-= 31.197%

Effective prestress = 9x555x(1-0.31197) = 3436.71 kN

f_{pc} = 3436.71/1.085 + 3436.7x0.669/0.5489 = 7356.14 kN/m^2

MU1 = 1.3(740.61x1+1.25(1.224x779.78+97.10)) = 2671.56 kN-m
MU2 = 1.3(740.61 + 1.67(779.78 + 97.10)) = 2866.50 kN-m

$\therefore M_{max}$ = 2866.50 kN-m

M_{cr} = (15.75√30000+7356.14-6038.46)x0.7895/1.4385 = 2220.40 kN-m

V_{ci} = 1.575√30000x0.40x1.76+233.12+332.17x2220.40/2866.5 = 682.47 kN>544.32 kN

d) WEB Shear Strength

$V_{cw} = (9.19 \times \sqrt{f'_c} + 0.3\, f_{pc}) b'd + V_p$

Where,
f_{pc} = Compressive stress at c.g. of composite section due to effective prestress force
V_p= Vertical component of effective prestress force at section

Calculation of V_{cw} at 'a'

Effective prestress, EPF = 3153.34 kN
Moment due to effective prestress, M_{PF} = 3153.34x0.005 = 15.77 kN-m
Moment due to self wt. of girder at the section, M_G = 325.106 kN-m
Area of girder section, ANC = 0.9931 m^2
Distance of girder bottom. from N.A. Y_B = 0.9991 m
Moment of inertia of girder section I_{NA} = 0.4351 m^4
Distance of girder bott. from NA of comp. sec., Y_{BC} = 1.3468 m

f_{pc} = EPF/ANC - (M_{PF}-M_G)(Y_{BC}-Y_B)/I_{NA}
= 3153.34/0.9931-(15.77-325.106)x(1.3468-0.9991)/0.4351
= 3422.45 kN/m^2

Average inclination angle of 9 cables, θ = 7.8 deg.

$\therefore$ Vp = EPF sinθ = 3153.34 sin 7.8 = 427.54 kN

$\therefore V_{cw}$ = (9.19x√30000+0.3x3422.45)x0.4x1.76+427.54 = 2270.96 kN

Calculation of V_{cw} at 'b'

EPF = 3165.48 kN
M_{PF} = 3165.48x0.275 = 870.507 kN-m
M_G = 605.88 kN-m
ANC = 0.7728 m^2
I_{NA} = 0.3848m^4
Y_{BC} = 1.4385 m

f_{pc} = 3165.48/0.7228-(870.507-605.88)x(1.4385-1.0068)/0.3848 = 4082.59 kN/m^2

Average inclination angle of 9 cables, $\theta = 7.68^0$

$\therefore$ Vp = 3165.48 Sin 7.68 = 423.035 kN

$\therefore$ Vcw = (9.19√30000+0.3x4082.59)x0.4x1.76+423.05 = 2408.87 kN

Calculation of V_{cw} at 'c'

FPF = 3436.71 kN
M_{PF} = 3436.71x0.669 = 2299.158 kN-m
M_G = 1502.27 kN-m
ANC = 0.7228 m^2
I_{NA} = 0.3848 m^4
Y_{BC} = 1.4385 m

f_{pc} = 3436.71/0.7228-(2299.158-1502.27)x(1.4385-1.0068)/0.3848 = 3860.70 kN/m^2

Average inclination angle of 9 cables, $\theta = 4.283^0$
$\therefore$ Vp = 3436.71xSin 4.283 = 256.66 kN
$\therefore$ V_{cw} = (9.19√3000+0.3x3860.70)x0.40x1.76+256.66 = 2192.64 kN

e) Check Adequacy and Provide Web Reinforcement :

Cross-section at 'a'
V_{ci} = 2683.58 kN
V_{cw} = 2270.96 kN
$V_{cw} < V_{ci}$

$\therefore$ Allowable shear strength at Sec. 'a'

$V_c = V_{cw}$ = 2270.96 kN

Max. factored shear at section

V_u = 1042.62 kN < 0.85x2270.96 = 1930.32 kN

So, theoretically no web reinforcement is required. But a nominal steel of **Y10-150** 2-legged stirrup will be provided.

Cross section at 'b'

V_{ci} = 2113.39 kN
V_{cw} = 2405.87 kN
$V_{ci} < V_{cw}$

$\therefore$ $V_c = V_{ci}$ = 2113.39 kN

Max. factored shear at section

V_u = 960.64 < 0.85x2113.39 = 1796.38 kN

So, a nominal steel of **Y10-150** 2-legged stirrup will be provided.

Cross section at 'c'

V_{ci} = 682.47 kN
V_{cw} = 2192.64 kN

$\therefore$ V_c = 682.47 kN
V_u = 635.23 > 0.85x682.47 = 580.10 kN

Shear force to be resisted by steel

Vs = V_u - 0.85xV_c = 635.23-0.85x682.47 = 55.13 kN

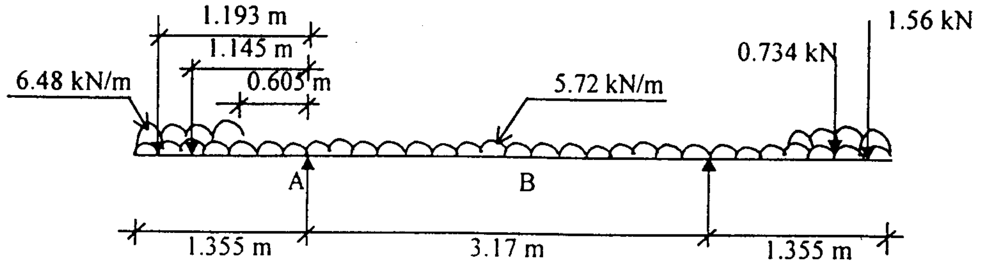
Using **Y10** 2-legged stirrup

As = $78.5\text{x}10^{-6}$ x2 = $1.57\text{x}10^{-4}$ m2

Spacing = $1.57\text{x}10^{-4}$x275000x1.76x1000/55.13 = 1378.34 mm

But max. spacing of web reinforcement has been limited to 250 mm. So, a nominal steel of **Y10-250**, 2-legged stirrup may be provided.

2.2 Structural Design of Deck Slab



Deck Slab design

Self wt. of deck slab = 24 x 0.20 = 4.8 kN/m^2
Self wt. of W.C. = 23x.04 = 0.92 kN/m^2
Self wt. of S. Walk = 6.48 kN/m of span
Self wt. of S. Railing = 0.734 kN/m of span
C/C distance of girder = 3170 mm
Clear span + thickness of deck slab = (3170-400)+200 = 2970 mm

Dead Load Moments

$(M_A)DL = 0.734x1.145+6.48x0.937+1.56x1.193+5.72x.58^2/2 = 9.74$ kN-m/m
$(M_B)DL = 0.734x2.63+6.48x2.422+1.56x2.678+5.72x.58x1.775-20.586x1.485 = -3.968$ kN-m/m

Considering partial fixity of deck slab on the PC girder

$(M_B)DL = 5.72x2.97^2/10 = 5.045$ kN-m/m

Live Load Moments

Moment due to sidewalk LL, pxMs = (2.88x0.4)x0.83 = 0.956 kN-m/m

Width of wheel load distribution

E = 0.8x0.73x+1.143 = 1.727 m
$(M_A)LL$ = 71.172/1.727x0.73 = 30.048 kN-m/m
$(M_B)LL$ = (2.97+0.61)/9.74)x71.172 = 26.16 kN-m/m

Impact fraction (IF) = 15.24/2.97+38 = 0.37>0.3 ∴ IF = 0.3

Cracking moment of deck slab section, $M_{cr} = 0.6228\sqrt{f'c} \times Z_b$
$Z_b = 1000x200^3/(12x100) = 6.67 \times 10^6\ mm^3$
∴ $M_{cr} = 0.622\sqrt{20} \times 6.67 \times 10^6/10^6 = 18.577$ kN-m
Minimum flexural strength $M_F = 1.2 \times M_{cr} = 1.2 \times 18.577 = 22.29$ kN-m

Total Factored Moments :
$M_A = 1.3(1x9.74 +1.0(0.956+1.3x30.084)) = 64.746$ kN-m/m>M_F
$M_B = 1.3(1x5.045+1.0(1.3x26.16)) = 50.768$ kN-m/m>M_F

Design moment :
$(M_A)_D = 64.746$ kN-m
$(M_B)_D = 50.768$ kN-m

Using **Y16-90** as -ve reinforcement on Cantilevered Part.

$(As)_A$ = 201x1000/90 = 2233.33 mm^2
a = (2233.33x275)(0.85x20x1000) = 36.13 mm
d = 200-50-8 = 142 mm
$(\varphi M_n)_A$ = 0.9x2233.33x275x(142-36.13/2)/10^6
= 68.505 kN-m/m >64.746 kN-m/m, Hence **OK.**

Using **Y16-125** +ve reinforcement at mid span.

$(As)_B$ = 201x1000/125 = 1608 mm^2
a = (1608x275)(0.85x20x1000) = 26.01 mm
d = 200-40-8 = 152 mm

$(\varphi M_n)_B$ = 0.90x1608x275(152-26.01/2)/10^6
= 55.317 kN-m/m >50.768 kN-m/m, Hence **OK.**

Distribution Reinforcement

Asd = 121/√S not greater than 67% of $(As)_B$
= 121/√2.97 = 70.21

Asd = 0.67x1608 = 1077.36 mm^2

Using Y12, spacing = 113/1077.36x1000 ≈ 104 mm c/c
So, use **Y12-100** as distributed reinf. bar on the bottom layer.

Temperature and Shrinkage Reinforcement :

Ast = .0025x1000x200/2= 250 mm^2

Using Y12, spacing = 113/250x1000 = 452 mm c/c
So, use **Y12-250** as Temperature and Shrinkage reinforcement on the top layer.

ρ = 2233.33/1000x142 = 0.0157 f'_c = 20 N/mm^2
R_A = $\rho x f_y (1-0.59\rho f_y/f'_c)$ f_y = 275 N/mm^2
= 0.0157x275(1-0.59x0.015x2.75/20)
= 3.767
d = √(M/φRb)
$(d)_A$ = √(64.746x10^6 /0.9x3.767x1000) = 138.18 mm

Total thickness of deck slab from -ve moment consideration = 138.18+50+8 = 196.18 mm

ρ_b = 1608/1000x152 = 0.01057
R_B = 0.01057x275(1-0.59x0.01057x275/20) = 2.657
$(d)_B$ = √50.768x10^6 /0.9x2.675x1000 = 145.70 mm

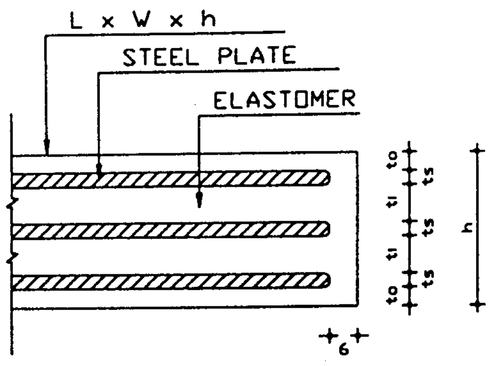
Total thickness of deck from +ve moment consideration = 145.706+40+8 = 193.706m

Hence, Provided thickness of deck slab 200 mm is **OK.**

CHAPTER 3

DESIGN OF ELASTOMERIC BEARING

3.1 Design of Elastomeric Bearing



a) Data from Superstructure

Span length c/c brg. = 30.0
Total dead load reaction, R_{DL} = 618714 N
Total live load reaction, R_{LL} = 92515 N
Deflection of bridge at mid span, Def. = 12.37 mm
Total rotation at support, α = 0.0045

Properties of elastomer and steel

Shear modulus of elastomer at 73^0 F = 0.80 N/mm^2
Modifying factor, β = 1.0
Constant dependent on elastomer hardness, $\bar{k}$ = 0.75
Allowable stress of steel plate, fs = 100.0 N/mm^2
Co-efficient of thermal expansion concrete = 0.000012/^{0}C
Temparature difference = 35^{0}C
Shrinkage strain of concrete = 0.0003

Provided size of Elastomeric Bearing

Length, L = 250 mm
Width, W = 450 mm

Thickness of internal elastomer layer, t_i = 10 mm
Thickness of external elastomer layer, t_o = 5 mm
Thickness of steel layer, t_s = 3 mm
Clear cover, c= 6 mm
Total no. of internal elastomer layer, N_1 = 3
total no. steel layer, N_2 = 4

Total thickness of elastomer in bearing :

$H_{rt} = N_1 \times t_i + 2 \times t_o$
$= 3 \times 10 + 2 \times 5 = 40$ mm

Min. vertical reaction = 618714 N
Max. vertical reaction = 618714+92515 = 711229 N

Shape factor, S = LxW/(2 t_o (L+W)) = 250x450/(2x10x(250+450)) = 8.036

b) Check for compressive stress

Allowable compressive stress for rectangular elastomeric bearing (1000 psi) = 6.896 N/mm^2
Required plan area of elastomeric bearing = 711229/6.896 = 103136.45 mm^2

Provided area = 250x450 = 112500.0 mm^2

So, provided plan area for bearing is **OK.**

Average compressive stress on bearing = 711229/(250x450) = 6.322 N/mm^2 <6.896 N/mm^2

So, the bearing is **OK**. for compressive stress.

c) Check for compressive deflection

Modulus of elasticity of elastomer :

$E_c = 3G(1+2\ \bar{K}S^2) = 3x0.8(1+2x0.75x8.036^2) = 234.88\ N/mm^2$

Average compressive deformation of elastomer due to external loads = 6.322x40/234.88
= 1.0766 mm

Allowable compressive strain of elastomer due to average compressive stress of 6.322 N/mm^2 (916.69 psi), (Ref. Figure 14.4.1.2A AASHTO'92.)

$\varepsilon_{ci} = 4.5\%$

Total compressive deflection
$\Delta_c = 40x4.5/100 = 1.80$ mm

Since, actual compressive deflection of 1.0766 mm at service load is less than allowable compressive deflection of 1.80 mm. Hence the bearing is **OK** for compressive deflection.

d) Check for shear

Total strain due to temp. and shrinkage = $\alpha\Delta t + \Delta_{SH}$ = 0.000012x35+0.0003 = 7.2x10^{-4}

Deformation due to temp. and shrinkage, Δ_{T+S} = 7.2x10^{-4} x 30000/2 = 10.8 mm

Horizontal force due to braking per bearing
H_F = 0.05(9.34x30.80+80.068)x1000/4 = 4596.75 N

Shear stiffness, Kq = LxWxG/H_{rt} = 250x450x0.8/40 = 2250 N/mm

Deformation due to braking force, $\Delta_b = H_F/Kq$ = 4596.75/2250 = 2.043 mm

Total shear deformation, $\Delta_s = \Delta_{T+S} + \Delta_b$ = (10.8+2.043)mm = 12.843 mm

Total thickness of elastomer (40 mm) is greater than 2Δs (25.686mm). So, is the design is **OK** for shear.

e) Check for rotation

Relative rotation of top and bottom surface of bearing, $\theta = 2\ \Delta_c/L$ = 2x1.8/250 = 0.0144 rad

Rotation of girder end at bearing due to applied loads, $\propto$ = 0.0045 rad.

Allowable rotation θ is greater than max. rotation $\propto$ at service load. So, the design is **OK** for rotation.

f) Check for Stability

Total thickness of bearing , h = 40+3x4 = 52 mm

For rectangular bearing :

L/3 = 250/3 = 83.33 mm
W/3 = 450/3 = 150.0 mm
∴ Min. of L/3 and W/3 = 83.33 mm

Total thickness of elastomeric bearing (52 mm) is less than 83.33 mm. Hence, the design is **OK** for stability.

g) Check for Reinforcement

Average thickness of elastomer surrounding steel layer = 10 mm

Stress in reinforcement = 11.72x10 = 117.2 N/mm

Allowable stress of reinforcement = 3x100 N/mm
= 300 N/mm > 117.2 N/mm.

So, the design is **OK** for reinforcement.

CHAPTER 4

SUBSTRUCTURE

4.1 Structural Design of Abutment-Wing Wall

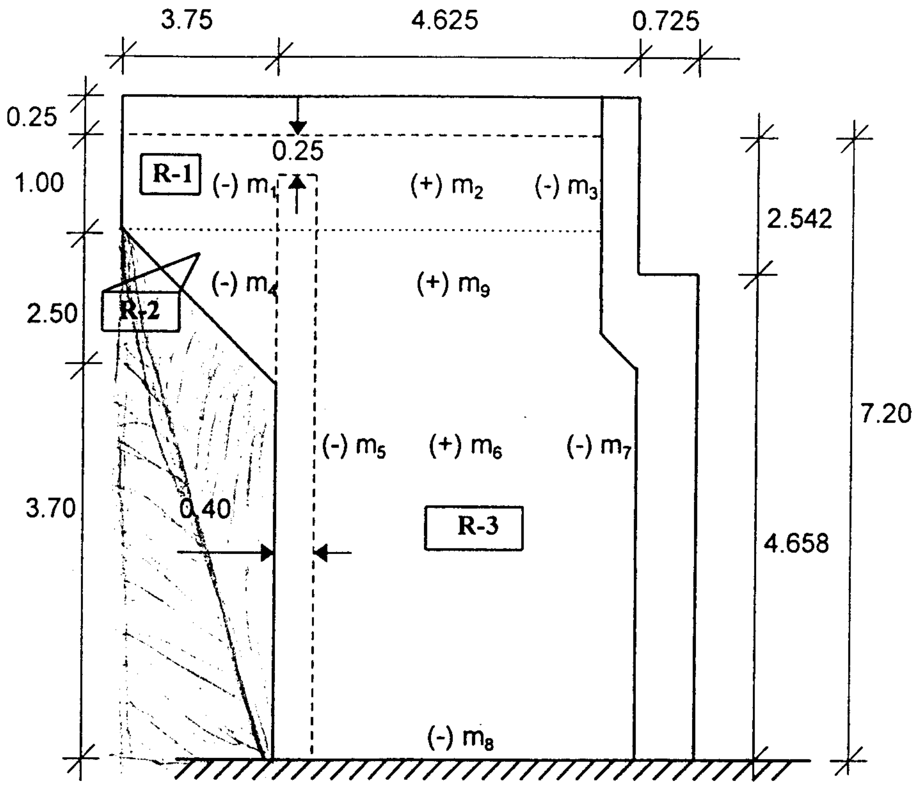


Fig. Side Elevation

Notes:

- (+) m means positive moment.
- (-) m means negative moment.
- All (-) m checked against minimum flexural strength of 1.2 times cracking moment.
- All (+) m checked against minimum flexural strength and 1.33 times factored moment.

Design Data

Height of surcharge		= 0.61 m
Unit weight of water		= 10.0 kN/m^3
Unit weight of moist soil		= 18.0 kN/m^3
Co-efficient of active earth pressure		= 0.333
Depth of water table		= 0.75 m
Concrete strength,	F_{ci}	= 20000 kN/m^2
	E_c	=19560000.0 kN/m^2
Steel strength,	F_y	= 275000 kN/m^2

a) Wing Wall

Region - 1 (R-1)

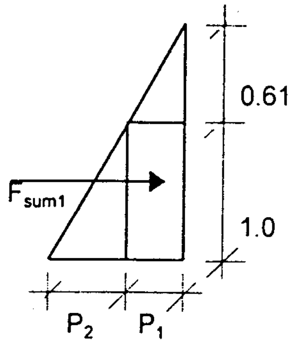


Fig. Horizontal Pressure Diagram

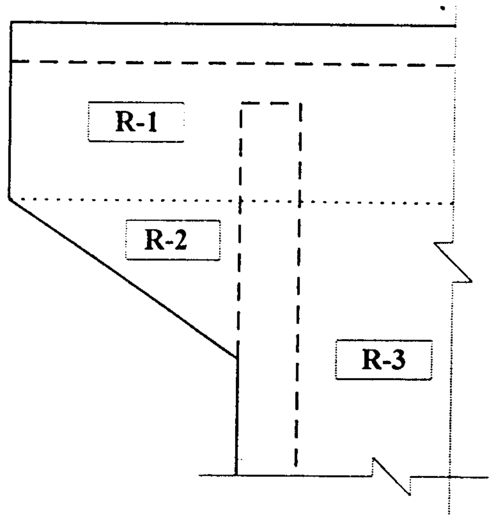


Fig. Flag Portion of Abutment

$P_1 = 0.333 \times 18.0 \times 0.61$
$= 3.656\ kN/m^2$

$P_2 = 0.333 \times 18.0 \times 1.0$
$= 5.994\ kN/m^2$

$F_{sum1} = P_1 \times 1.0 + 0.5 \times P_2 \times 1.0$
$= 3.656 \times 1.0 + 0.5 \times 5.994 \times 1.0$
$= 6.653\ kN/m^2$

Cantilever bending moment

(-) $m_1 = 1.69 \times 6.653 \times (3.75 + 0.25/2)^2/2$
$= 84.41\ kN/m^2$

Section modulus, $Z_{sec} = (1.0 - 0.25) \times 0.4^3/12 \times (0.4/2)$
$= 0.02\ m^3$

Cracking moment, $M_{cr} = f_r \times Z_{sec}$

where, f_r = modulus of rupture

Minimum flexural strength $= 1.2 \times M_{cr}$
$= 1.20 \times 19.70 \times \sqrt{f'_c} \times Z_{sec}$
$= 1.20 \times 19.70 \times \sqrt{2000} \times 0.02$
$= 66.864$ kN-m

Since, cantilever bending moment (-) m_1 = 84.41 kN-m > minimum flexural strength = 66.864 kN-m. Hence, external moment (-) m_1 governs the design.

Providing **Y16-125**

Steel area $= (201/125) \times 1000 \times (1.0 - 0.25) = 1206\ mm^2$

$a = (1.206 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times (1.0 - 0.25)) = 0.026$

Design strength, ϕM_n $= 0.90 \times 1.206 \times 10^{-3} \times 275000 \times (0.332 - 0.026/2)$
$= 95.22$ kN-m

Since, design strength is greater than external moment, so, provided reinforcement **Y16-125** is OK.

Calculation of positive moment in horizontal direction in R-1

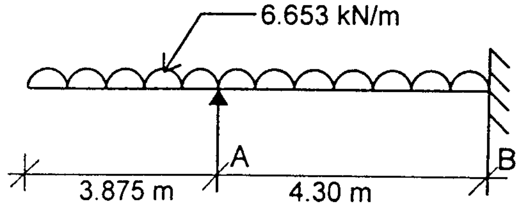
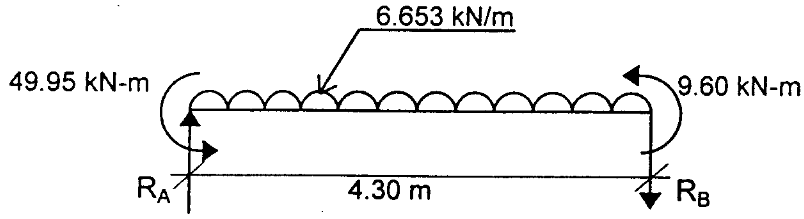


Fig. Horizontal Span Loading on Wing Wall (R-1)

Moment distribution:

+ 49.95 (Conc.)	-10.25	+ 10.25
	- 39.70 →	- 19.85
	- 49.95	- 9.60



$R_A = (49.95 + 9.60 + (6.653 \times 4.3^2/2))/4.30$
$= 28.153$ kN

Location of maximum positive moment, $x = 28.153/6.653 = 4.232$ m

Bending moment

$(+)\ m_2 = 1.69 \times (28.153 \times 4.232 - 6.653 \times 3.875^2/2 - 6.653 \times 4.232^2/2)$
$= 16.253$ kN-m.

Average thickness of concrete at the section $= 0.45$ m

Section modulus, Z_{sec} $= ((1.0\text{-}0.25) \times 0.45^3/12)/(0.45/2)$
$= 0.0253\ m^3$

Minimum flexural strength $= 1.2 \times 19.70 \times \sqrt{20000} \times 0.0253$
$= 84.58$ kN-m

Since, minimum flexural strength is greater than external positive moment, so, the former one governs the design.

Providing **Y16-150**

Steel area = (201/150) x 1000 x (1.0-0.25) = 1005 mm^2

a = $(1.005 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times (1 - 0.25)) = 0.0216$ m

Design strength, ϕM_n = $0.90 \times 1.005 \times 10^{-3} \times 275000 \times (0.382 - 0.0216/2)$
= 92.33 kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-150** is OK.

Moment in Span AB

From the moment distribution it is clear that in span AB in R-1, direction of horizontal moment (-) m_3 changes direction from negative to positive in the span. Hence, reinforcement provided for maximum positive moment will be continued upto abutment end.

Region - 2 (R-2)

Active earth pressure at c.g. of R-2

$P_3 = 0.333 \times 18 \times (1.0 + 2.5/3)$
= 10.989 kN/m^2

F_{sum2} = $10.989 \times (3.75 \times 2.5)/2$
= 51.51 kN/m

Bending moment

(-) m_4 = $1.69 \times 51.51 \times 3.75/3$
= 108.815 kN-m

Thickness of concrete at the section = 0.40 m

Section modulus, Z_{sec} = $(2.5 \times 0.4^3/12)/(0.4/2)$
= 0.0666 m^3

Minimum flexural strength = $1.2 \times 19.7 \times \sqrt{20000} \times 0.0666$
= 222.66 kN-m > 108.815 kN-m

So, minimum flexural strength governs the design.

Providing **Y16-150**

Steel area = (201/150) x 1000 x 2.5 = 3350 mm^2

a = $(3.35 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 2.5) = 0.0216$ m

Design strength, ϕM_n = $0.90 \times 3.35 \times 10^{-3} \times 275000 \times (0.330 - 0.0216/2)$
= 257.36 kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-150** is OK.

Region - 3 (R-3)

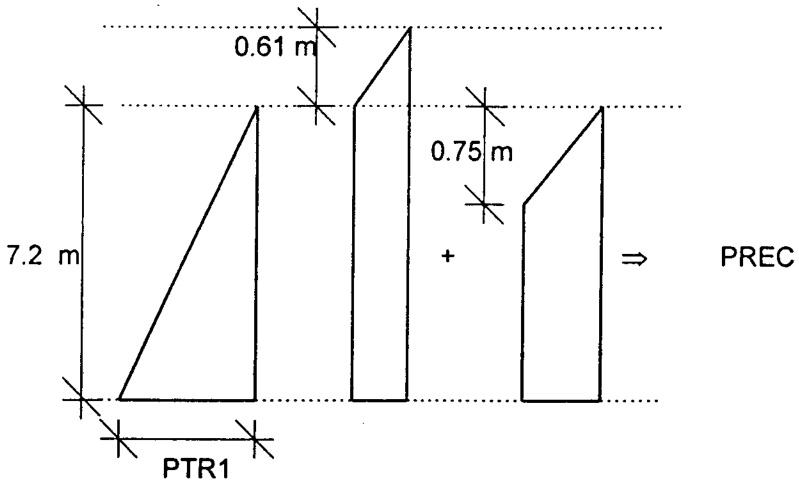


Fig. Horizontal Pressure Diagram (R-1)

Moment co-efficient for triangular load
(Ref. Reynolds & Steedman, RC Designer's Hand Book, 1988, Table 53)

Ratio, k = 4.615/7.2 = 0.64

Co-efficient for negative horizontal moment	= 0.03766
Co-efficient for negative vertical moment	= 0.018357
Co-efficient for positive horizontal moment	= 0.019009
Co-efficient for positive vertical moment	= 0.003755

Moment co-efficient for rectangular load [Ref. W.T. Moody, Moment and Reactions for Rectangular Plates (Figure-1)]

Co-efficient for negative horizontal moment	= 0.036104
Co-efficient for negative vertical moment	= 0.02221
Co-efficient for positive horizontal moment	= 0.01875
Co-efficient for positive vertical moment	= 0.005961
Triangular pressure at base, PTR1	= 0.333 x 18 x (8.0 - 0.8)
	= 43.157 kN/m^2

Total rectangular pressure (earth pressure + excess hydrostatic pressure)

PREC = 0.333 x 18 x 0.61 + 10.0 x 0.75
= 11.165 kN/m^2

Factored horizontal negative moment:

(-) m_5 = 1.69 x (0.03766 x 43.157 x 4.615^2 + 0.036104 x 11.156 x 7.2^2)
= 93.79 kN-m

Thickness of concrete section = 0.50 m

Section modulus, Z_{sec} = $(1.0 \times 0.5^3/12)/(0.5/2)$
= $0.04166\ m^3$

Minimum flexural strength = $1.20 \times 19.70 \times \sqrt{20000} \times 0.04166$
= 139.28 kN-m > 93.79 kN-m

So, minimum flexural strength governs the design.

Providing **Y16-150**

Steel area = (201/150) x 1000 = 1340 mm^2

$a = (1.34 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.0216$ m

Design strength, ϕM_n = $0.9 \times 1.34 \times 10^{-3} \times 275000 \times (0.432 - 0.0216/2)$
= 139.69 kN-m

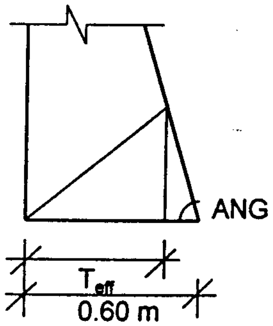
Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-150** is OK.

Factored vertical negative moment:

(-) m_8 = $1.69 \times (0.018357 \times 43.157 \times 7.2^2 + 0.02221 \times 11.156 \times 7.2^2)$
= 91.11 kN-m

Thickness of concrete section at bottom:

ANG = $TAN^{-1}(7.2 - 0.25)/(0.6 - 0.4)$
= 88.35^0



Effective thickness, T_{eff} = $0.60 - (0.60 \times \sin(90^0 - 88.35^0)) \times \cos 88.35^0$
= 0.596 m

Section modulus, Z_{sec} = $(1.0 \times 0.596^3/12)/(0.596/2)$
= $0.0582\ m^3$

Minimum flexural strength = $1.2 \times 19.7 \times \sqrt{20000} \times 0.0582$
= 194.57 kN-m > 91.11 kN-m

So, minimum flexural strength governs the design.

Providing **Y16-125**

Steel area $= (201/125) \times 1000 = 1608\ mm^2$

$a = (1.608 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.02601\ m$

Design strength, $\phi M_n = 0.90 \times 1.608 \times 10^{-3} \times 275000 \times (0.528 - 0.02601/2)$
$= 204.95$ kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-125** is OK.

Factored horizontal positive moment:

$(+)\ m_6 = 1.69 \times (0.019009 \times 43.157 \times 4.615^2 + 0.01875 \times 11.156 \times 7.2^2)$
$= 47.85$ kN-m

Average thickness of concrete section = 0.50 m

Section modulus, $Z_{sec} = (1.0 \times 0.5^3/12)/(0.5/2)$
$= 0.04166\ m^3$

i) Minimum flexural strength $= 1.2 \times 19.7 \times \sqrt{20000} \times 0.04166 = 139.28$ kN-m
ii) Minimum flexural strength $= 1.33 \times 47.85 = 63.64$ kN-m

For positive moment minimum of (i) & (ii) has been considered as minimum flexural requirement. Above calculation shows that minimum flexural strength governs the design.

Providing **Y12-175**

Steel area $= (113/175) \times 1000 = 646\ mm^2$

$a = (0.646 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.0104\ m$

Design strength, $\phi M_n = 0.90 \times 0.646 \times 10^{-3} \times 275000 \times (0.432 - 0.0104/2)$
$= 68.24$ kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-175** is OK.

Factored vertical positive moment:

$(+)\ m_9 = 1.69 \times (0.003755 \times 43.157 \times 7.2^2 + 0.005961 \times 11.156 \times 7.2^2)$
$= 20.02$ kN-m

Average thickness of concrete section = 0.50 m

Section modulus, $Z_{sec} = 0.4166\ m^3$

i) Minimum flexural strength = 139.28 kN-m
ii) Minimum flexural strength $= 1.33 \times 20.02 = 26.63$ kN-m

From (i) & (ii) minimum flexural strength = 26.63 kN-m

Providing **Y12-150**

Steel area = (113/150) x 1000 = 753 mm^2

a = (0.753 x 10^{-3} x 275000)/(0.85 x 20000 x 1.0) = 0.01218 m

Design strength, ϕM_n = 0.90 x 0.753 x 10^{-3} x 275000 x (0.432 - 0.01218/2)
= 79.375 kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-150** is OK.

b) Abutment Wall

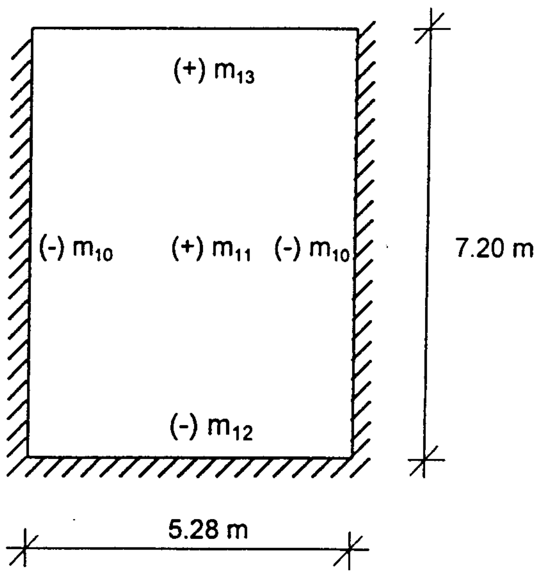


Fig. Vertical Section of Abutment

Moment co-efficient for triangular load:

Ratio, k= 5.28/7.2 = 0.7333

Co-efficient for negative horizontal moment	= 0.035518
Co-efficient for negative vertical moment	= 0.022492
Co-efficient for positive horizontal moment	= 0.017810
Co-efficient for positive vertical moment	= 0.005061

Moment co-efficient for rectangular load:

Co-efficient for negative horizontal moment	= 0.04582
Co-efficient for negative vertical moment	= 0.02778
Co-efficient for positive horizontal moment	= 0.02378
Co-efficient for positive vertical moment	= 0.007853

Triangular pressure at base, PTR1 = 43.157 kN/m^2
Rectangular pressure at base, PREC = 11.156 kN/m^2

Factored horizontal negative moment:

(-) m_{10} = 1.69 x (0.035518 x 43.157 x 5.28^2 + 0.04582 x 11.156 x 7.2^2)
= 117.0 kN-m/m

Thickness of abutment = 0.60 m (although)

Section modulus, Z_{sec} = (1 x 0.6^3 /12)/(0.6/2)
= 0.06 m^3

Minimum flexural strength = 1.20 x 19.70 x √2000 x 0.06
= 200.59 kN-m/m > 117.0 kN-m/m

So, minimum flexural strength governs the design.

Providing **Y16-125**

Steel area = (201/125) x 1000 = 1608 mm^2

a = (1.608 x 10^{-3} x 275000)/(0.85 x 20000 x 1.0) = 0.026 m

Design strength, ϕM_n = 0.90 x 1.608 x 10^{-3} x 275000 x (0.532 - 0.026/2)
= 206.55 kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-125** is OK.

Factored horizontal positive moment:

(+) m_{11} = 1.69 x (0.017810 x 43.157 x 5.28^2 + 0.02378 x 11.156 x 7.2^2)
= 59.455 kN-m/m

i) Minimum flexural strength = 200.59 kN-m/m
ii) Minimum flexural strength = 1.33 X 59.455 = 79.08 kN-m/m

From (i) & (ii) minimum flexural strength = 79.08 kN-m/m > 59.455 kN-m/m

So, minimum flexural strength governs the design.

Providing **Y12-150**

Steel area = (113/150) x 1000 = 753 mm^2

a = (0.753 x 10^{-3} x 275000)/(0.85 x 20000 x 1.0) = 0.01218 m

Design strength, ϕM_n = 0.90 x 0.753 x 10^{-3} x 275000 x (0.532 - 0.01218/2)
= 98.0 kN-m > 79.08 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-150** is OK.

Factored vertical negative moment:

$(-)\ m_{12} = 1.69 \times (0.022492 \times 43.157 \times 7.2^2 + 0.02778 \times 11.156 \times 7.2^2)$
$= 112.19$ kN-m/m

Minimum flexural strength = 200.59 kN-m/m > 112.19 kN-m/m
So, minimum flexural strength governs the design.

Providing **Y16-125**

Steel area = (201/125) x 1000 = 1608 mm^2

$a = (1.608 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.026$ m

Design strength, $\phi M_n = 0.90 \times 1.608 \times 10^{-3} \times 275000 \times (0.532 - 0.026/2)$
$= 206.55$ kN-m > 200.59 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-125** is OK.

Factored vertical positive moment:

$(+)\ m_{13} = 1.69 \times (0.005061 \times 43.157 \times 7.2^2 + 0.007853 \times 11.156 \times 7.2^2)$
$= 26.81$ kN-m/m

i) Minimum flexural strength = 200.59 kN-m/m
ii) Minimum flexural strength = 1.33 x 26.81 = 35.66 kN-m/m

From (i) & (ii) minimum flexural strength = 35.66 kN-m/m > 26.81 kN-m/m
So, minimum flexural strength governs the design.

Providing **Y12-150**

Steel area = (113/150) x 1000 = 753 mm^2

$a = (0.753 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.01218$ m

Design strength, $\phi M_n = 0.90 \times 0.753 \times 10^{-3} \times 275000 \times (0.532 - 0.01218/2)$
$= 98.0$ kN-m > 35.66 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-150** is OK.

c) **Back Wall**

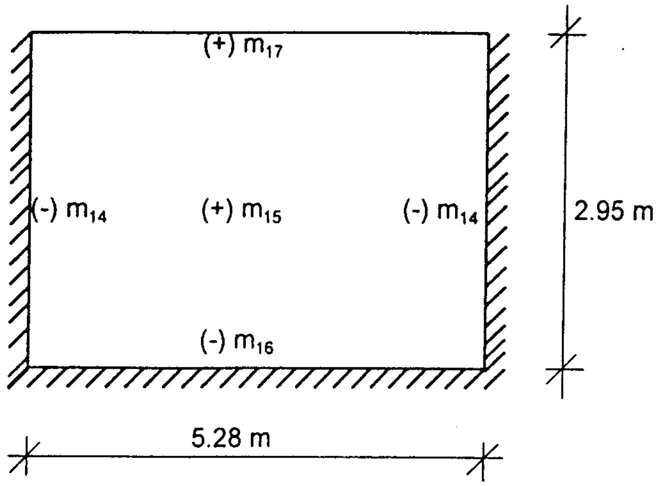


Fig. Vertical Section of Back Wall

Depth of girder = 2.60m

Triangular earth pressure at a depth 2.95 m, PTR1 = 0.333 x 18.0 x 2.95 = 17.682 kN/m^2

Rectangular pressure, PREC = 11.156 kN/m^2

Moment co-efficient for triangular load:

Ratio, k= 5.28/2.95 = 1.7898

Co-efficient for negative horizontal moment	= 0.014615
Co-efficient for negative vertical moment	= 0.075321
Co-efficient for positive horizontal moment	= 0.0048752
Co-efficient for positive vertical moment	= 0.01701

Moment co-efficient for rectangular load:

Co-efficient for negative horizontal moment	= 0.226538
Co-efficient for negative vertical moment	= 0.16937
Co-efficient for positive horizontal moment	= 0.092351
Co-efficient for positive vertical moment	= 0.024094

Factored horizontal negative moment:

Force on back wall due to earth pressure on wing wall
Total earth pressure on wing wall at level of back wall

EPB = (0.333 x 18 x 0.61 +(0.333 x 18 x 2.95)/2) x 2.95
= 36.867 kN/m

TEPB = 36.867 x 4.3/2
= 79.264 kN

Factored, TEPB = 1.69 x (79.264/2.95)
= 45.41 kN

Total area of steel required to resist 45.41 kN tensile force:

ASB = 45.41/(0.7 x 275000)
= $2.356 \times 10^{-4}\ m^2$

Calculation of equivalent moment capacity of steel ASB

a = $(2.356 \times 10^{-4} \times 275000)/(0.85 \times 20000 \times 1)$
= 3.815×10^{-3} m

Moment capacity, BMN = $2.356 \times 10^{-4} \times 275000 \times (0.332 - 3.815 \times 10^{-3}/2)$
= 21.386 kN-m/m

Total factored horizontal negative moment
(-) m_{14} = $1.69 \times (0.014615 \times 17.682 \times 5.28^2 + 0.226538 \times 11.156 \times 2.95^2)$
= 70.73 kN-m/m

Thickness of back wall = 0.40 m (although)

Section modulus, Z_{sec} = $(1 \times 0.4^3/12)/(0.4/2)$
= $0.0266\ m^3$

Minimum flexural strength = $1.20 \times 19.70 \times \sqrt{2000} \times 0.0266$
= 88.93 kN-m/m > 70.73 kN-m/m

So, minimum flexural strength governs the design.

Providing **Y16-150**

Steel area = (201/150) x 1000 = 1340 mm^2

a = $(1.34 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.02167$ m

Design strength, ϕM_n = $0.90 \times 1.34 \times 10^{-3} \times 275000 \times (0.332 - 0.02167/2)$
= 106.51 kN-m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-150** is OK.

Factored horizontal positive moment:

(+) m_{15} = $1.69 \times (0.0048752 \times 17\ 682 \times 5.28^2 + 0.092351 \times 11.156 \times 2.95^2)$
= 40.60 kN-m/m

i) Minimum flexural strength = 88.93 kN-m/m
ii) Minimum flexural strength = 1.33 x 40.60 = 53.998 kN-m/m

From (i) & (ii) minimum flexural strength = 53.998 kN-m/m > 40.60 kN-m/m
So, minimum flexural strength governs the design.

Providing **Y12-150**

Steel area $= (113/150) \times 1000 = 753\ mm^2$

$a = (0.753 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.01218\ m$

Design strength, ϕM_n $= 0.90 \times 0.753 \times 10^{-3} \times 275000 \times (0.332 - 0.01218/2)$
$= 60.74$ kN-m > 53.998 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-150** is OK.

Factored vertical negative moment:

$(-)\ m_{16} = 1.69 \times (0.075321 \times 17.682 \times 2.95^2 + 0.16937 \times 11.156 \times 2.95^2)$
$= 47.38$ kN-m/m

Minimum flexural strength = 88.93 kN-m/m > 47.38 kN-m/m

So, minimum flexural strength governs the design.

Providing **Y16-150**

Steel area $= (201/150) \times 1000 = 1340\ mm^2$

$a = (1.34 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.02167\ m$

Design strength, ϕM_n $= 0.90 \times 1.34 \times 10^{-3} \times 275000 \times (0.332 - 0.02167/2)$
$= 106.51$ kN-m > 88.93 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y16-150** is OK.

Factored vertical positive moment:

$(+)\ m_{17} = 1.69 \times (0.01701 \times 17.682 \times 2.95^2 + 0.024094 \times 11.156 \times 2.95^2)$
$= 8.38$ kN-m/m

i) Minimum flexural strength = 88.93 kN-m/m
ii) Minimum flexural strength $= 1.33 \times 8.38 = 11.145$ kN-m/m

From (i) & (ii) minimum flexural strength = 11.145 kN-m/m > 8.38 kN-m/m

So, minimum flexural strength governs the design.

Providing **Y12-150**

Steel area $= (113/150) \times 1000 = 753\ mm^2$

$a = (0.753 \times 10^{-3} \times 275000)/(0.85 \times 20000 \times 1.0) = 0.01218\ m$

Design strength, ϕM_n $= 0.90 \times 0.753 \times 10^{-3} \times 275000 \times (0.332 - 0.01218/2)$
$= 60.74$ kN-m > 11.145 kN-m/m

Since, design strength is greater than minimum flexural strength, so, provided reinforcement **Y12-150** is OK.

4.2 Structural Design of Tie Wall

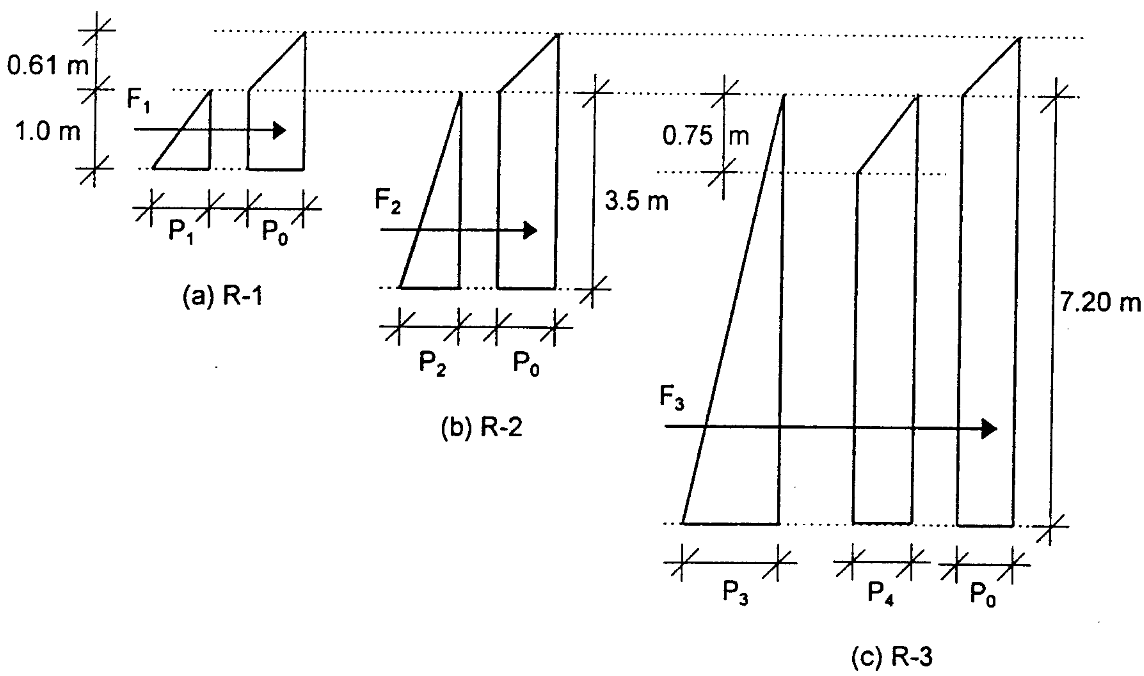


Fig. Horizontal Pressure Diagram

Earth pressure:

P_0 $= 0.333 \times 18 \times 0.61$
$= 3.656$ kN/m^2

P_1 $= 0.333 \times 18 \times 1.0$
$= 5.994$ kN/m^2

P_2 $= 0.333 \times 18 \times 3.5$
$= 20.979$ kN/m^2

P_3 $= 0.333 \times 18 \times 7.20$
$= 43.157$ kN/m^2

P_4 $= 10 \times 0.75$
$= 7.50$ kN/m^2

Earth force per linear meter:

$F_1 = (P_0 + P_1/2) \times 1.0$
$= (3.656 + 5.994/2) \times 1.0$
$= 6.653$ kN

$F_2 = (P_0 + P_2/2) \times 3.50$
$= (3.656 + 20.979/2) \times 3.50$
$= 49.51$ kN

$F_3 = (P_0 + P_4 + P_3/2) \times 7.20$
$= (3.656 + 7.5 + 43.157/2) \times 7.20$
$= 235.688$ kN

Earth force on flag portion:

FFLAG = ((6.653 + 49.51)/2) x 3.75
= 105.305 kN

Earth force on wing wall portion:

FWING = 235.688 x 4.30/2
= 506.73 kN

Total force on tie wall:

FTIE = 105.305 + 506.73
= 612.035 kN

Factored tensile force (gross) per m height of tie wall

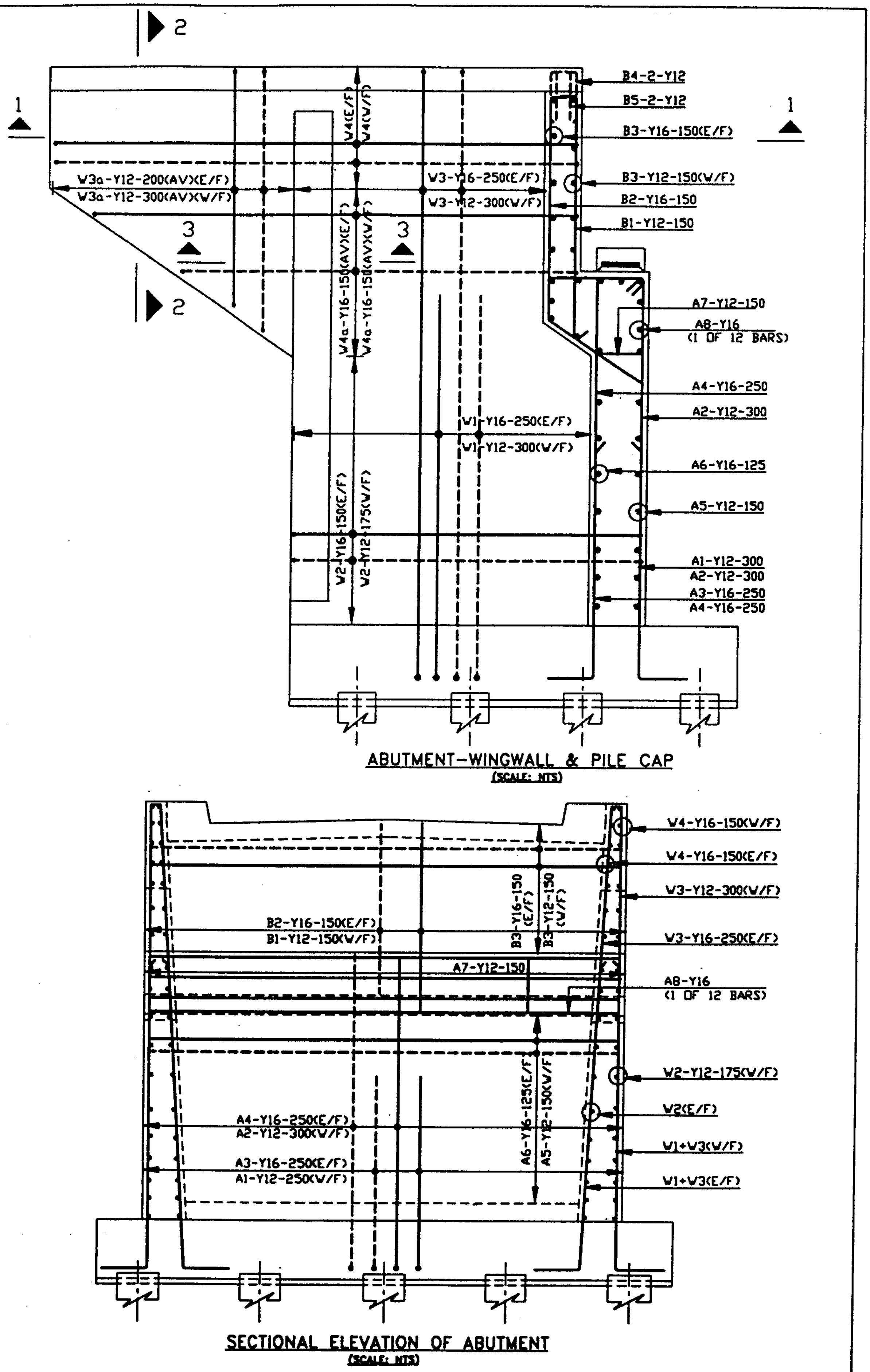
FFTIE = 1.69 x (612.035/(8.0 - 0.8 - 0.25 - 0.25))
= 154.38 kN/m

Total area of steel required
$As_{tie} = 154.38/(0.70 \times 275000)$
$= 8.0197 \times 10^{-4}\ m^2/m$

Providing **Y16**

Spacing $= ((2.01 \times 10^{-4}) \times 1000)/(8.0197 \times 10^{-4})$
= 250.63 mm C/C

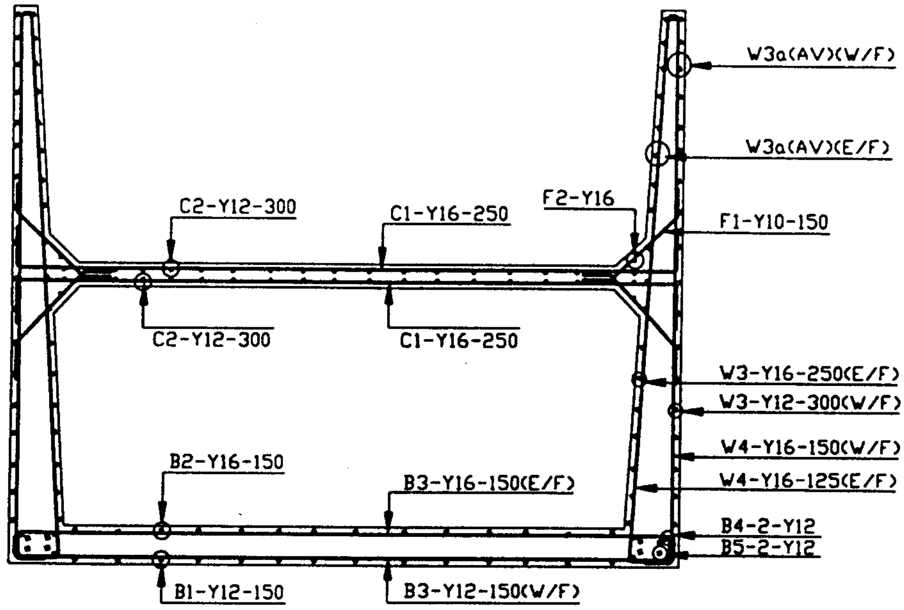
(But maximum spacing of tensile reinforcement in tie wall has been limited to 250 mm C/C.)
So, provide **Y16-250.**



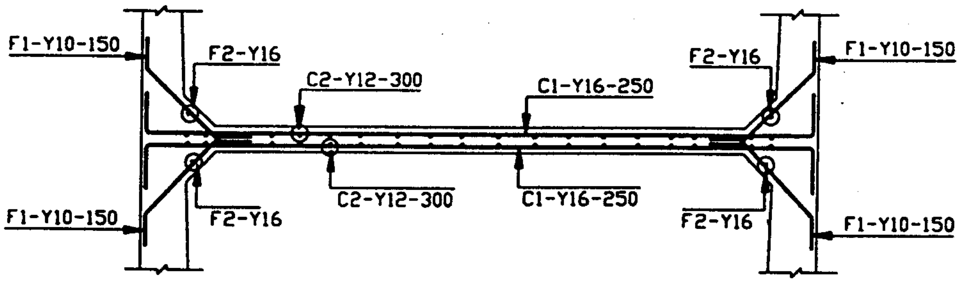
REINFORCEMENT DETAILS OF ABUTMENT AND WING WALL

FIG. 4.1

DPM-C3\LGED\ABUT 1:120



SEC. 2-2
SECTIONAL PLAN OF BACK WALL AND WING WALL
(SCALE: NTS)



TIE-WALL DETAILS (SEC. 2-2)
(SCALE: NTS)

REINFORCEMENT DETAILS OF BACK WALL, WING WALL AND TIE WALL

FIG. 4.2

DPH-C3\LGEB\ABUT1.DWG 1:120

CHAPTER 5

FOUNDATION

5.1 Pile Load Calculation and Structural Design of Pile

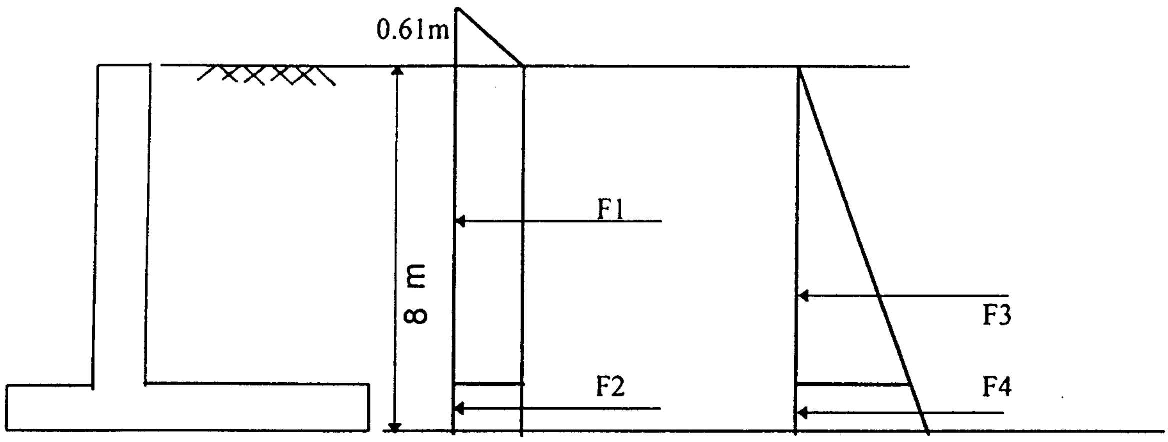
Bridge Length (c/c brg.)= 30.0 m
Abutment Height = 8.0 m
Foundation Type = F
Deck Type = IIA
Ref. Fig. Nos. = 5.1 & 5.2

Stability Analysis of Abutment-wing wall

Restoring Moment (about toe line)

Load Description	Wedge No.	Vertical Load Calculation	Weight VSL (kN)	Lever Arm (m)	Restoring Moment (kN-m)
Superstructure Dead Load		From Program DPM-PCG.For	1237.400	2.000	2474.800
Superstructure Live Load		From Program DPM-PCG.For	220.000	2.000	440.000
Abut. Wall	1	3.72x0.6x5.88x24	314.980	2.000	629.960
Seat Beam	2 minus 13	(1.125x0.9-0.5x.525x 0.35)x5.88x24	129.920	2.224	288.942
Back Wall	3	2.58x0.4x5.88x(24-19)	30.340	2.625	79.643
Approach Slab Support	4	Not Present	00.000	0.000	00.000
Approach Slab	5	Not Present	00.000	0.000	00.000
Wing Walls	6	2(7.2x4.50x.45)x(24-19)	145.800	4.550	663.390
Wing Flags	7	2(3.75x(1.0+3.5)/2)x0.3X24	121.500	8.578	1042.227
Base Slab	8	7.05x8.4x0.8x24	1137.020	3.525	4008.000
Soil	9	7.2x4.75x5.88x19	3820.820	4.675	17,862.334
Guard Wall	10	(0.25x0.3x0.15)x2x24	0.540	2.000	1.080
Tie Wall or Counterfort	11	6.95x0.25x5.88x(24-19)	51.080	6.925	353.729
Curb	12	2(0.25x8.375x0.25)x24	25.125	6.613	166.152
Surcharge		0.61x4.75x5.88x19	323.700	4.6750	1513.298
Total			**7558.225**		**29,523.555**

Overturning Moment



Lateral Pressure Diagram: (a) Due to surcharge (b) Due to soil

Considering $\phi = 30^0$, ka = 0.33

P_{SUR} = Pressure due to surcharge

P_3 = Pressure at pilecap top due to soil

P_4 = Pressure at pilecap bottom due to soil

P_{SUR}	=	0.61x0.33x19	= 3.82 kN/m^2
F1	=	3.82x7.2x5.88	= 161.72 kN
F2	=	3.82x0.8x8.4	= 25.67 kN
P3	=	0.33x19x7.2	= 45.14 kN/m^2
P4	=	0.33x19x8	= 50.16 kN/m^2
F3	=	0.5x45.14x7.2x5.88	= 955.52 kN
F4	=	0.5x(45.14+50.16)x0.8x8.4	= 320.21 kN
F_{SUR}	=	F1+F2 = 187.4 kN	
F_{SOIL}	=	F3+F4 = 1275.73 kN	
M_{SUR}	=	161.72x(0.8+7.2/2)+25.67x0.8/2	= 721.84 kN-m

M_{SOIL} = 955.52x(0.8+7.2/3)+320.21x(2x45.14+50.16)/(45.14+50.16)x0.8/3
= 3183.50 kN-m

Total Overturning Force

F_Y = $F_{SOIL} + F_{SUR}$ = 1463.13 kN

Total Overturning Moment

M_X = $M_{SUR} + M_{SOIL}$ = 3905.34 kN-m

Wind Load

Basic wind speed, V = 66.67 m/s (240 Km/hr)

Design Wind speed, V_s = $S_1\ S_2\ S_3\ V$ = 1x1x1x66.67 = 66.67 m/s

Wind Speed Factor = $(66.67/44.69)^2$ = 2.225

Height of Bearing top from pilecap bottom = 8.0-0.04-0.2-2.2 = 5.56 m

Area of Beam , Deck & Kerb exposed to wind = (2.2+0.2+0.25+0.2)x30.8/2 = 43.89 m^2

Area of Railing exposed to wind = 0.8x30.8/2x0.5 = 6.16 m^2

According to AASHTO'92 (Art. 3.15),

Pressure on structure due to wind

in Transverse direction, P_X = 50 psf = 2.395 kN/m^2

in Longitudinal direction, P_Y = 12 psf = 0.575 kN/m^2

Force due to wind, F_{X1} = 2.395x2.225(43.89+6.16) = 266.71 kN

F_{Y1} = 0.575x2.225x(43.89+6.16) = 64.03 kN

Force on live load due to wind

In Transverse direction = 100 lb/ft = 1.459 kN/m

In Longitudinal direction = 40 lb/ft = 0.584 kN/m

Force on LL, F_{X2} = 1.459x30.8/2x2.225 = 49.99 kN

F_{Y2} = 0.584x30.8/2x2.225 = 20.01 kN

Moment due to wind,

M_{XX} = $(F_{Y1}+F_{Y2})$x5.56 = 467.26 kN-m

M_{YY}	=	$(F_{X1}+F_{X2})x5.56 = 1760.85$ kN-m

Braking Force

F_Y	=	0.5x1x0.05x(9.34x30.8+80.064)x1 = 9.19 kN
M_X	=	9.19x5.56 = 51.10 kN-m

Shrinkage

Strain, ε_{SH}	=	$520x10^{-6}$ mm/mm
Shrinkage Deformation, Δ_{SH}	=	$30x520x10^{-6}x0.5 = 0.0078$ m = 7.80 mm
Bearing Size	=	250x400x52
Elastomer thickness ,tq	=	40 mm = 0.04 m
Area of Bearing, A_0	=	0.100 m^2
Elastomer Stiffness Factor, G	=	0.8 N/mm^2 = 800 kN/m^2
Shear Stiffness of Bearing , K_q	=	$A_oG/tq = 0.100x800/0.04 = 2.0x10^3$ kN/m
Force due to Shrinkage, F_Y	=	$2x2.0x10^3x0.0078 = 31.2$ kN
Moment due to Shrinkage M_X	=	31.2x5.56 = 173.47 kN-m

Temperature

Coeff. of Thermal Expansion, α	=	$12x10^{-6}/^0C$
Variation of Temperature, t	=	$(\pm)\ 35^0C$
Temp. Deformation, Δ_t	=	$30x12x10^{-6}x35x0.5 = 6.3x10^{-3}$ m
Force due to Temp., F_Y	=	N.Kq. Δt
	=	$2x2.0x10^3x6.3x10^{-3} = 25.20$ kN
Moment due to Temp.,M_X	=	25.20x5.56 = 140.11 kN-m

Stream Force

Max. mean velocity, $\bar{V}$ = 1.5 m/s

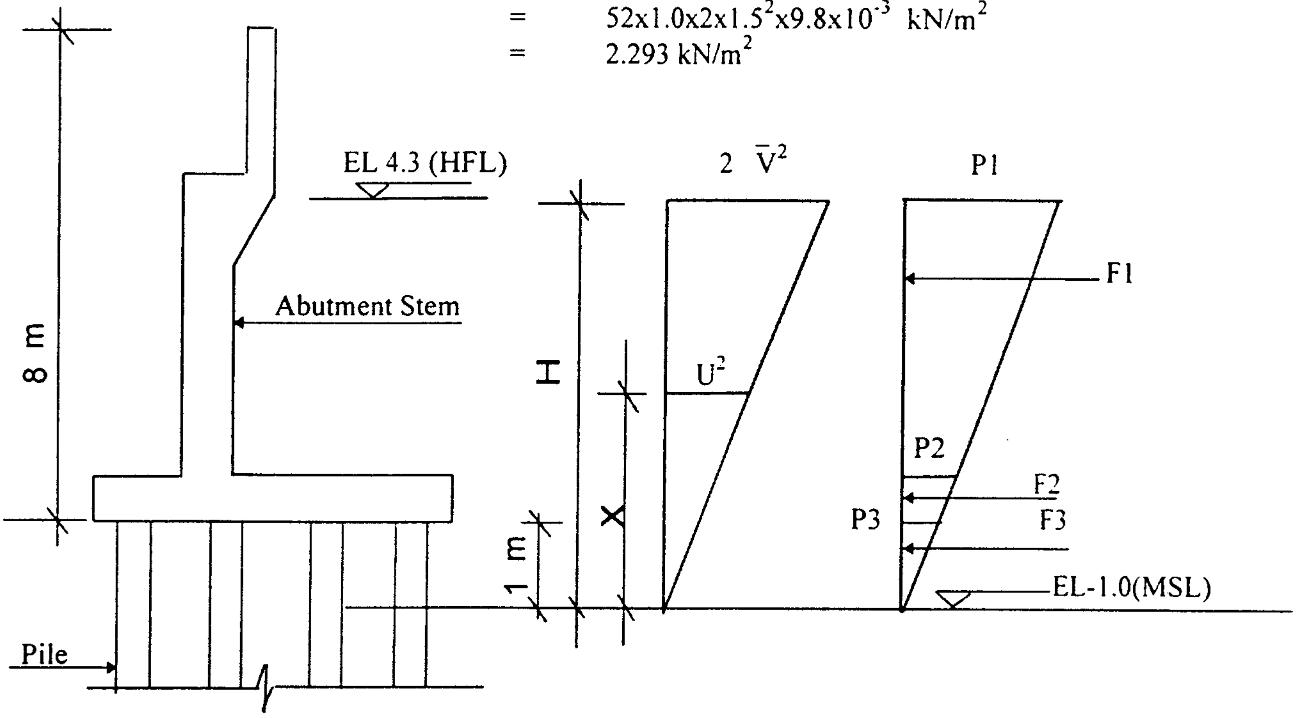
V_{max} = $\sqrt{2}\ \bar{V}$

V^2_{max} = $2\ \bar{V}^2$

P1 = $52K\,V^2_{max} = 52K\text{x}\,2\ \bar{V}^2$ = $52\text{x}1.0\text{x}2\text{x}1.5^2$ kg/m^2

= $52\text{x}1.0\text{x}2\text{x}1.5^2\text{x}9.8\text{x}10^{-3}$ kN/m^2

= 2.293 kN/m^2



$U^2 = (2\ \bar{V}^2\ .X) / H$

P2 = 2.293x1.8/5.3 = 0.779 kN/m^2

P3 = 2.293x1.0/5.3 = 0.433 kN/m^2

Length of wing wall = 4.5 m

Stream Force along X-X, F_{X1} = (2.293+0.779)/2x(4.3-0.8)x4.5xSin45^0

= 17.11 kN

F_{X2} = (0.779+0.433)/2x0.8x7.05xSin45^0

= 2.42 kN

Stream Force along Y-Y, F_{Y1} = (2.293+0.779)/2x(4.3-0.8)x5.88xCos45^0

= 22.35 kN

F_{Y2} = (0.779+0.433)/2x0.8x5.88xCos45^0

= 2.02 kN

Total Stream Force, F_X = $F_{X1} + F_{X2}$ = 19.53 kN

= $F_{Y1} + F_{Y2}$ = 24.37 kN

Moment due to Stream Force, M_X = 22.35x(2.04+0.8)+2.02x0.438
= 64.36 kN-m

M_Y = 17.11x(2.04+0.8)+2.42x0.438
= 49.65 kN-m

Earthquake

Total DL of Superstructure on Abut. = 1237.40 kN

Total DL of Abutment Wedges = 5777.125 kN

Force due to Earth quake, F_Y = (1237.40+5777.125)x0.06
= 420.87 kN

Moment due to Earth quake, M_X = (1237.40x5.56+5777.125x8.0/2)x0.06
= 1799.31 kN-m

M_Y = 1799.31x0.06
= 107.96 kN-m

Service Load on Pile : AASHTO Gr. I Loading

By examination, AASHTO GR.I loading is found critical for max. load on piles at Service Load Design (SLD)

ΣV_{SLD} = 7558.225 kN
ΣM_{RX} = 29523.555 kN-m
ΣM_{OX} = 3905.34 kN-m
ΣM_{RY} = 7558.225x8.4x0.5 = 31744.545 kN-m
ΣM_{OY} = 49.65 kN-m

Eccentricity, e_Y = 3.3- (29523.555-3905.34)/7558.225 = -0.089 m

Eccentricity, e_X = 4.2- (31744.545-49.65)/7558.225 = 0.007 m

$\Sigma(X^2)_{piles}$ = $[(4.2-0.6)^2x4+(4.2-2.4)^2x4]x2$ = 129.60 m

$\Sigma(Y^2)_{piles}$ = $[(3.3-0.6)^2x5+(3.3-2.4)^2x5]x2$ = 81.00 m

Pile Load, Q_p = $\Sigma V_{SLD}/N \pm (\Sigma V_{SLD} \times e_y \times Y) / \Sigma(Y^2)_{piles} \pm (\Sigma V_{SLD} \times e_x \times X) / \Sigma(X^2)_{piles}$

= 7558.225/20 ± (7558.225x0.089xY)/81.0 ± (7558.225x0.007xX)/129.6

= 377.91 ± 8.30Y ± 0.408X

$Q_{p(max)}$ = 377.91 + 8.30x2.7 + 0.408x3.6 = 401.79 kN

$Q_{p(min)}$ = 377.91 - 8.30x2.7 + 0.408x3.6 = 354.03 kN

Factored Load on Pile : AASHTO Gr. VII Loading

By examination, AASHTO Gr. VII loading is found critical for max. load on piles at Factored Load System (LFD)

ΣV_{LFD} = (7558.225 - 220.0 - 323.7) x 1.3 = 9118.88 kN

MRSDLX = Resist. moment about X-X due to DL of superstructure & DL of soil & abut.
= 29523.555 - 440.0 - 1513.34
= 27570.22 kN-m

MRSDLY = Resist. moment about Y-Y due to DL of superstructure & DL of soil & abut.
= (7558.225 - 543.7) x 8.4 x 0.5
= 29461.05 kN-m

ΣM_{RX} = 27570.22 x 1.0 x 1.3 = 35841.28 kN-m

ΣM_{OX} = M_{SOIL} + M_{EQ}

= (3905.34 x 1.3 + 1799.31 x 1.0)x1.3 = 8938.85 kN-m

ΣM_{RY} = 29461.05 x 1.0 x 1.3 = 38299.37 kN-m

ΣM_{OY} = M_{SF} + M_{EQ}

= (59.65 x 1.0 + 107.96 x 1.0)x1.3= 204.89 kN-m

Eccentricity, e_Y = 3.3- (35841.28 - 8938.85)/9118.88 = 0.350 m

Eccentricity, e_X = 4.2- (38299.37 -204.89)/9118.88 = 0.022 m

$Q_{P(LFD)}$ = $\Sigma V_{LFD}/N \pm (\Sigma V_{LFD} \times e_y \times Y) / \Sigma(Y^2)_{piles} \pm (\Sigma V_{LFD} \times e_x \times X) / \Sigma(X^2)_{piles}$

= 9118.88/20 ± (9118.88x0.35xY)/81.0 ± (9118.88x0.022xX)/129.6

= 455.944 ± 39.4Y ± 1.548X

$Q_{p(max)}$ = 455.944 + 39.4x2.7 + 1.548x3.6 = 567.90 kN

$Q_{p(min)}$ = 455.944 - 39.4x2.7 - 1.548x3.6 = 343.99 kN

Structural Design of Pile

Total Horizontal Force (LFD), ΣF_Y = (1463.13x1.3 + 420.87 x1.0)x1.3 = 3019.82 kN

Max. Horizontal Force per pile, H = 3019.82/20 = 151.00 kN

Assumed Depth of Scour from pilecap bottom, Ds = 1.0 m

Modulus of Elasticity of pile concrete, E = 21.152×10^6 kN/m^2

Coefficient of subgrade modulus, η_h = 1500 kN/m^3

Now for 600mm dia. pile having Y32 reinforcing bars,

hs/h = (600-2x75-2x10-32)/600 = 0.663

Moment of inertia of pile, I = $\pi (0.6)^4/64$ = 0.0064 m^4

Stiffness Factor, $T = [\,E\,I/\eta_h\,]^{0.2}$
$= [21.152 \times 10^6 \times 0.0064/1500]^{0.2}$ = 2.46

Min. Length of pile, L_p = 4T = 4x2.46 = 9.84 m

Moment on pile, M = (1.8T+Ds)xHx0.5
= (1.8x2.46+1.0)x151.0x0.5 = 409.81 kN-m

$M/h^3 = 409.81/0.6^3$ = 1897.3 kN/m^2 = 1.897 N/mm^2

$N/h^2 = 343.99/0.6^2$ = 955.53 kN/m^2 = 0.956 N/mm^2

Now using CP110 : Part 3 : 1972 of British Standard Code of Practice (Chart 93 & 94)

For hs/h = 0.6, Required Steel Area, As = 4.0 %

For hs/h = 0.7, Required Steel Area, As = 3.5 %

By interpolation, For hs/h = 0.663 Required Steel Area, As = 3.69 %

So we can provide top cage reinf. 14-Y32 ,which will give As = 3.98% > 3.69% (OK)

and lower cage reinf. 7-Y25, which will give As = 1.2% > 1.0% (OK)

5.2 Structural Design of Pile Cap

Pile Arrangement : Rectangular
Foundation Type : F
Abutment Height : 8.0 m
Design Method : LFD
Ref. Fig. No. : 5.2

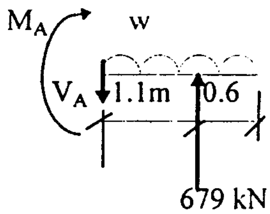
Analysis Along Y-Y (Traffic Direction)

Toe Side : Bending Moment

Strip Width = 0.6 + 1.8/2 = 1.5 m
Strip Length = 1.7 m

Pile Reaction in Row 1, PR(1) = 679 kN [From Program DPM-PCAP.FOR]
Pile Reaction in Row 1, PR(2) = 636 kN
Pile Reaction in Row 1, PR(3) = 593 kN
Pile Reaction in Row 1, PR(1) = 549 kN

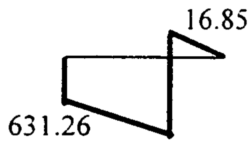
UDL due to Pile Cap self weight,



$w = (0.8 \times 24 \times 1.5) \times 0.75 \times 1.3$

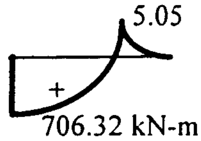
$= 28.08$ kN/m

$M_A = 679 \times 1.1 - 28.08 \times 1.7^2 \times 0.5$
$= 706.32$ kN-m
$= 470.88$ kN-m/m



Cracking Moment, Mcr

Modulus of rupture,
$\sigma_{cr} = 7.5 \sqrt{f_c'}$ where f_c' is in psi [AASHTO'92, Art.8.15.2.1.1]
or, $= 19.70 \sqrt{f_c'}$ where f_c' is in kN/m²
$= 19.70\sqrt{20000} = 2786.00$ kN/m²



$I_g = 1 \times (0.8)^3/12 = 0.0427\ m^4$

$$M_{cr} = \frac{\sigma_{cr} . I_g}{C} = \frac{2786.00 \times 0.0427}{0.8/2}$$ [AASHTO'92, Art.8.13.3]

$= 297.41$ kN-m/m

Min. Flexural Strength = $1.2\ M_{cr}$ = 356.89 kN-m/m

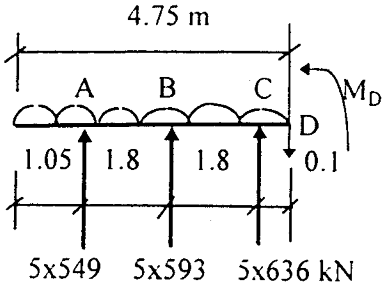
Since External Moment, $M_A > 1.2$ Mcr . So Design Moment = M_A = 470.88 kN-m/m

Provide Y20 - 100 for which As = 3141.6 mm² ; $a = As.fy / 0.85\ f_c' b = 50.82$ mm

So, Design Strength , $\varphi Mn = 0.9\ As.fy\ (d - a/2) = 0.9 \times 3141.6 \times 275 \times (650 - 50.82/2)$

$= 485.65$ kN-m/m $> M_A$ (OK)

Heel Side : Bending Moment



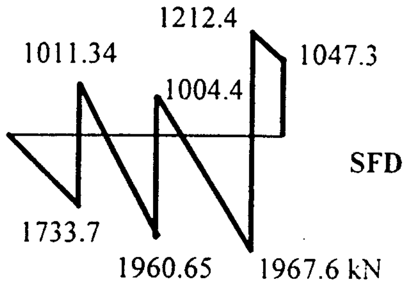
Strip width = 8.40 m

Strip Length = 4.75 m

UDL due to self wt. and soil,

$w \quad = \quad (0.8 \times 24 \times 0.75 + 7.2 \times 19 \times 1.0)1.3 \times 8.4$

$\quad = \quad 1651.104$ kN/m



$M_A \quad = \quad - w\,(1.05)^2/2 = 910.17$ kN-m

$M_B \quad = \quad - w\,(1.8 + 1.05)^2/2 + 549 \times 5 \times 1.8$

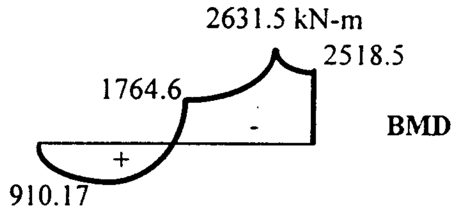
$\quad = \quad - 1764.55$ kN-m

$M_C \quad = \quad - w\,(1.05 + 1.8 \times 2)^2/2 + 549 \times 5 \times 3.6 + 593 \times 5 \times 1.8$

$\quad = \quad - 2631.50$ kN-m

$M_D \quad = \quad - w\,(4.75)^2/2 + 549 \times 5 \times 3.7 + 593 \times 5 \times 1.9 + 636 \times 5 \times 0.1$

$\quad = \quad - 2518.52$ kN-m



Max. -ve external moment about X-X = Mc = - 2631.50 kN-m

Since external moment ,Mc = -2631.50/8.4 = -313.27 kN-m/m < 1.2 Mcr

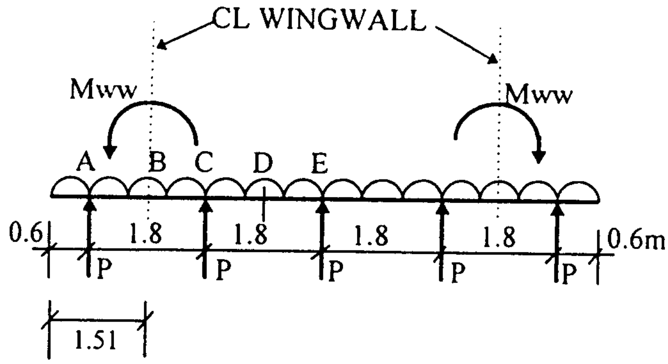
So Design Moment = 1.2 Mcr = 356.89 kN-m/m

Provide Y20 - 125 for which Design Strength , φMn = 391.68 kN-m/m > 1.2Mcr (OK)

Analysis Along X - X (Perpendicular to Traffic Direction)

Heel Side : Bending Moment

Width of Strip = 4.75 m
Length of Strip = 8.40 m
Pile Load, P = 636 + 593 + 549
= 1778 kN

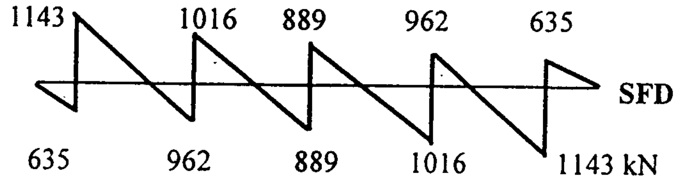


Balanced UDL = 1778 x 5/8.4
= 1058.33 kN/m

Moment on Pilecap from wingwall :

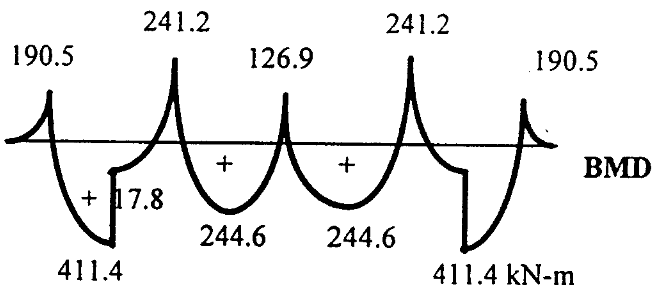
Ratio of Length & Height,

lx/ly = 4.5/6.95 = 0.65



Using moment coeff. from Reynold's Hand Book (Table 53), Moment on pilecap due to soil pressure on wingwall

= $[0.019x(0.33 \times 19 \times 7.2)\ 7.2^2] \times 1.3 \times 1.3$
= 75.15 kN-m/m



Using moment coeff. from 'Moments and Reactions for Rectangular Plates' by W.T. Moody(fig. 1), Moment due to surcharge

= $[0.023x(0.33 \times 19 \times 0.61) \times 7.2^2] \times 1.3 \times 1.3$
= 7.71 kN-m/m

Total moment on pilecap from Wingwall = (75.15+7.71)x4.75 = 393.58 kN-m

M_A = - 1058.33 x $(0.6)^2/2$ = -190.50 kN-m

M_{BL} = - 1058.33 x $1.51^2/2$ + 1778 x 0.91 = 411.43 kN-m

M_{BR} = 411.43 - 393.58 = 17.85 kN-m

M_C = - 1058.33 x $2.4^2/2$ -393.58 + 1778 x 1.8 = - 241.17 kN-m

M_D = - 1058.33 x $3.3^2/2$ -393.58 + 1778 x 2 x 1.8 = 244.61 kN-m

M_E = - 1058.33 x $4.2^2/2$ -393.58 + 1778 x 3 x 1.8 = - 126.85 kN-m

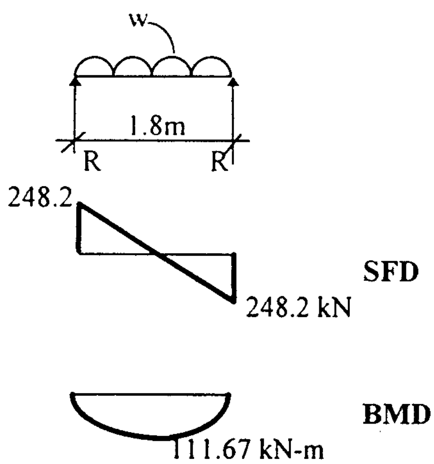
Max. -ve moment about Y-Y = Mc = - 241.17 kN-m

Max. -ve external moment = 241.17/4.75 = - 50.78 kN-m/m < 1.2 Mcr

So -ve Design Moment = 1.2 Mcr = - 356.89 kN-m/m

Provide Y20 - 125 for which Design Strength , φM_n = 391.68 kN-m/m > 1.2Mcr (OK)

+ve moment on Pile Cap = M_D = 411.4 /4.75 = 86.61 kN-m/m



Considering Piles as supports with max. downward factored load, we get UDL

w = [0.8 x 24x1.0 + (7.2 + 0.61) x 19x1.3] x 1.3

= 275.74 kN/m

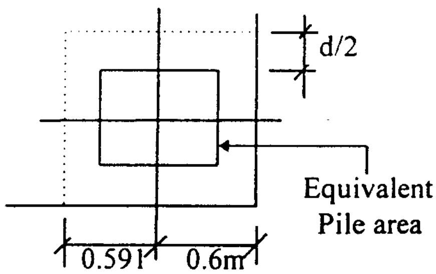
+ve Moment, M = w x $(1.8)^2/8$ = 111.67 kN-m/m

Max.+ve Moment about Y-Y = 111.67 kN-m/m < 1.2 Mcr

So Design Moment = 1.2 Mcr = 356.89 kN-m/m

Provide Y20 - 125 for which Design Strength , φM_n = 391.68 kN-m/m > 1.2Mcr (OK)

<u>**Punching Shear Check for a Corner Pile**</u>

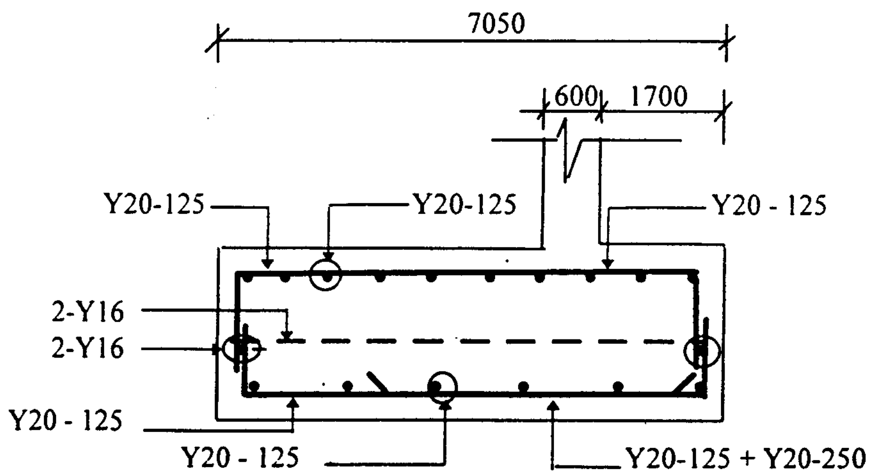


Pile dia. = 0.6 m
Pile X-sec. Area = 0.2827 m^2
Length of each side of an equivalent Square = 0.532 m
Effective depth of pile cap, d = 0.65 m.

Effective Perimeter for Punching Shear, b_0 = (0.591 + 0.6) x 2
= 2.382 m

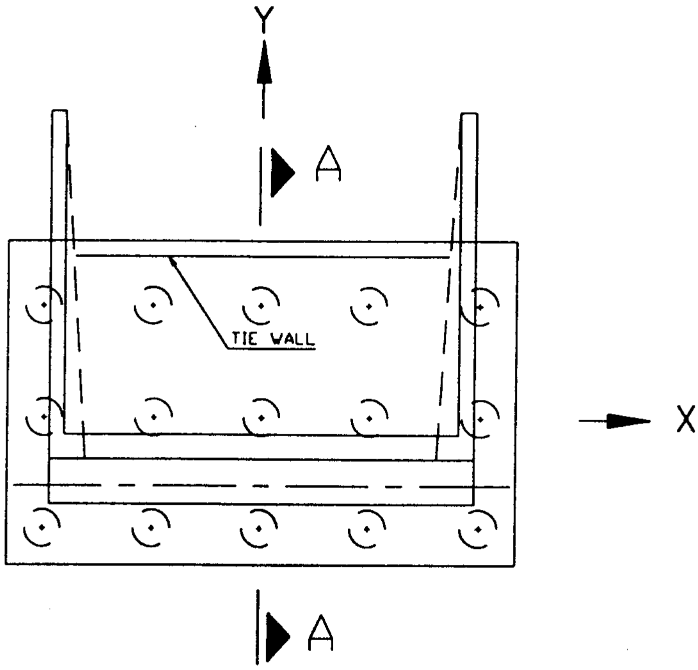
Allowable shear Strees,
$V_c = \varphi \times 0.332 \sqrt{f'c}\, b_0 d$ = 0.85 x 0.332 x $\sqrt{20}$ x 2382 x 650 N = 1954.01 kN > 679 kN

Since Vc > PR_{MAX} , So Punching Shear is OK.

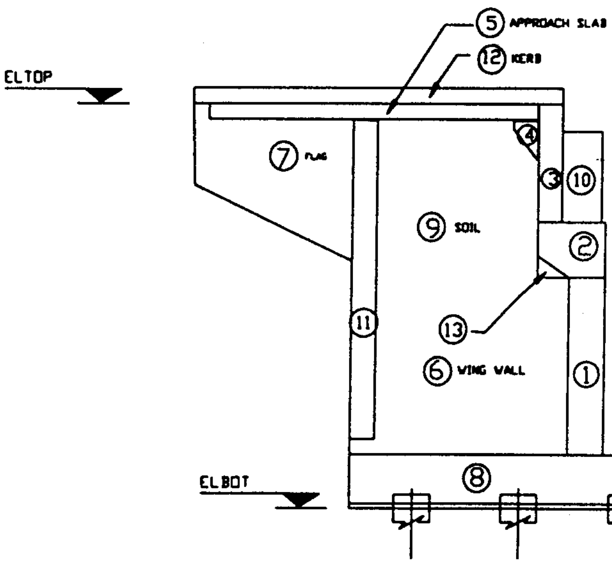


Designed Section of Pile Cap

Web Copy



PLAN OF ABUTMENT-WING WALL WITH PILE CAP

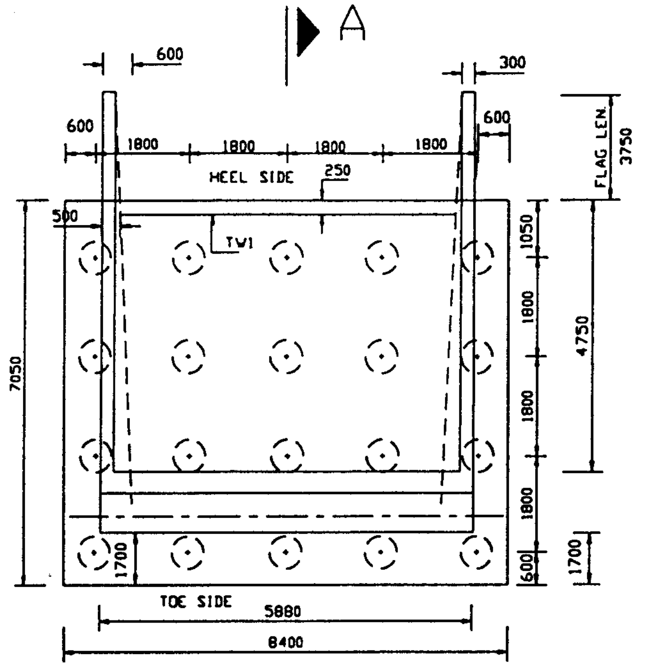


SECTION A-A

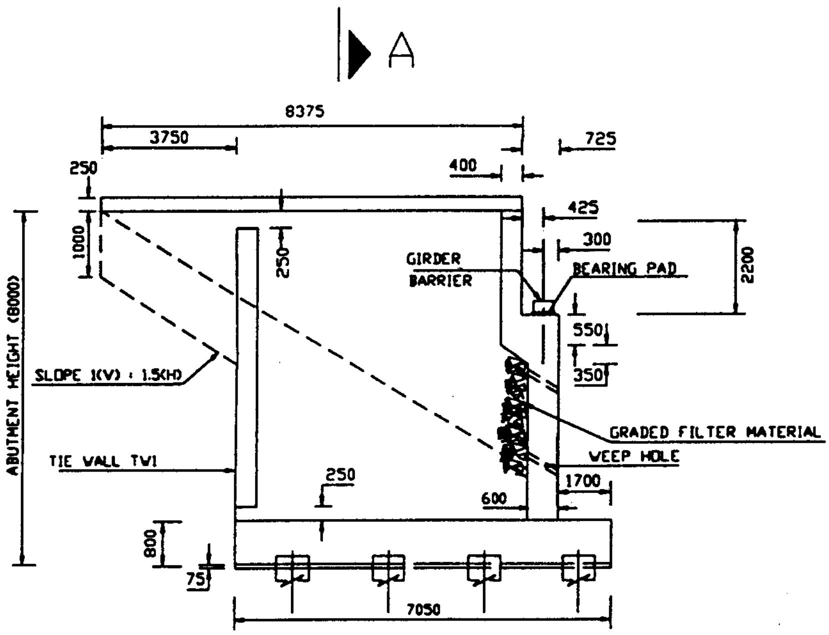
TYPICAL SKETCHES SHOWING WEDGE NUMBERS OF SUBSTRUCTURE AND FOUNDATION

FIG. 5.1

DPM-C4 >FORILOAD 1:90



PLAN OF ABUTMENT AND FOUNDATION
(FOUNDATION TYPE: F)



SECTION A-A

CONCRETE OUTLINE DETAILS
OF ABUTMENT AND FOUNDATION
FOUNDATION TYPE: F ; ABUT. HEIGHT: 8.0 m
GIRDER LENGTH: 30.0 m : DECK TYPE: IIA

FIG. 5.2