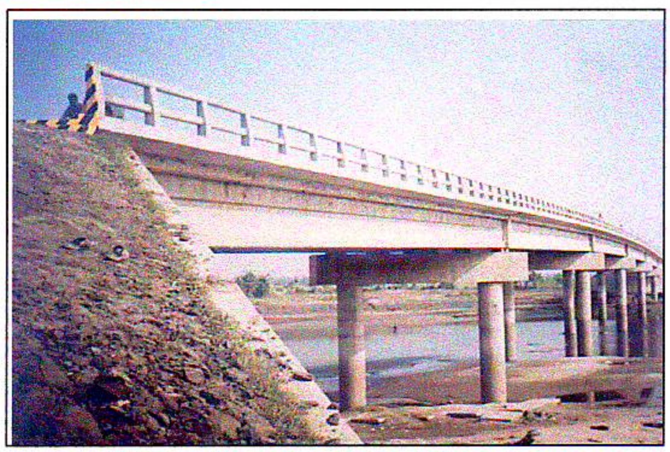
GOVERNMENT OF PEOPLE'S REPUBLIC OF BANGLADESH

MINISTRY OF LOCAL GOVERNMENT & RURAL DEVELOPMENT
LOCAL GOVERNMENT ENGINEERING DEPARTMENT (LGED)

ROAD STRUCTURES MANUAL FOR DOUBLE LANE BRIDGES

PART-C DESIGN EXAMPLES OF BRIDGES,CULVERTS AND SLOPE PROTECTION WORKS



FUNDED BY:
JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)
THROUGH:
RURAL DEVELOPMENT ENGINEERING CENTER (RDEC)

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DATE: NOVEMBER 2008

Manual on RC Girder & PC Girder Bridges
Part C- Design Examples

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CHAPTER-1

SUPERSTRUCTURE

1.1 RC DECK OF CONCRETE BRIDGE

The design example comprises an 8.26 m wide deck (Type I) with 4-girder arrangement. The span of the girders is 25.00 m(c/c brg), and the overall girder length is 25.65 m. The girders are spaced at 1.85 m (c/c girder). Both deck and girder concrete shall be of 28 days crushing cylinder strength f'c = 25 MPa and reinforcing steel shall be of yield strength fy = 400 MPa.

Fig. 1.1.1 shows the deck cross-section showing the concrete outline details.

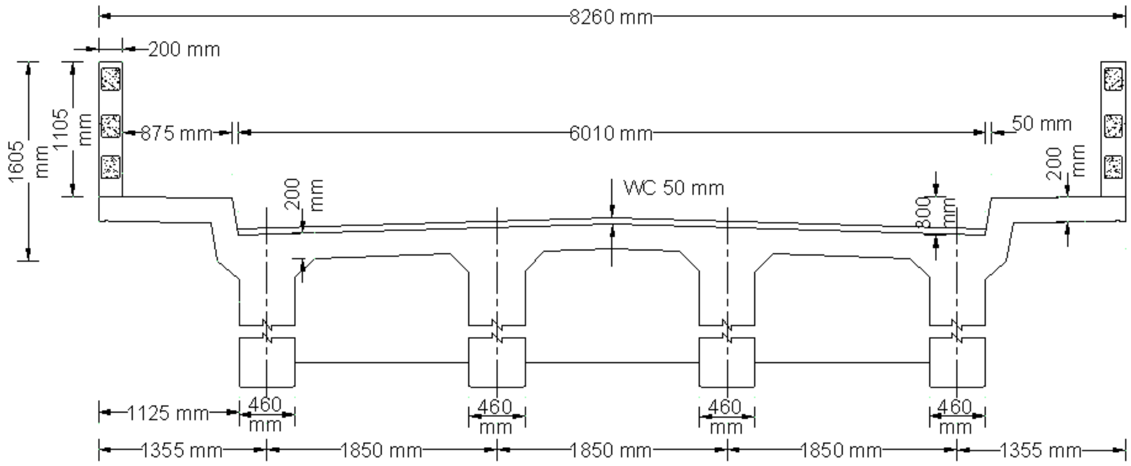


Fig 1.1.1 Cross section of deck slab

1.1.1 Deck slab

1.1.1.1 Design procedure

The structural design of the deck slab depends mainly on spacing of the main and cross girders and the cantilever overhang at either end of the deck. In general the following analysis and design methods are recommended.

1. AASHTO "Approximate Methods of Analysis" (Ref. AASHTO 07, Art.4.6.2)
2. AASHTO "Refined Methods of Analysis" (Ref. AASHTO 07, Art.4.6.3)
3. AASHTO "Empirical Design Methods" (Ref. AASHTO 07, Art.9.7.2)
4. Yield line Analysis
5. Hillerborg strip method
6. Purcher's chart

Here, "Approximate Methods of Analysis", based on AASHTO'07 has been followed.

1.1.1.2 Geometrical Data and material properties

Slab	Thickness of deck	= 200 mm = 0.2 m.
	Thickness of WC	= 50 mm = 0.05 m.
	Side walk width	= 875 mm =0.875 m.
	Overall girder length	= 25650 mm=25.65m.
Railing	Cross section of rail post	= 200 mm x 200 mm.
	Height of rail post above deck	= 1105 mm = 1.105 m.
	Cross section of rail bar	= 185 mm x 150 mm.
	Rail post spacing	= 1580 mm = 1.58 m.
	Number of rail posts	= 18
Girder	c/c girder spacing	= 1850 mm =1.85 m.
	Height of Girder web	= 1800 mm = 1.8 m.
	Girder width	= 460 mm = 0.46 m.
Material properties	Concrete strength. fc′	= 25 MPa.
	Yield strength of steel, fy	= 400 MPa.
	Unit wt. of concrete	= 24 kN/m^3.
	Unit wt. of wearing course	= 23 kN/m^3.

1.1.3 Structural analysis

1.1.3.1 Interior slab

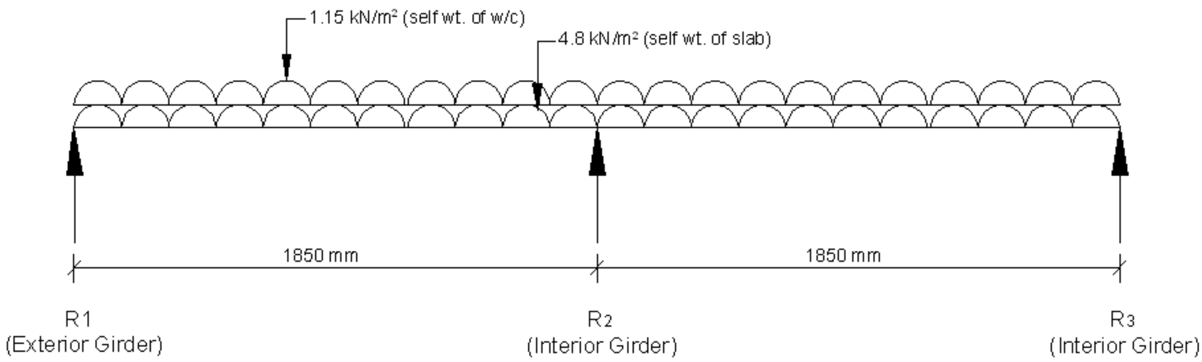


Fig 1.1.2 c/c Girder Spacing Loading Diagram

(**Note:** Deck overhang and its loading diagram is shown separately in Fig. 1.1.3)

Dead load:

Self wt. of deck slab = 24 x 0.2 = 4.8 kN/m^2.
Self wt. of wearing course = 23 x 0.05 = 1.15 kN/m^2.

+ve Moment due to dead load

+ve MDLIS1 (due to deck slab) = $4.8 \times 1.85^2 / 8 = 2.053$ kN-mlm.

+ve MDLIS2 (due to WC) = $1.15 \times 1.85^2 / 8 = 0.492$ kN-mlm.

Live load:

Vehicular live loading on roadways of bridge deck is designed by vehicle type HL-93, truck loading where, wheel load, P = 72.5 kN, plus lane loading 9.30 kN/lm of lane witdth. Lane width is considered 3.00m.

To get the load per unit width of equivalent strip, total load on one design traffic lane is divided by calculated strip width.

Distribution width (for +ve moment) = 660 + 0.55 S
(Ref. AASHTO 07, Art.4.6.2.1)

Strip width (for -ve moment) = 1220 + 0.25S
(Ref. AASHTO 07, Art.4.6.2.1)

Here, S = c/c spacing of girder = 1850 mm = 1.85 m.

Distribution width (for +ve moment) = 660 + 0.55 x 1850 = 1678 mm
= 1.678 m.

Strip width (for -ve moment) = 1220 + 0.25 x 1850 = 1683 mm = 1.683 m

Dynamic load allowance, IM = 33%
(Ref. AASHTO 07, Art.4.6.2.1)
= 0.33

+ve moment due to live load at interior span:

$$MLLIS = \{(Pwheel / B_{strip}) \times S/8\} + \{(9.3/3.0) \times (S^2/8)\}$$

(Ref. AASHTO 07, Art.4.6.2.1)

$$= \{(72.5/1.678) \times (1.85/8)\} + \{(9.3/3.0) \times (1.85^2/8\}$$

$$= 11.32 \text{ kN-m/lm.}$$

Total factored moment for interior span (Strength-i):

$$MPOS = 1.25 \times MDLIS1 + 1.5 \times MDLIS2 + 1.75 \times MDLIS\ (1 + IM)$$

$$= 1.25 \times 2.053 + 1.5 \times 0.492 + 1.75 \times 11.32\ (1+0.33)$$

$$= 29.65 \text{ kN-m/lm.}$$

1.1.3.2 Deck overhang

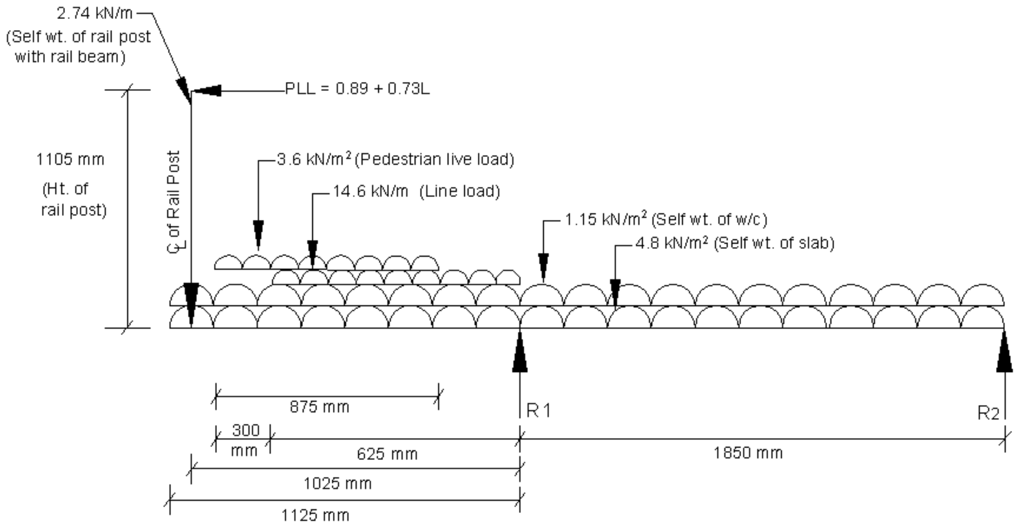


Fig 1.1.3 Loading Diagram Deck Overhang.

Note: 875 mm = Side walk Width
625 mm = Distributed Overhang Line Load Width
1025 mm = Distance between CL of Rail Post and edge of Girder
1355 mm = Distance between edge of Girder and edge of Sidewalk

Dead load: Self wt. of rail post with rail beam = (24 x 18 x 0.2 x 1.105 x 0.2)/ 25.65 + 3 x 0.15 x 0.185 x 24
= 2.74 kN/lm.

- ve moment due to dead load at overhang part:

Here,

Distance from edge of girder to edge of sidewalk = 1125 mm
= 1.125 m.
Distance from edge of girder to CL of rail post = 1025 mm
= 1.025 m.

Now,

-ve MDL overhang1 = $2.74 \times 1.025 + (4.8 \times 1.125^2) / 2$
= 5.85 kN-m/lm. (due to deck slab)

-ve MDL overhang2 = $1.15 \times 1.125^2/2 = 0.728$ kN-m/lm.
(due to WC)

Live load:

1. For deck overhang < 1800 mm.

   Line load = 14.6 kN/m, located at 300 mm from the face of railing.
   (Ref. AASHTO 07, Art.3.6.1.3.4)

2. Concentrated design horizontal live load on each post,

   $P_{LL} = 0.89 + 0.73L$ (Ref. AASHTO 07, Art.13.8.2)

   Here, L = Post spacing = 1580 mm = 1.58 m.

$P_{LL} = 0.89 + 0.73 \times 1.58 = 2.04$ kN/m.

3. Bridge pedestrian load = 3.6×10^{-3} N/m^2 = 3.6 kN/m^2
(Ref. AASHTO 07, Art.13.8.2)

-ve moment due to dead load at overhang part:

Here,

Rail post height = 1105 mm.

Slab thickness = 200 mm.

Curb height = 300 mm.

Center of girder to top of rail post height = 1105 + 200 +300

= 1605 mm.

= 1.605 m.

Distance of distributed line load on overhang = 625 mm =0.625 m.

Side walk width = 875 mm = 0.875 m.

-ve MLL overhang = $(14.6 \times 0.625^2)/2 + 3.6 \times 0.875 \times (1.125-0.2)/2$
$+ 2.04 \times 1.61$
= 7.59 kN-mlm.

Total factored moment for deck overhang (Strength-I):

MNEG = 1.25 x (-ve MDL overhang1) + 1.5 x (-ve MDL overhang2)
+ (-ve MLL overhang)
= 1.25 x 5.85 + 1.5 x 0.728 + 1.75 x 7.59
= 21.69 kN-m/m.

Design moment: +ve MDES = MPOS = 29.65 kN-mlm.
-ve MDES = MNEG = 21.69 kN-mlm.

1.1.4 Provision of reinforcement:

1.1.4.1 +ve reinforcement:

Here,

Slab thickness = 200 mm.

Clear cover = 50 mm.

Distance of reinforcement =1.5 x 12 = 18 mm.

It is proposed to use T12-150 as +ve reinforcement for interior span.

Here,

Area of reinforcement = 113 mm^2.

A_S =113 x 1000/150 = 753.3 mm^2.

Lever arm factor, a = $(A_S f_y) / (0.85 f_c' b)$

= (753.3 x 400) / (0.85 x 25 x 1000)

= 14.18 mm = 0.1418 m

Effective depth, d =200-50-18 = 132 mm =0.132 m.

Moment capacity, $\varphi M_n = 0.9 A_S f_y (d-a/2)$

= 0.9 x 753.3 x 400 x {132-(14.18/2)}x 10^{-6}

= 33.88 kN-m/m.

Ultimate +ve MDES, 29.65 kN-m/m. < φM_n, 33.88 kN-m/m.

OK

1.1.4.2 -ve reinforcement:

Here,

-ve MDES = MNEG = 21.69 kN-m/m.

Ultimate -ve MDES, 21.69 kN-m/m. < φM_n, 33.88 kN-m/m.

OK

So, to obtain above strength T12-150 is provided as –ve reinforcement.

1.1.4.3 Cracking moment:

For flexural member, AASHTO 07 requires that $\varphi M_n > M_F$

Modulus of rupture of concrete, $f_r = 0.52\sqrt{f_c'}$ =0.52 x $\sqrt{25}$ = 2.6 MPa.

(Ref. AASHTO 07, Art.5.4.2.6)

Moment of inertia, $I_g = bh^3/12 = 1\times0.2^3/12 = 0.0007\ mm^4$.

Here,

Distance from neutral axis to extreme tension fiber, y_t = 100 mm

Cracking moment, $M_{cr} = f_r I_g / y_t$ (Ref. AASHTO 07, Art.5.7.3.6.2)

= (2.6 x 0.0007 x $10^{6})$/100

= 18.2 kN-m

Minimum flexural strength, $M_F = 1.2\ M_{cr}$ =1.2 x 18.2 = 21.84 kN-m.

So, $\varphi M_n > M_F$

OK

1.1.4.4 Temperature & shrinkage reinforcement:

Here, minimum reinforcement required is

A_{st} = 0.003bh

= 0.003 x 1000 x 200/2 = 300 mm^2. (Ref. AASHTO 07, Art.5.6.3.6)

Assured Reinf. is T12-200

= 0.003 x 1000 x 200/2 = 300 mm^2.

As_{prov} = 113 x 1000/200 = 565 $mm^2 > A_{st}$=300mm^2

T12-200 provided as temperature & shrinkage reinforcement on the top layer is OK

1.1.4.5 Distribution reinforcement:

The distribution reinforcement parallel to the traffic is required,

$A_{sd} = 3840/\sqrt{S} \leq 67\%$

(Ref. AASHTO 2007, Art.9.7.3.2)

Effective span of deck slab, S = 1850-460 = 1390 mm = 1.39 m.
So, required distribution reinf. A_{sd} =504.73 m^2
$A_{sd} = 3840/\sqrt{1390}$ =103 mm^2; or
67% of A_S = 0.67 x 753.3 = 504.73 mm^2 whichever is greater .
So,
$A_{sd} \leq$ 67% of $A_{S.}$
Using T12 bar required spacing = (113 x 1000)/504.73
= 223.88 mm.
So, T12-200 is provided as distribution reinforcement on the bottom layer. OK

1.2 STRUCTURAL DESIGN OF RC GIRDER

1.2.1 Introduction

The design example demonstrates the design of a 25.0 m (c/c brg) span reinforced concrete (RC) girder. The deck cross section is the same as shown in Fig. 1.1. This comprises 200mm thick RC deck slab, 50 mm wearing course and cast in situ railing is considered. This example illustrates in detail the design of typical interior and exterior girders. The design is accomplished in accordance with the AASHTO LRFD Bridge Design Specification 07.

Vehicular live loading on the road ways of bridges or incidental structures, designated HL-93, and shall consist of combination of the:

- Design truck similar to HS20-44 of the previous AASHTO Bridge Standards or design tandem of 110000N axles spaced 1200mm apart in addition with
  (Ref. AASHTO- 07, Article 3.6.1.2.1)
- Deign lane load consist of 9.3 N/mm uniformly distributed in longitudinal direction
  (Ref. AASHTO-2007, Article 3.6.1.2.1)

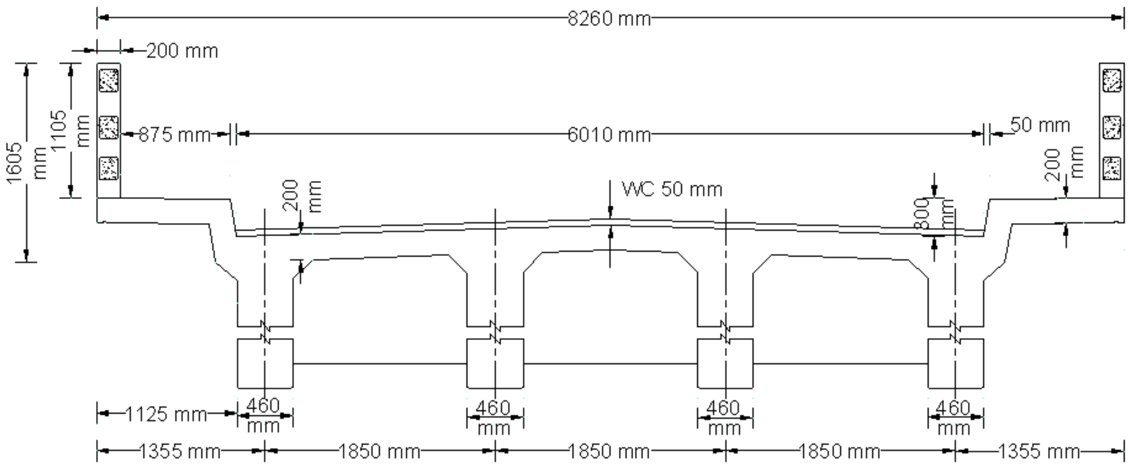


Fig 1.2.1 Cross Section of deck slab (Reproduce of Fig 1.1.1)

1.2.2 Geometrical data

Span length	:	25.0 m
Thickness	:	0.2 m
Wearing course, WC	:	.05 m
Rail Post height	:	1.1 m
Rail Post Width	:	0.2 m
C/C Rail		
Rail Post spacing	:	1.58 m
Rail beam x-section	:	0.185 m x 0.15 m
Side walk height	:	0.2 m
Side walk width	:	0.875 m
Curb height	:	0.3 m
Girder height	:	1.8 m

Girder Width	:	0.46 m
C/C Girder Spacing	:	1.85 m
X-girder height	:	1.3 m
X-girder Width	:	0.375 m

1.2.3 Material Specifications:

Concrete strength, fc	= 25 MPa
Yield Strength of Reinforcing steel, fy	= 400 MPa
Unit wt of concrete	= 24 kN/m^3
Unit wt of wearing course	= 23 kN/m^3

1.2.4 Calculation of Centroid & Moment of Inertia of Girder about X-X axis

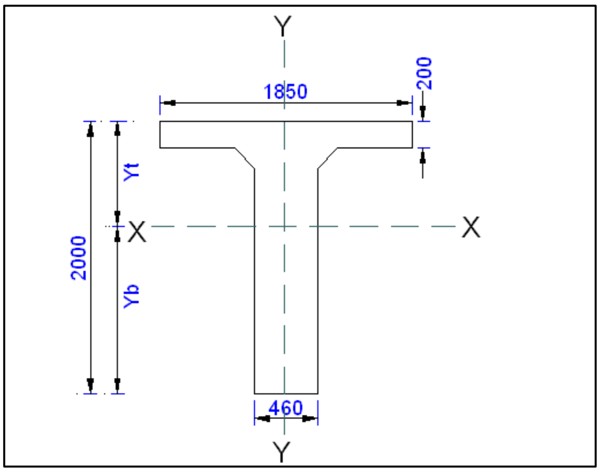


Fig 1.2.2 Centroid about X-X axis

Table: Showing Section Properties of Girder

Component	b	d	A	y	Ay	y_bNA	I	ybar	A(ybar)^2
	mm	mm	mm2	mm	mm3	mm	mm4	mm	mm4
1	1850	200	3.70E+05	1900	7.03E+08		1.23E+09	681.18	1.72E+11
2	460	1800	8.28E+05	900	7.45E+08	1228.43	2.24E+11	318.82	8.42E+10
3	150	150	2.25E+04	1750	3.94E+07		2.81E+07	531.18	6.35E+09
		∑A =	1.243E+06	∑ Ay =	1.526E+09	∑ I =	2.25E+11	$\sum A(ybar)^2$ =	2.62E+11

y_b = distance from centroid to the extreme bottom fiber of girder.

$y_b = \sum Ay / \sum A = 1228.43$ mm

y_t = distance from centroid to extreme top fiber of girder = 771.57 mm

I = moment of inertia of composite section = ∑ I +∑A(ybar)^2
$= 4.86 \times 10^{11}$ mm^4

S_b = section modulus for the extreme bottom fiber = $I / y_b = 3.97 \times 10^8$ mm^4

S_t = section modulus for the extreme top fiber = $I / y_t = 6.29 \times 10^8$ mm^4

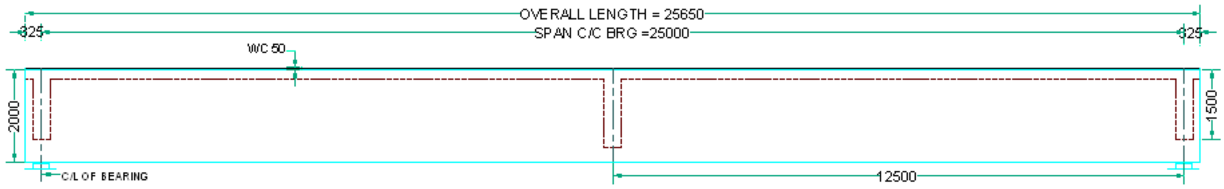


Fig. 1.2.3: Long section of girder

1.2.5 Live load distribution factor for typical interior beam:

Live load bending moment and shear force are determined by using the simplified distribution factor formulas (AASHTO 07, Art-4.6.2.2). To use the simplified live load distribution factor formula the following conditions apply:

- Width of slab is constant **OK**
- Number of girder, $N_b \geq 4$ ($N_b = 4$) **OK**
- Beams are parallel and of same stiffness **OK**
- Road way part of overhang, $d_e \leq 910$ mm (d_e=0) **OK**
- Curvature is less than 4^0 (curvature= 0˚) (Ref. AASHTO 07 table 4.6.1.2.1-1) **OK**

Number of design lanes = 2

1.2.5.1 Distribution factor for bending moment

For two or more design lane loaded,
For interior beam
$DFM = 0.075 + (S/2900)^{0.6} (S/L)^{0.2} (K_g/Lts^3)^{0.1}$
where,

	DFM	=	Distribution factor for interior beam
$1100 \leq S \leq 4900$	S	=	Beam/girder spacing, mm = 1850mm
$110 \leq t_s \leq 300$	t_s	=	Thickness of deck, mm = 200mm
$6000 \leq L \leq 73000$	L	=	Length of girder, mm = 25650mm
$N_b \geq 4$	K_g	=	$n(I + Ae_g^2)$= longitudinal stiffness parameter
$4 \times 10^9 \leq K_g \leq 3 \times 10^{12}$			in which $n = E_b / E_d = 1/1 = 1$, So, n = 1
	e_g	=	Distance between the cg of beam and cg of deck
		=	1000mm

Here, concrete strength for both beams and deck are $f\acute{c} = 25$ MPa
Here, cylinder strength of beam = cylinder strength of deck = 25 MPa
A = Cross-sectional area of girder, $mm^2 = 460 \times 1800 = 828000\ mm^2$
I = Moment of inertia of beam, $mm^4 = 2.24 \times 10^{11} = mm^4 = 2.24 \times 10^{11} = mm^4$

Thus,
$K_g = n(1+aeg^2) = 1\,(2.24 \times 10^{11} + 828000 \times 1000^2)$
$= 1.05 \times 10^{12}\ mm^4$

Therefore,

$DFM = 0.075 + (1850/2900)^{0.6} (1850/25650)^{0.2} (1.05x10^{12}/25650 \text{ x } 200^3)^{0.1}$

= 0 .60 per/girder

1.2.5.2 Distribution factor for shear force

For two or more lane loaded,

$DFV = 0.2 + s/3600 – (s/10700)^{2.0}$

Provided that, $1100 \leq S \leq 4900$ where S = 1850 mm

$6000 \leq L \leq 73000$ where L = 25650 mm

$110 \leq t_s \leq 300$ where $t_s = 200$ mm

$N_b \geq 4$ where $N_b = 4$

OK.

In which,

DFV = Distribution factor for shear for interior beam

This gives,

$DFV = 0.2 + (1850/3600) – (1850/10700)^{2.0}$

= 0.2 + 0.5138-0.029

= 0.68 per / girder (AASHTO 07 Table 4.6.2.2.3a-1)

1.2.5.3 Dynamic Allowance

IM = 33% (Ref. AASHTO 07 Table 3.6.2.1-1)

Where, IM = dynamic load allowance applied to truck load only.

1.2.6 Live load distribution factor for typical exterior beam

1.2.6.1 Distribution factor for bending moment

For exterior girder,

DFM = e x g_{girder}

Where, $-300 \leq d_e \leq 1700$

e = $0.77 + d_e/2800$

in which,

e = correction factor (AASHTO 07 Table 4.6.2.2.2d)

g = distribution factor

d_e = distance from the exterior web of exterior girder to the interior edge of curb as traffic barrier(mm)

= 50 mm

This gives,

e = 0 .77 + 50/2800 = 0.77

& DFM = e x $g_{interior}$

= 0 .787 x 0 .6
= 0.473 per/girder

1.2.6.2 Distribution factor for shear force

For exterior girder
DFV,
DFV = e x g $_{interior}$ (Ref. AASHTO 07 Table 4.6.2.2.3b-1)0
e = 0.60+de/3000
Here, e = 0.60 + 50/3000 = 0.62

This gives
DFV $_{interior}$ = 0.68 per/girder (previous calculation)
DFV $_{exterior}$ = 0.62 x 0.68 = 0.42 girder

1.2.7 Simplified conventional method for live load distribution factor

1.2.7.1 For Interior Girder

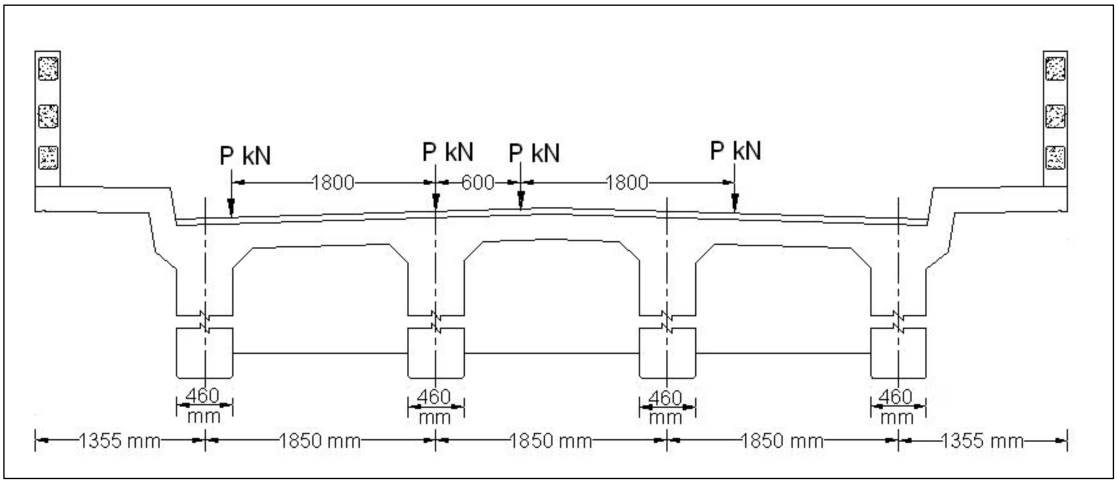


Fig 1.2.3 Truck Load Distribution (interior girder)

Load per girder = P + (1850-1800)/1850 x P + (1850 -600)/1850 x P

= 1.7 x P here, P = wheel load

1.2.7.2 For Exterior Girder

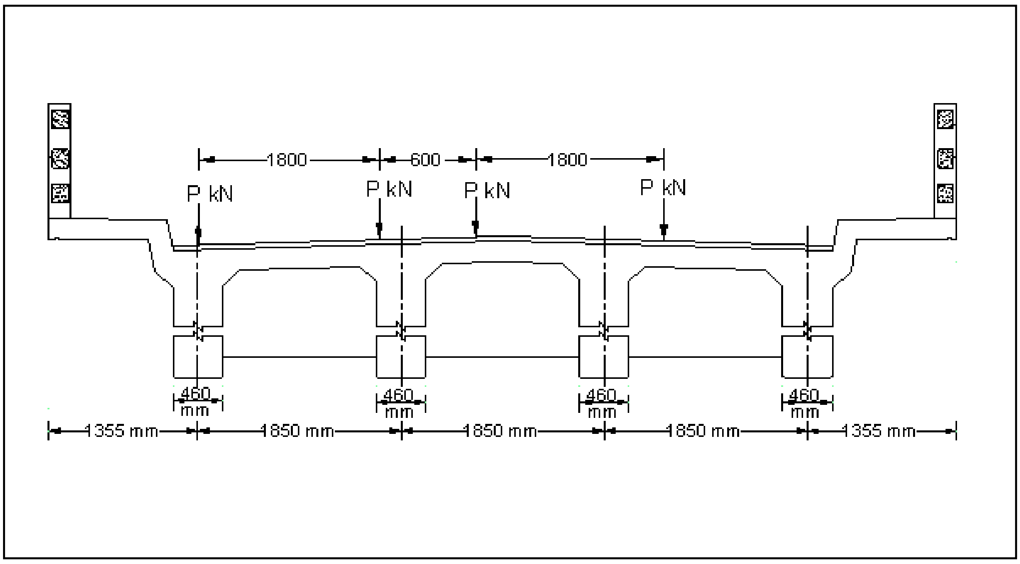


Fig 1.2.4 Truck Load Distribution (exterior girder)

Load per girder = P + (1850-1800)/1850 x P
= 1.03 x P   here, P = wheel load, kN

1.2.8 Service shear force and bending moment:

The self weight of girder, X- girder, deck slab, wearing course and live loads with wheel loads, lane load and impact load are considered. For exterior girder design calculation, self weight of rail post, rail bar, sidewalk are considered, in addition to previously considered dead loads. For live load calculation, pedestrian live load is considered along with wheel load and lane load.

1.2.8.1 Interior girder

1.2.8.1.1 Calculation of Weight

Self wt of girder = 0 .46 x1.8 x 24 = 19.87 kN/m
X- girder = 0 .375 x 1.3 x 1.85 x 24 = 21.65 kN
Deck slab = 0.20 x 1.85 x 24 = 8.88 kN/lm
Wearing course = 0.05 x1.85 x 23 = 2.13 kN/lm

1.2.8.1.2 Bending Moment due to Dead Load

Moment due to dead load at mid point,

Girder = $19.87 \times 25^2/8 = 1552.5$ kN-m
Deck slab = $8.80 \times 25^2/8 = 687.5$ kN-m
Wearing course = $2.13 \times 25^2/8 = 166.5$ kN-m
Support reaction of X- girder

$\sum F_y = 0$

►21.65 x 12.5 + 21.65 x 25 –R x 25 = 0
►R = 32.5 kN

X- girder moment = 32.5x12.5-21.6x12.5 = 135.5 kN-m

Dead load moment due to self weight, x-girder and deck slab

M_1 = (1552.5 + 135.5 + 687.5)
= 2375.5 kN-m

Dead load moment due to wearing course

M_2 = 166.5 kN-m

1.2.8.1.3 Shear Force due to Dead Load

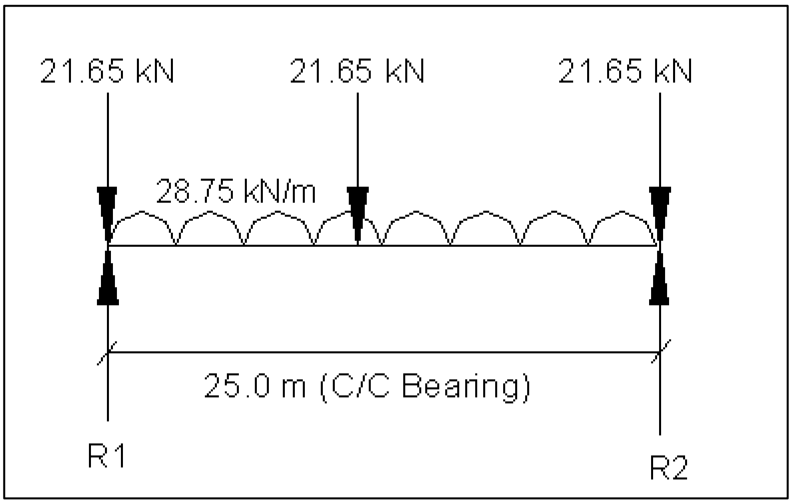


Fig 1.2.5 Load Diagram of Interior Girder (Dead load)

R_1 = {(21.65x25) + (21.65x12.5) +(28.75x25x12.5)}/25 = 391.85 kN

NOTE: For this calculation self wt of deck slab, self wt of girder has been considered as uniform load = 28.75 kN/lm and self wt of cross girder have been considered as concentrated Load of 21.65kN. (Previous Calculation)

Maximum shear at support

= (391.85– 21.65)

= 370.20 kN

Shear force due to wearing course = (2.13x25)/2 = 26.63 kN

1.2.8.1.4 Bending Moment Due to Truck Load

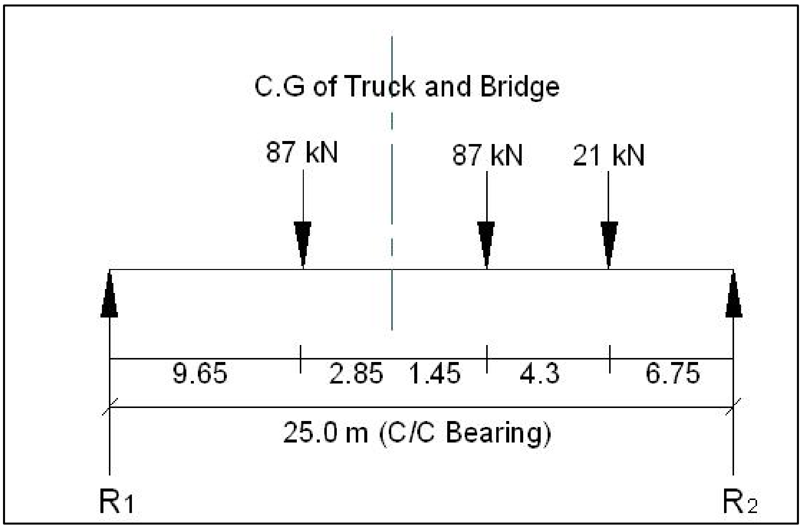


Fig 1.2.6 Loading Diagram of Interior Girder with truck load only

For obtaining maximum moment as single span bridge the C.G of truck is placed at the centre of the bridge.

(AASHTO 2007, Table 3.6.2.1-1)

P_1 = Rear end wheel load of truck = (72.5 x 2 x DFM) = 87 kN

P_2 = Middle wheel load of truck = (72.5 x 2 x DFM) = 87 kN

P_3 = Front wheel load of truck = 17.2 x 2 x DFM = 21 kN

W = lane load = 9.3 x DFM (Ref. AASHTO 07 Article No 3.6.1.2.4)
= 5.58 kN/m

R_1 = 21x6750 + 87 x (4300+67500) + 87x(4300+4300+6750)/25000 = 97.55 kN

Maximum moment at mid span due to truck load,

(97.55 x 125000) – (87 x 2850) = 971425 kN-mm = 972 kN-m

Total truck load moment with impact moment = 972(1+IM)
= 972(1+0.33)
= 1292.76 kN-m

1.2.8.1.5 Bending Moment Due to Lane Load

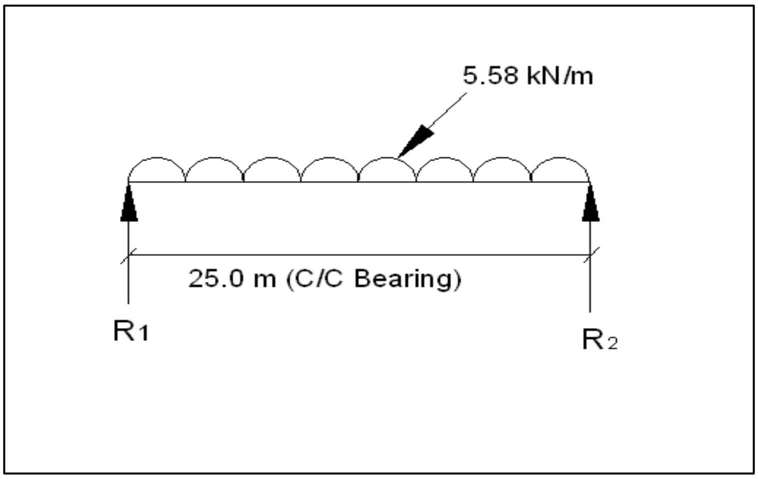


Fig 1.2.7 Loading Diagram of Interior Girder with Lane Load

Maximum moment due to lane load at mid span,

$$= 5.58 \text{ x } (25)^2/8 = 435.93 \text{ kN-m}$$

1.2.8.1.6 Shear Force due to Truck Load

P_1 = DFVx2P=0.68 x 2 x 72.5 = 98.6 kN, P= 72.5 kN
P_2 = DFVx2P= 0.68 x 2 x 72.5 = 98.6 kN, P= 72.5 kN
P_3 = DFVx2P= 0.68 x 2 x 17.5 = 23.8 kN, P= 17.5 kN

(NOTE: For maximum shear, rear wheel have been placed at support)

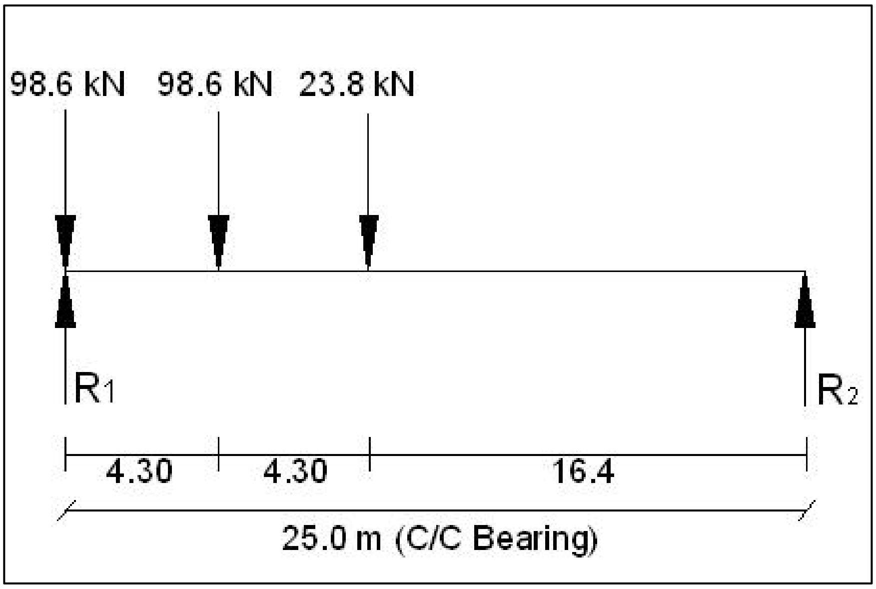


Fig 1.2.8 Loading Diagram of Truck Load for Shear Force Calculation

R_1 = {23.8 x 16.4 + 98.6 x (4.3 +16.4) +98.6x (4.3 +4.3 + 16.4)}/25
= 195.85 kN

Maximum shear at support,

= 195.85 -98.6 kN = 97.3 kN

Maximum shear with impact load,
= 97.3(1 +IM)
= 97.3(1 +.33)
= 130 kN

1.2.8.1.7 Shear Force due to Lane Load

Lane load = DFVx9.3 = 6.32 kN/m

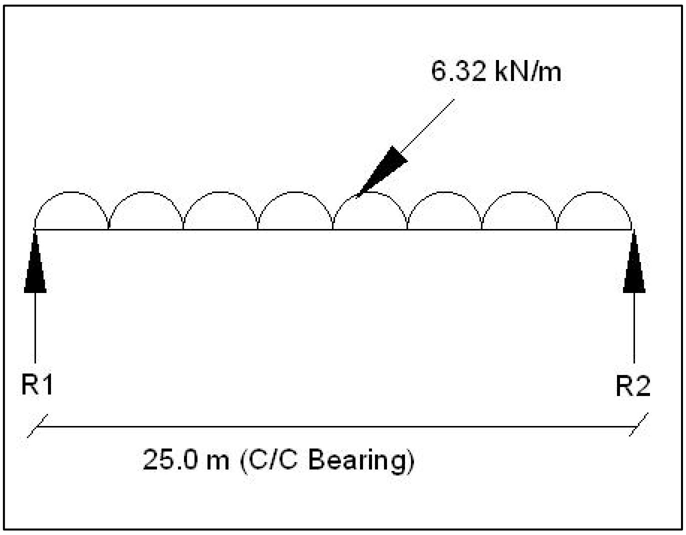


Fig 1.2.9 Loading Diagram of Lane Load for Shear Force Calculation

R_1 = (96.32 x 25 x25/2)/25 = 79 kN

Maximum shear at support = 79 kN

1.2.8.1.8 Bending Moment from Simplified Conventional Method for Truck Load

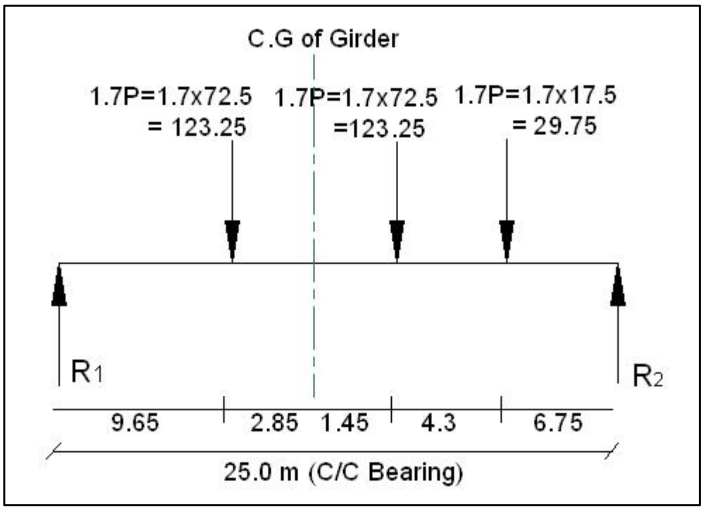


Fig 1.2.10 Truck Load Diagram for Bending Moment

R_1 = (29.75 x 6.75) +123.25 x (4.3 +6.75) + 123.25(2.85 +1.45 +4.3 +6.75)
= 138.18 kN

Moment at mid span = 1376.04 kN-m

Moment with impact factor = 1376.04(1 + 0.33) = 1830.13 kN-m

1.2.8.1.9 Bending Moment from Simplified Conventional Method for Lane Load

In this method lane load considered as

= 9.3 x 1.85 /3 = 5.73 kN/m

here, c/c girder distance = 1.85 m

Total lane load moment at mid span

M = 5.73 x 25^2/8 = 447.65 kN-m

1.2.8.1.10 Summary of design moment and shear factored

For interior girder,

Total factored moment (AASHTO 07)

= (1.25 x 2375.5)+(1.5 x 166.5) +(1.75 x 1292.76) +(1.75 x 435.93)

= 6244.33 kN-m

Total factored moment (Simplified Conventional Method)

= 1.25 x 2375.55 + 1.5 x 166.5 +1.75 x 830.13 + 1.75 x 447.65

= 7205.24 kN- m

Total factored shear

= (1.25 x DL shear due to self weight of girder, x-girder & deck) + (1.5 x DL shear due to self weight of wearing course) + (1.75 x LL shear from Truck Load and impact) + (1.75 x LL shear from lane load)

= (1.25 x 370.2) + (1.5 x 26.63) + (1.75 x 130) + (1.75 x 79)

= 868.45 kN

Since total moment calculated for simplified distribution method from AASHTO 07 is less than that of simplified conventional method, design moment is chosen for reinforcement calculation from simplified conventional method, given below:

Design Flexural Moment = 7205.24 kN-m

For calculation of reinforcement, refer to 1.2.8.1.11 below.

1.2.8.1.11 Crack Width Calculation

Bar stacing for maximum crack width of 0.22mm as oer AASHTO '07, Art. 5.7.3.4-1

In which,

$S \leq (123000\ \gamma e / \beta_{s\ fss}\ ) - 2d_c$

$\gamma_e\ = 0.5$

d_c = clear cover + bar dia

(we use T10 for shear reinforcement)

= 50 + 5 = 55

$\beta_s\ = [\{\ 1 + d_c\ \{\ 0.7(\ h\text{-}\ d_c\ )\}] = [\ \{\ 1 + 55/\{0.7(2000\text{-}55)\}]$

$= 1.040$

Service bending moment = 4270 kN-m

= $4270\ x10^6$ N-mm

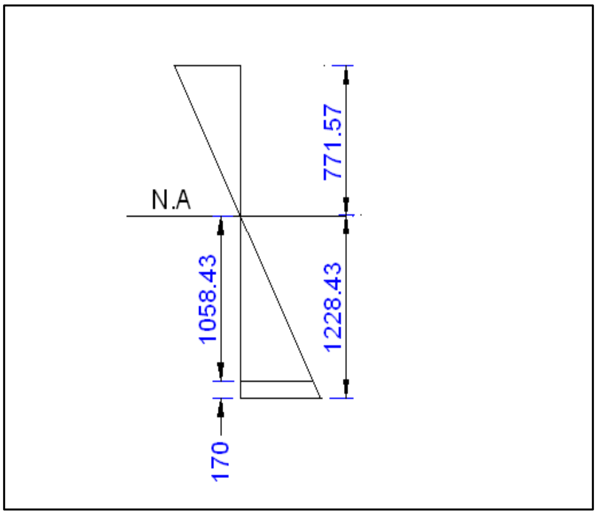


Fig 1.2.11 Stress Diagram of Interior Girder

$f_b\ = M\ C_b / I = (4270\ x\ 10^6\ x\ 1228.43)/4.86\ x\ 10^{11}$

$= 10.79\ N / mm^2$

$f_{sc}\ = 9.29\ N/\ mm^2$ [$10.79/1228.43 = f_{sc}\ /1058.43,\ f_{sc} = 10.79/1228.43\ \ x\ 1058.43$]

$f_{ss} = 9.2\ x\ 8.33$

$= 77.38$ MPa

$f_{ss} = (E_s/E_c)\ x\ f_{sc}$

$E_c = 24\ x\ 10^3$ MPa

$E_s = 200\ x\ 10^3$ MPa

$E_s/E_c = 8.33$

$S\ \ \leq (123\ x\ 10^3\ x\ 0.5/77.38\ x1.040) - 2x55$

≤ 654.15 mm

BS Method:

$f_t = (4270x10^6\ x\ 771.57)/4.86\ x\ 10^{11}$

$= 6.77N/mm^2$

Concrete strain $= 6.77\ /24\ x\ 10^3\ \ = .0003$

Crack width $= 3\,\acute{\varepsilon}_{min} \times a_{cr} = 3 \times .0003 \times 107.35$ [acr $= \sqrt{(100^2 + 55^2)} = 107.35$]
$= 0.090 < 0.2$
Maximum crack width 0.22 mm.

1.2.8.1.12 Deflection for 25.0m Girder (due to live load):

Deflection due to beam self wt,

$\Delta_g = 5wL^4/(384EI)$

$w =$ beam self wt. = 19.87 kN/m

$\Delta_g = (5\text{x}19.87 \times (25.65)^4) / (384\text{x}24\text{x}10^6\text{x}0.486)$

$= 0.0096$ m = 9.6 mm (↓)

Deflection due to slab wt,

$\Delta_s = 5wL^4/(384EI)$

$w =$ slab wt. = 8.88 kN/m

$\Delta_s = (5\text{x}8.88\text{x} (25.65)^4) / (384\text{x}24\text{x}10^6\text{x}0.486)$

$= 0.0043$ m = 4.3 mm (↓)

Deflection due to X-girder load,

$\Delta_{X\text{-girder}} = PL^3/(48EI)$

$P =$ X-girder load = 21.65 x3 = 37.95 kN

$\Delta_{x\text{-girder}} = (37.95 \times (25.65)^3\text{x}1000)/(48 \times 24 \times 10^6 \times 0.486)$

$= 1.15$ mm (↓)

Deflection due to wearing course,

$\Delta_{wc} = 5wL^4/(384EI)$

$w =$ wearing course wt. = 2.13 kN/m

$\Delta_{wc} = (5\text{x}2.13\text{x} (25.65)^4)/ (384\text{x}24\text{x}10^6\text{x}0.486) = 1$ mm (↓)

Deflection due to lane load,

$\Delta_{LL} = 5wL^4/(384EI)$

$w =$ lane load = 5.58 kN/m

$\Delta_{LL} = (5\text{x}5.58\text{x} (25.65)^4)/ (384\text{x}24\text{x}10^6\text{x}0.486)$

$= 2.69$ mm (↓)

Deflection due to truck & impact load,

$\Delta_{WL} = PL^3/(48EI)$

$P =$ wheel load = (145x2+35) x DFM x Dynamic allowance

$= 325\text{x}0.6\text{x}1.33$

$= 259.3$ kN

$\Delta_{WL} = (259.3 \times (25.65)^3\text{x}1000)/(48 \times 24 \times 10^6 \times 0.486)$

$= 7.8$ mm (↓)

Allowable deflection for live load = L/800 (Ref. AASHTO 07 Art. 2.5.2.6.2)
= (25.65) x 100/800 = 32.06 mm
Total live load deflection = 7.8+2.69 = 10.49 mm<32.06 mm

OK

1.2.8.1.13 Reinforcement calculation

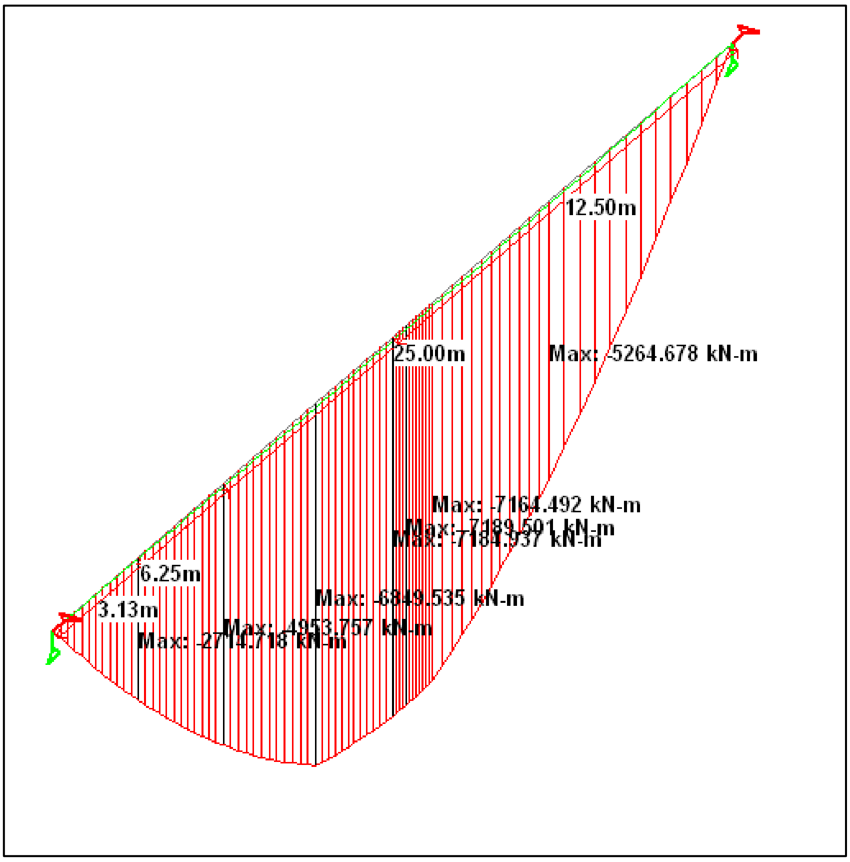


Fig 1.2.12 Moment Diagram of Interior Girder of 25.0m span (From STAAD.PRO analysis)

a) Flexural Moment

Total moment, M_u = 7205.24 kN-m (Simplified Conventional Method).

Using STAAD.Pro 2006, this analysis is again confirmed using live load distribution factor from simplified conventional method and thus design moment is chosen as 7200 kN-m.

Using, 20-T28 = 12300 mm^2

Effective depth of girder, d = 1800 mm

Lever arm factor, $a = A_s f_y / 0.85 \times f'_c \times b$
$= (12300 \times 400) / (0.85 \times 25 \times 1850)$
= 128.27 mm

Nominal moment, ΦM_n $= 0.9 \times A_s \times f_y (d - a/2)$
$= 0.9 \times 12300 \times 400 (1800 - 128.27/2)$
$= 7690$ kN-m

As, $\Phi M_n > M_u$
Flexure design is OK

b) Cracking Moment

Cracking moment, $M_{cr} = S_c (f_r + f_{cpe}) - M_{dnc}(S_c/S_{nc} - 1)$ (AASHTO 07 Art 5.7.3.3.2-1)

f_{cpe} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

= 0 (for RC girder)

S_c = S_{nc} = section modulus of composite girder= I_g / y_t (for RC girder)

M_{dnc} = total service dead load moment acting on the monolithic or non-composite section (N-mm)

So, $M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}(S_c/S_{nc} - 1)$

$= (fr \times I_g) / y_t$

Modules of rupture, fr (for normal density concrete) = $0.52 \sqrt{fc'}$
(Ref. AASHTO 07 Art 5.4.2.6)

$= 0.52 \sqrt{25}$
$= 2.6$ MPa

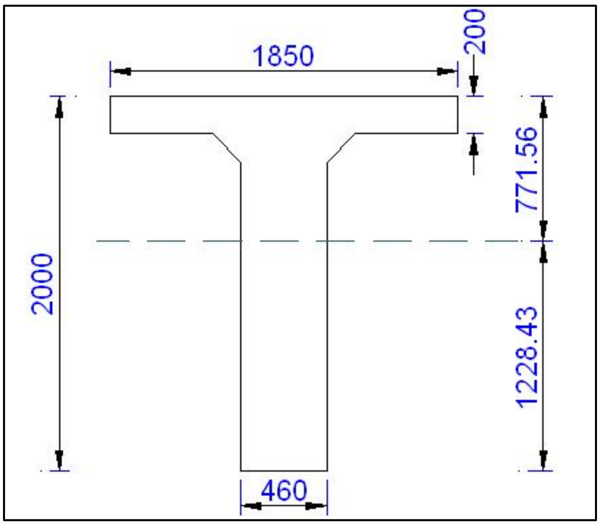


Fig 1.2.13 X-section of Girder

Distance from N.A to the top fiber of the section, yt = 771.56

Moment of inertia, I = $4.86\text{x } 10^{11}$ mm^4

Cracking moment, $M_{cr} = (2.6 \text{ x } 4.86 \text{ x}10^{11}) / 771.2 =$ 1638.48 kN-m

1.2 x M_{cr} = 1.2 x1638.48 = 2383 kN-m

As $\Phi M_n > M_{cr}$ flexure design is OK

c) Surface Reinforcement

Minimum reinforcement = 0.003bt
= 0.003x1000x460/2
= 690 mm^2
Providing T-10,
Spacing = (1000x78.5)/690
= 113.76 mm
= 100 mm
Effective depth = 1800 mm
Total no of surface reinforcement = 1800/100x2
= 9

d) Shear Reinforcement

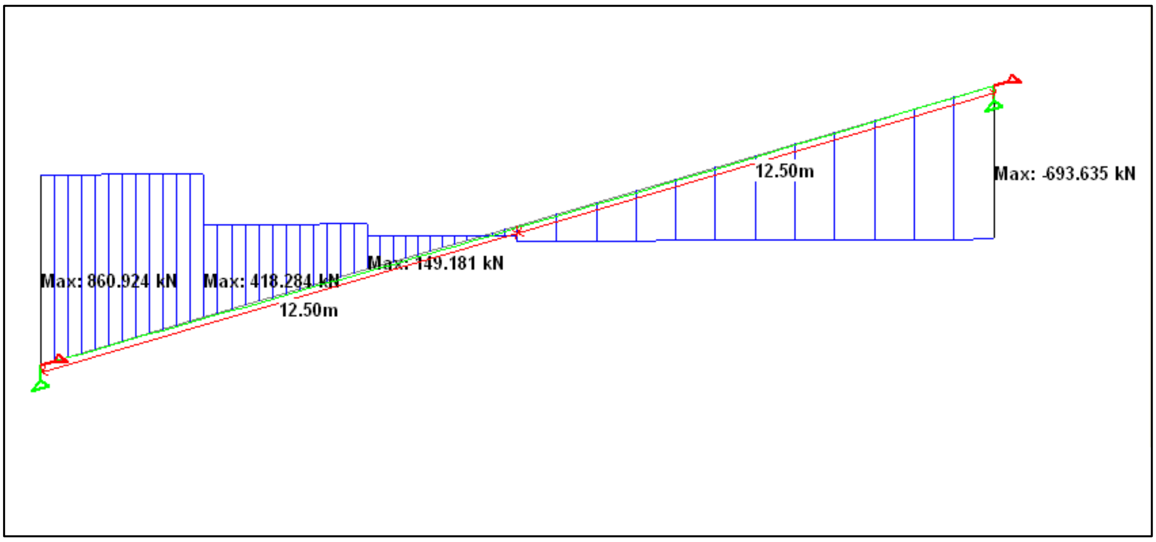


Fig 1.2.14 Shear Force Diagram for 25.0m Span (From STAAD.Pr0 Analysis)

i) Nominal shear resistance calculation

Nominal shear resistance, V_n shall be determined as lesser of

$V_n = V_c + V_s + V_p$ (Ref. AASHTO 07 Art. 5.8.3.3-1)
$V_n = 0.25\, f'_c\, b_v d_v + V_p$ (Ref. AASHTO 07 Art. 5.8.3.3-2)

Here, $V_c = 0.083\beta\sqrt{f_c'}\, b_v\, d_v$ (Ref. AASHTO 07 Art. 5.8.3.3-3)

$V_s = (A_v\, f_y\, d_v\, \cot\theta)/s$

b_v = effective web width taken as the minimum web width within the depth d_v.

d_v = effective shear depth .

s = spacing of stirrups.

θ = angle of inclination of diagonal compressive stresses.

β = factor indicating ability of diagonally cracked concrete to transmit tension.

Av = area of shear reinforcement within a distance "s"

Vp = component in the direction of the applied shear of the effective prestressing force.

For flexural members, the distance between the resultants of the tensile and compressive forces due to flexure can be determined as

$d_v = M_n / A_s f_y$ (AASHTO Art 5.8.2.9)

$= A_s f_y (d - a/2) / A_s f_y$

$= 1736$ mm

d_v need not to be taken to be less than the greater of 0.9de or 0.72h (mm)

Here, d = 1800 mm

$a = (A_s f_y)/(0.85\, f'_c\, b)$

$= 128.27$ mm

Now, 0.72h = 0.72 x 2000 = 1440mm

0.9de = 1794 x .9 = 1614.6 mm

$d_v >$ greater of 0 .72h and 0.9 de .

So, OK.

ii) Calculation of β & θ

θ = angle of inclination of diagonal compressive stresses.

β = factor indicating ability of diagonally cracked concrete to transmit tension.

$v_u / f_c' = 0.69/25$ here, v_u = total factored shear/area

$= .028$

$= 868.45/1243 \times 10^3$

$= 6.9 \times 10^{-4}$ kN/mm^2

$= 0.69$ N/mm^2

from AASHTO Table-5.8.3.4.2-1

let $\varepsilon_x = .001$

θ = 36.4

β = 2.23

Hence,

$V_c = 0.083\beta\sqrt{f_c'}\, b_v\, d_v$

$= 0.83 \times 2.23 \times \sqrt{25} \times 460 \times 1730$

$= 736.47$ kN

Using, T12-150

V_s shall be determined by,

V_s = $A_v\ f_y d_v \cot \theta /s$ (Ref. AASHTO 07 Art 5.8.3.3-1)

= (2 x 113 x 400 x 1736 x 1.37)/150

= 1428.38 kN

$V_n = V_c + V_s + V_p$

= 736.47 + 1428.38 +0 = 2164.85 kN

$V_n = 0.25\ f_c'\ b_v\ d_v + V_p$

= 0.25 x 25 x 460 x 1736 + 0

= 4973.75 kN

Nominal Shear resistance, V_n shall be lesser than these two values.

So, V_n = 2164.85 kN

Again, Using T10-150

$V_s = (A_v \times f_y\ d_v \cot \theta)/s$ = (2x78.5 x 400 x1736 x1.37) /150 = 992.28 kN

$V_n = V_c + V_s + V_p$ =736.47 +992.28 +0 = 1728.75 kN

$V_n = 0.25 \times f'_c\ b_v\ d_v + V_p$ = 4973.75 kN

Therefore,

V_n = 1728.75 kN

V_u = 868.45 kN

For the conservative design critical shear have been taken at support.

$V_u < V_n$ so OK.

So we provide T10-150 at a distance "d" from support and then provide T10-200 after distance "d" .

iii) Regions for requiring Transverse Reinforcement

For beam, $V_u > 0.5\varphi\ (Vc+Vp)$ (Ref. AASHTO 07, Art. 5.8.2.4-1)

V_u = 868.45 kN

$0.5 \times \varphi \times V_c$ = 0.5 x 0.9 x 637 = 286.65 kN

as $V_u > V_c$ so shear reinforcement is needed.

iv) Minimum Transverse Reinforcement

$A_v \geq 0.083\sqrt{f_c'}\ b_v\ s / f_y = 0.083\sqrt{25}x460x150/400 = 71.58\ mm^2$

(Ref. AASHTO 07 Art. 5.8.2.5)

v) Maximum Spacing of Transverse Reinforcement:

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing s_{max} determined as:

If $v_u < 0.125 \times fc'$ then:

s_{max} = $0.8d_v \leq 600$ mm (Ref. AASHTO 07 Art. 5.8.2.7-1)

If $v_u \geq 0.125xfc'$

then:

s_{max} = $0.4\ d_v \leq 300$ mm (AASHTO Art. 5.8.2.7-2)
v_u = $V_u/A = 868x10^3/1243x10^3 = 0.69\ N/mm^2$
$v_u/\ fc'$ = $0.69/25 = 0.027$

Therefore,

s_{max} = $0.8d_v \leq 600$ mm
= $0.8x1736 \leq 600$
= $1388 \leq 600$
provided maximum spacing = 200 mm < 600 mm, so ok.

1.2.8.2 Exterior Girder

Bending moment and shear fore due to dead load

Self wt of girder =0.46 x1.8 x 24 = 19.87 kN/m
X – girder = 0.375 x1.3 x 24x1.85/2 = 10.82 kN
Deck slab = 0.2 x1.85/2 x 24 = 4.44 kN/m
W/c = 0 .05 x1.85/2x24 = 1.11 kN/m
Rail post = (0.2 x 0.2x 1.1x24) /1.58 = 0.67 kN/m
Rail beam = 0.185 x 0.15 x 24 x 3 = 2 kN/m
Side walk slab = 1.125 x.2 x 24 = 5.4 kN/m

1.2.8.2.1 Bending Moment due to Dead Load

Moment due to girder = $19.87 \times 25^2/8$ = 1552.34 kN-m
Moment due to deck slab = $4.4 \times 25^2/8$ = 344 kN-m
Moment due to w/c = $1.11 \times 25^2/8$ = 86.72 kN-m
Moment due to rail post = $0.67 \times 25^2/8$ = 52.34 kN-m
Moment due to rail beam = $2 \times 25^2/8$ = 156.2 kN-m
Moment due to side walk = $5.4 \times 25^2/8$ = 421.87 kN-m
Moment due to cross girder,
= [{(10.82 x 12.5) + (10.82 x 25)}/25] x 12.5
–(10.82x12.5)
= 67.62 kN-m

Dead load moment due to girder, deck, rail post, rail beam, side walk and wearing course
M_1 = 2594.17 kN-m

Dead load moment due to wearing course
M_2 = 86.72 kN-m

1.2.8.2.2 Shear Force due to Dead Load

For this calculation self wt of deck slab, girder, rail post and rail beam, side walk have been considered as uniform load = 32.38 kN/m and self wt of cross girder have been considered as concentrated load value of 10.82 kN

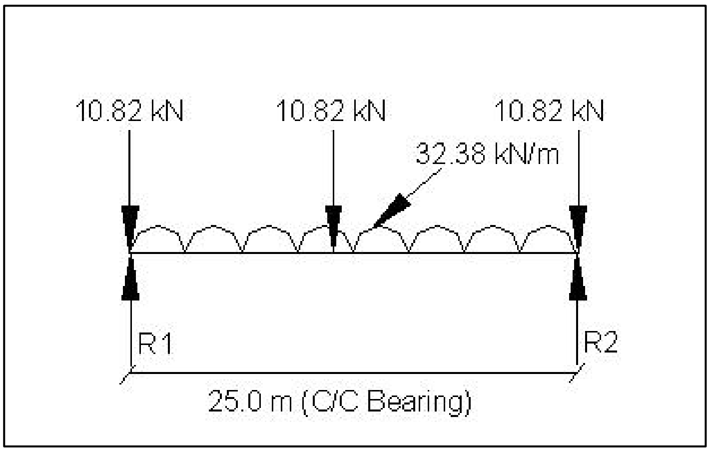


Fig 1.2.15 Loading Diagram for Shear (Exterior Girder)

R_1 = {(32.38 x 25 x25/2) + (10.82 x12.5) + (10.82 x25)} /25 = 420.98 kN

Maximum shear at support,
= 420.98 – 10.82
= 410.16 kN

Wearing course shear force = (1.11x25)/2 = 13.87 kN

For live load moment calculation,

Pedestrian live load of 3.6x 10^{-3} MPa shall be applied to all side walks wider than 600 mm and considered simultaneously with the vehicular design live load.

Total side walk = 0.875 m
Distribution load over side walk = 0.875x3.6x10^{-3} = 3.15 kN/m
Moment due to pedestrian load = 3.15x25^2/8 = 246.09 kN-m

Rest of the calculation are similar to that of as provided for interior girder.

1.3 STRUCTURAL DESIGN OF PC GIRDER

1.3.1 Introduction

The design example demonstrates the design of a 40.0 m span prestressed concrete girder. For this example, 200mm thick RC deck slab, 50 mm wearing course and pre-cast railing are considered. This example illustrates in detail the design of typical interior girders. The design is accomplished in accordance with the AASHTO LRFD Bridge Design Specification 2007.

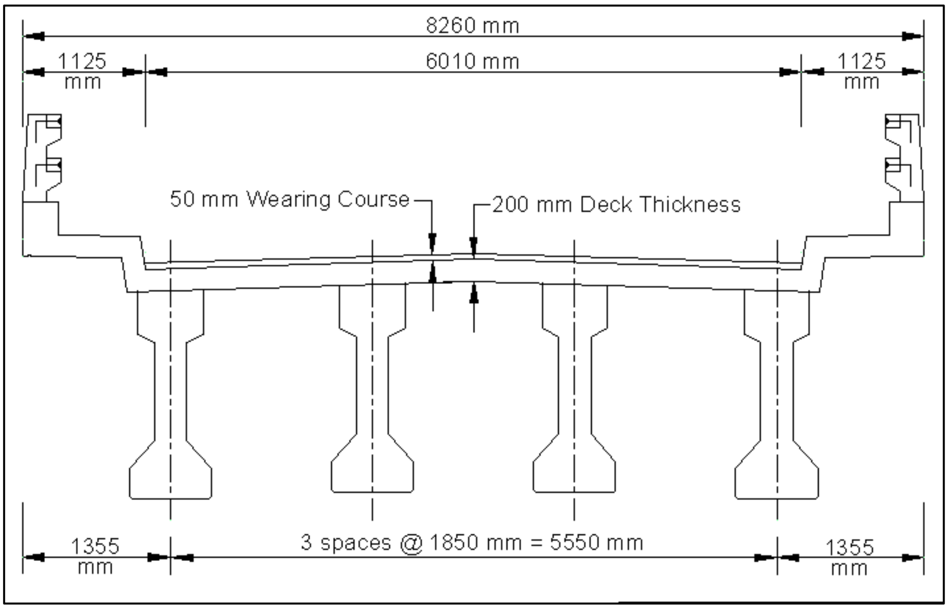


Fig 1.3.1 Bridge Cross Section

1.3.2 Data

Cast in place deck slab:

Actual thickness, ts = 200 mm
Concrete strength at 28 days, f'_c = 25 MPa

Pre-cast girder:

Concrete strength at transfer, f'_{ci} = 0.75 x 35 = 26.25 MPa
Concrete strength at 28 days, f'_c = 35 MPa
Concrete unit weight = 24 kN/m^3
Overall beam length = 40.65 m
Design span = 40 m

Pre-stressing strands:

12.7 mm dia , seven wire, low relaxation strands
Area of one strand = 98.71 mm^2

No of strands in one cable = 12

No of cable = 6

Ultimate strength, f_{pu} = 1860 MPa

Yield strength = 0.9 $f_{p\,u}$= 1674 MPa

Stress limit for pre-stressing strands:

[Ref. AASHTO 07, Table 5.9.3-1]

Before transfer, $f_{pbt} \leq 0.75\ f_{pu} = 0.75 \times 1860 = 1395$ Mpa

At service limit state after all Losses,

$f_{pe} \leq 0.80\ f_{py} = 0.80 \times 1674 = 1339.2$ MPa

Modulus of elasticity, E_p = 197000 MPa

[Ref. AASHTO 07, Art. 5.4.4.2]

Non pre-stressing reinforcement:

Yield strength of steel, f_y = 415 MPa

Modulus of elasticity, E_s = 200,000 MPa

1.3.3 Section Property:

1.3.3.1 Non Composite Section at Mid Span:

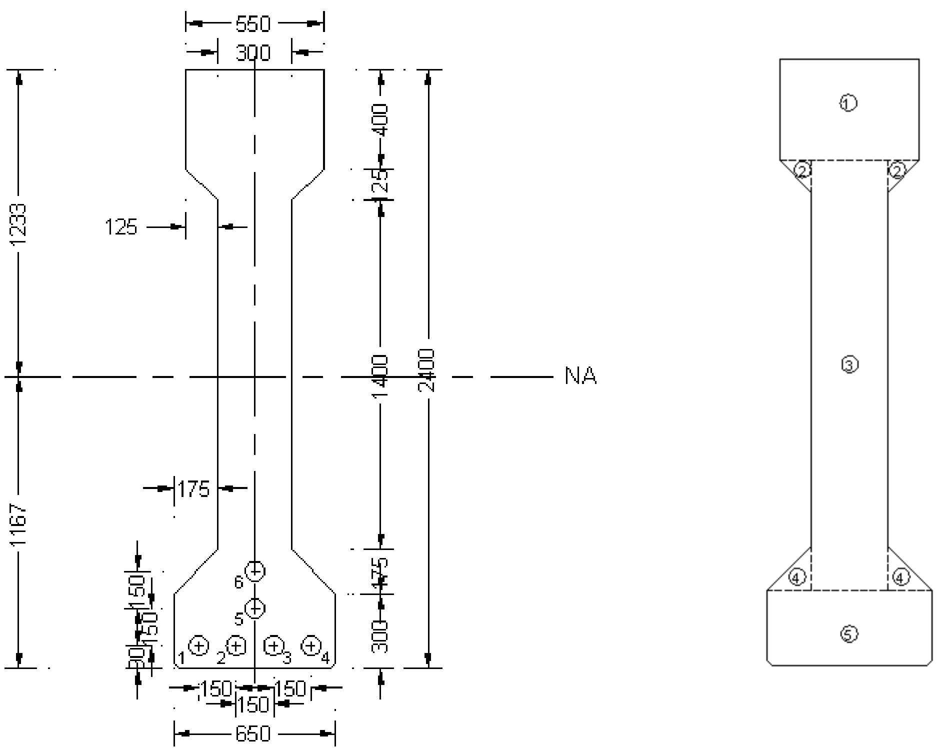


Fig 1.3.2 Non Composite Mid-Section

Table: Showing ∑ A, ∑ Ay, ∑ I, ∑A (ybar)^2 for non-composite mid-section of girder

Component	b	h	A	y	Ay	y NA	I	Y bar	A(y bar)^2
	mm	mm	mm2	mm	mm3	mm	mm4	mm	mm4
1	550	400	2.20E+05	2200	4.84E+08	1167.31	2.93E+09	1032.69	2.35E+11
2	125	125	1.56E+04	2083.3	3.26E+07		6.78E+06	916.02	1.31E+10
3	300	1700	5.10E+05	1150.0	5.87E+08		1.23E+11	-17.31	1.53E+08
4	175	250	4.38E+04	383.3	1.68E+07		7.60E+07	783.98	2.69E+10
5	650	300	1.95E+05	150.0	2.93E+07		1.46E+09	1017.31	2.02E+11
Non-Composite ∑ A =			9.84E+05	∑ Ay =	1.15E+09	∑ I =	1.27E+11	∑A(ybar)^2 =	4.77E+11

y_b = Distance from centroid to the extreme bottom fiber of non-composite girder
= 1167.31 mm
y_t = Distance from centroid to extreme top fiber of non-composite girder
= 1232.69 mm
∑ I = 1.32E+11 + 4.63E+11
= 6.04E+11 mm4

S_b = Section modulus for the extreme bottom fiber = I/ y_b = $\frac{6.04x10^{11}}{1167.31}$ = 5.17 x 10^8mm^3

S_t = I/ y_t = $\frac{6.04x10^{11}}{1232.69}$ = 4.90 x 10^8mm^3

For normal density concrete with, γ_c = 2320 kg/ m^3
Modulus of elasticity = 4800√ f'$_c$
Therefore, Modulus of elasticity for cast in place slab = E_c = 4800√ 2500
= 24000 MPa

Pre cast girder at transfer, Eci = 4800√ 26.25 = 24592 MPa
Pre cast girder at service load, Ec = 4800 √ 35 = 28397 MPa

Calculation for effective flange width of Non- composite section:

(Ref. AASHTO 07, Art. 4.6.2.6.1)

Effective flange width shall be the lesser of:

1. 1 / 4 span = 1/4 x 40 = 10m = 10000 mm
2. 12 t_s + greater of web thickness or 1/2 x beam top flange width
   =12 x 200 + 1/2 x 550 = 2675 mm
3. Average spacing between girder = 1850 mm
   Effective flange width = 1850 mm

1.3.3.2 Composite Section at Mid Span

Modular ratio between slab and girder materials, n $= \dfrac{E_c(slab)}{E_c(girder)}$

$= \dfrac{24000}{28397}$

$= 0.8452$

Transformed flange width = n x (effective flange width) = 0.8452 x 1850

= 1563.62 mm

Transformed flange area = n x (effective flange width) t_s= 0.8452 x 1850 x 200

= 312724 mm^2

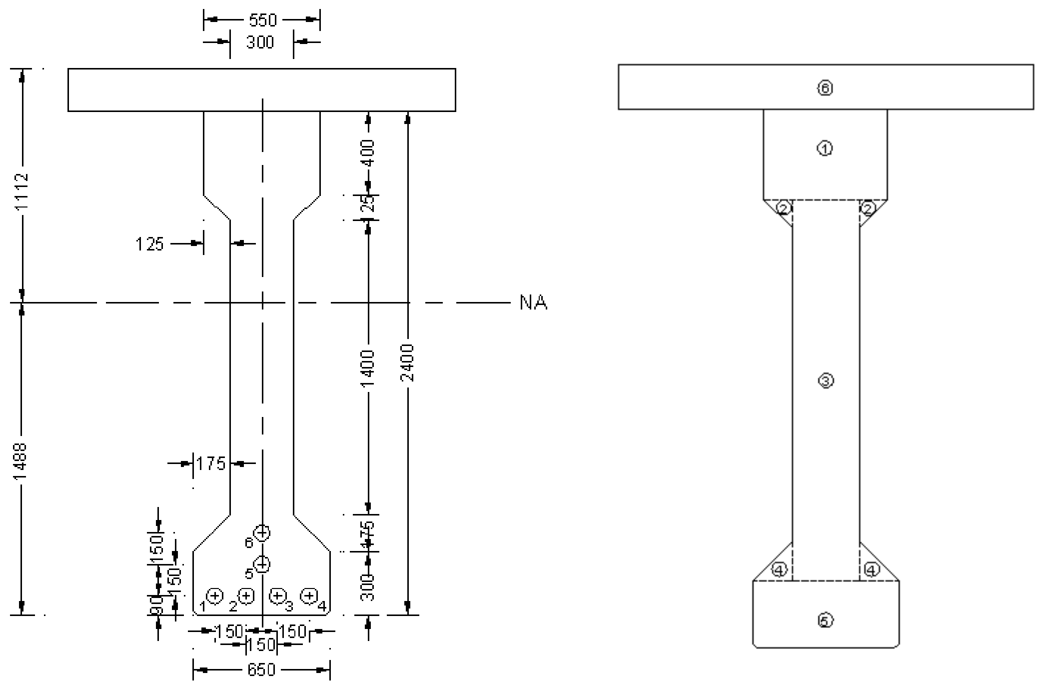


Fig 1.3.4 Composite Mid-Section

Table: Showing ∑ A, ∑ Ay, ∑ I, ∑A(ybar)2 for composite mid-section of girder

:Component	b	h	A	y	Ay	yNA	I	ybar	A(ybar)^2
	mm	mm	mm2	mm	mm3	mm	mm4	mm	mm4
1	550	400	2.20E+05	2200	4.84E+08	1488.60	2.93E+09	711.40	1.11E+11
2	125	125	1.56E+04	2083.3	3.26E+07		2.03E+07	594.73	5.53E+09
3	300	1700	5.10E+05	1150	5.87E+08		1.23E+11	-338.60	5.85E+10
4	175	250	4.38E+04	383.3	1.68E+07		2.28E+08	1105.27	5.34E+10
5	650	300	1.95E+05	150	2.93E+07		1.46E+09	1338.60	3.49E+11
6	1564	200	3.13E+05	2500	7.82E+08		1.04E+09	1011.40	3.20E+11
composite ∑ A=			1.28E+06	∑Ay=	1.93E+09	∑ I	1.29E+11	∑A(ybar)^2	8.98E+11

A_c = Total area of composite girder
= 1.30 E+06 mm2
h_c = overall depth of the composite section = 2400 + 200 = 2600 mm
I_c = moment of inertia of the composite section
= 1.34E+11 + 8.82E+11 = 1.03E+12
y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the pre-cast girder = 1488.60 mm
y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the deck
= 1111.40 mm
y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the pre-cast girder = (2400 – 1488.60) = 911.4 mm
S_{bc} = Composite section modulus for the extreme bottom fiber of the pre cast girder
= I_c / y_{bc} = 1.03×10^{12} /1488.60 = 6.9×10^8 mm^3
S_{tg} = Composite section modulus for the top fiber of the pre-cast girder = Ic / y_{tg}
= 1.03×10^{12} / 911.40 = 1.13×10^{9} mm^3
S_{td} = Composite section modulus for the extreme top fiber of the deck
= $1/n \times (Ic / y_{tc})$ = 1.03×10^{12} / 1111.40 x (1/0.8452)
= 1.09×10^9 mm^3

1.3.3.3 End section of PC Girder:

X – sectional area = (300 x 650) +(50 x 50) + (2100 x 550)
= 1.35×10^6 mm^2
= 1.35 m^2

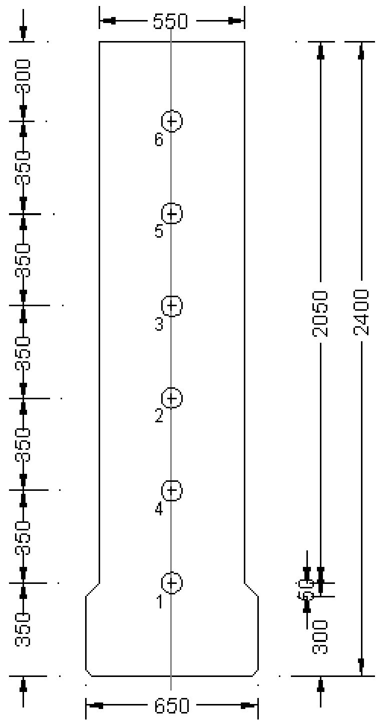


Fig 1.3.5 Non Composite End Section

1.3.4 Calculations for Interior Girder:

1.3.4.1 Dead Load Moment due to Self Weight:

Self wt. of end block = 1.35 x 24 = 32.4kN/m
Self wt of mid block = 0.984 x 24 = 23.62 kN/m

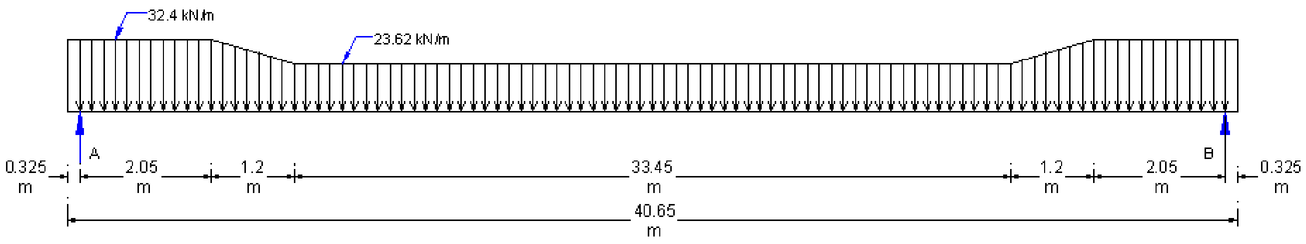


Fig 1.3.6 Load Diagram for girder self weight

R_A = (32.4 x 2.4) + (32.4 + 23.62)/2 x 1.2 + 23.62 x 16.725
= 506.42 kN

M_{cg} = 506.42 x 20 – 77.76 x 19.125 – 8.78 x (20.325 -2.4 -1/3 x 1.2) – 23.62 x 1.2 x 17.325
- 395.04 x 8.36
= 4693.79 kN-m

1.3.4.2 Dead Load Moment due to Cross Girder:

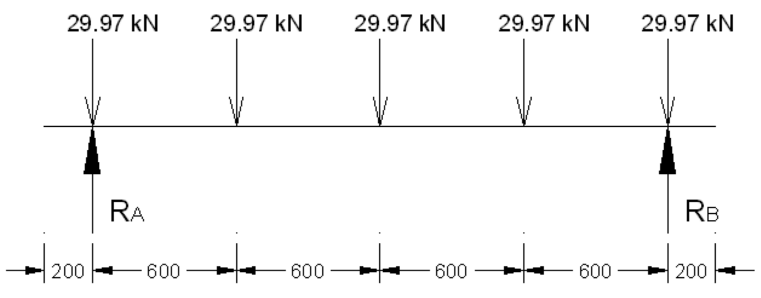


Fig 1.3.7 Load Diagram for Cross Girder

For interior girder, distance between girder = 1.85m c/c
Load from cross girder = (2400 x 3/4)/1000 x 375/1000 x 1.85 x 24
= 29.97 kN
Mc.xg = 5 x 29.97/2 x 20 – 29.97 x (10+20) = 599.4 kN-m

1.3.4.3 Dead Load Moment due to Deck Slab

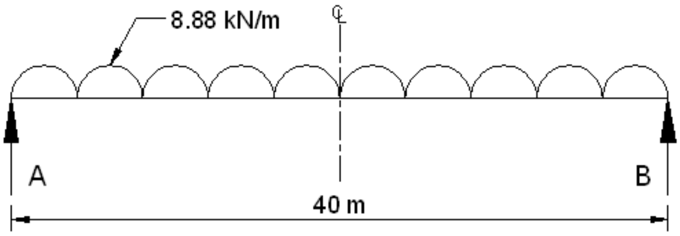


Fig 1.3.8 Load Diagram for Deck slab

W_d = 1.85 x 0.2 x 24 = 8.88 kN/m
Mc (deck) = 8.88 x 40^2 / 8 = 1776 kN-m

1.3.4.4 Dead Load Moment due to Wearing Course:

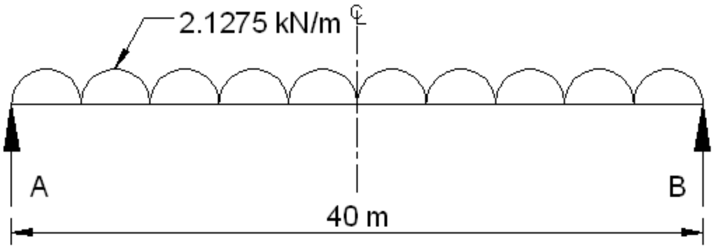


Fig 1.3.9 Load Diagram for wearing course

Considering 50 mm thickness of wearing course = 50/1000 x 23 x 1.85 = 2.1275 kN/m

$$M = \frac{wL^2}{8} = 2.1275 \times 40^2/8 = 425.5 \text{ kN-m}$$

1.3.4.5 Total Dead Load Moment:

Total service dead load moment**:**

= 4693.79 + 599.4 + 1776 + 425.5
= 7494.69 kN-m

Total factored dead load moment:

= (4693.79 + 599.4 + 1776) x 1.25 + 425.5 x 1.5
= 9474.74 kN-m

1.3.5 Live load distribution factor for typical interior beams:

Live load bending moment and shear force are determined by using the simplified distribution factor formulas (Ref. AASHTO 07, Art-4.6.2.2). To use the simplified live load distribution factor formula the following conditions must be made

- Width of slab is constant **OK**
- Number of girder, $N_b \geq 4$ (N_b = 4) **OK**
- Beam are parallel and of same stiffness **OK**
- Road way part of overhang, $d_e \leq 910$ mm (d_e=0) **OK**
- Curvature is less than 4^0 (Ref. AASHTO 07 table 4.6.1.2.1-1)
  (curvature= 0˚) **OK**

Number of design lanes = 2

1.3.5.1 Distribution factor for Bending Moment:

For two or more design lane loaded,

$DFM = 0.075 + (S/2900)^{0.6} (S/L)^{0.2} (K_g / Lts^3)^{0.1}$ (Ref. AASHTO 07, Table.4.6.2.2.2b-1)

Provided that,

DFM = Distribution factor for interior beam

S = Beam/girder spacing, mm = 1850mm

t_s = Thickness of deck, mm = 200mm

L = Length of girder, mm = 25650mm

$K_g = n(I + Ae_g^2)$= longitudinal stiffness parameter

Here,

$n = E_g / E_d = \sqrt{35}/\sqrt{25} = 1.18$

E_g = Modulus of elasticity of girder

E_d = Modulus of elasticity of deck

e_g = Distance between the C.G of beam and deck = 2600-1488.60-100 = 1011.4 mm

A = Cross-sectional area of girder, $mm^2 = 1.30 \times 10^6$ mm^2

I = Moment of inertia of beam, $mm^4 = 1.03 \times 10^{12}$ mm^4

Thus,

$$K_g = n(I + Ae_g^2)$$
$$= 1.18 \times (1.03 \times 10^{12} + 1.30 \times 10^6 \times (1011.4)^2)$$
$$= 2.785 \times 10^{12}$$

Therefore,

$$DFM = 0.075 + (1850/2900)^{0.6} (1850/40650)^{0.2} \{2.785 \times 10^{12} / (40650 \times 200^3)\}^{0.1}$$
$$= 0.586 \text{ lane/girder}$$

1.3.5.2 Distribution factor for Shear Force:

For two or more lane loaded,

$DFV = 0.2 + s/3600 - (s/10700)^{2.0}$ (Ref. AASHTO 07, Table.4.6.2.2.3a-1)

Provided that, $1100 \leq S \leq 4900$ where S = 1850 mm

$6000 \leq L \leq 73000$ where L = 40650 mm

$110 \leq t_s \leq 300$ where $t_s = 200$ mm

$N_g \geq 4$ where $N_g = 4$

OK.

DFV = Distribution factor for shear for interior beam

Therefore,

$DFV = 0.2 + (1850/3600) - (1850/10700)^{2.0}$

$= 0.2 + 0.5138 - 0.029$

$= 0.68$ lane / girder

(Ref. AASHTO 07, Table 4.6.2.2.3a-1)

1.3.5.3 Dynamic Allowance

IM = 33%

Where, IM = dynamic load allowance applied to truck load only.

(Ref. AASHTO 07, Table 3.6.2.1-1)

1.3.6 Calculation of Live load moment for interior girder:

1.3.6.1 Moment due to truck load:

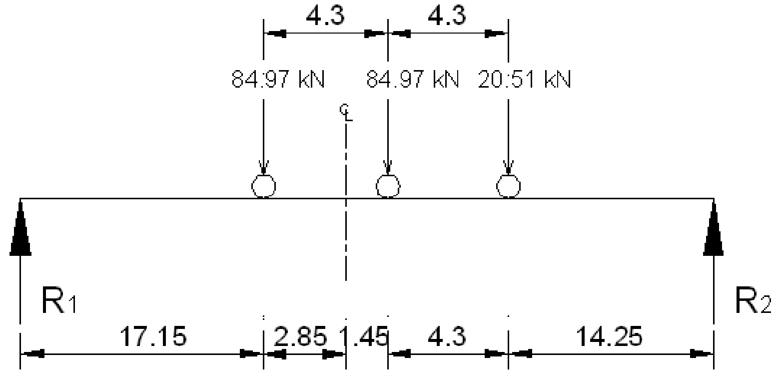


Fig 1.3.10 Load Diagram for Live Load

Rear wheel load = 145 x 0.586 = 84.97 kN

Front wheel load = 35 x 0.586 = 20.51 kN

R_1 = {20.51 x 14250 + 84.97 x (14250 + 4300) + 84.97 x (14250+4300+4300)}/40000
= 95.25 kN

Maximum service moment at mid span due to truck load with dynamic load allowance:
= 95.25 x 20.000 – 84.97 x 2.850
= 1662.84 kN-m

For Impact Moment, 1662.84 x 1.33 = 2211.58 kN-m

1.3.6.2 Moment due to lane load:

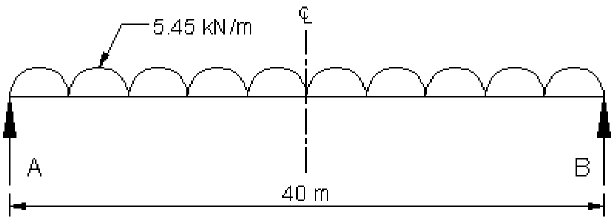


Fig 1.3.11 Load Diagram for Lane Load

Lane load, w = 9.3 x DFM (Ref. AASHTO 07, Art. 3.6.1.2.4)
= 9.3 x 0.586
= 5.45 kN/m
$M = wl^2 / 8$
$= 5.45 \times 40^2 / 8$
= 1090 kN-m

Factored live load moment = (2211.58 + 1090) x 1.75 = 5777.77 kN-m

1.3.7 Calculation of Shear Force for Interior Girder:

1.3.7.1 Shear force due to truck load

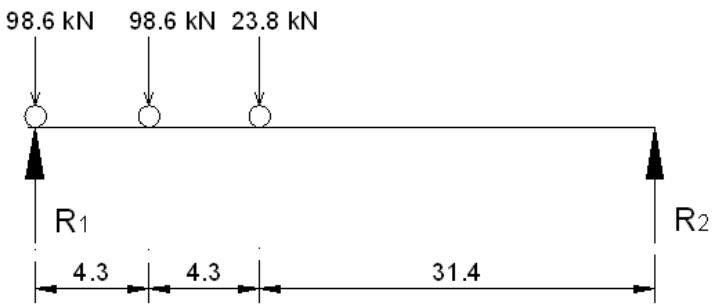


Fig 1.3.12 Load Diagram for Truck Load

Rear wheel load = 145 x 0.68 = 98.6 kN

Front wheel load = 35 x 0.68 = 23.8 kN

R_A = {23.8 x 31400 + 98.6(31400 + 4300) + 98.6(31400+4300+4300)}/40000

= 205.28 kN

V_A = 205.28 – 98.6 =106.68 kN

Live load shear with impact = 106.68 x 1.33

= 141.89 kN

1.3.7.2 Shear force due to lane load

W = 9.3 x 0.68 = 6.32 kN/m

Max shear = wl/2 = 6.32 x 40/2 = 126.4 kN

Total service dead load shear

Shear due to self wt of girder = 506.42 kN

Shear due to X girder:

R_A= 29.97 x (10+20+30+40)/40

= 74.925 kN

Shear due to X girder	=74.925 – 29.97 = 44.96 kN
Shear due to deck slab	= 8.88 x 20 = 178 kN
Shear due to WC	= 2.1275 x 20 = 43 kN
Total service dead load shear	= 506.42+44.96+178+43
	= 772.38 kN
Total service live load shear	= 141.89 + 126.4 = 268.29 kN
Total factored dead load shear	= (506.42 + 44.96 +178) x 1.25 + 43 x 1.5 = 976.23 kN
Total factored live load shear	= 268.29 x 1.75 = 469.51 kN
Total factored shear	= 976.23 + 469.51 kN
	= 1445.74 kN

1.3.8 Calculation of Losses: (Ref . AASHTO' 07 Art. 5.9.5.1)

1.3.8.1 Immediate Loss:

a) Loss due to wedge pull-in and friction:

Loss due to friction between the internal pre-stressing tendons and the duct wall may be taken as:

$\Delta f_{PF} = f_{pj} [1-e^{-(Kx + \mu\alpha)}]$

Where,

f_{pj} = Stress in the pre-stressing steel at jacking (MPa)
x = Length of a pre-stressing tendon from the jacking end to any point under consideration (mm).
K = Wobble friction co-efficient (per mm of tendon).
μ = Co-efficient of friction.
α = Sum of the absolute values of angular change of pre-stressing steel path from jacking end
if tensioning is done equally at both ends to the point under investigation (rad).
e = Base of Napierian logarithms.

Here the sample calculation for cable no. 5 is provided

Radius of curvature, $R = L^2/8d_r = 40.65^2/(8 \text{ x } 1.51) = 136.78$ m

Here,

d_r = Vertical sag = 1510 mm

When, X = 1 m

$\alpha = \sqrt{(\alpha_V^2 + \alpha_H^2)}$
$= \sqrt{(\alpha_V^2 + 0)}$
$= \alpha_V$
$= X/R$
$= 1/ 136.78$
$= 0.0073$ rad

Loss of pre- stress force due to friction per unit length,

$\Delta P = P_i [1-e^{-(Kx + \mu\alpha)}]$

Here,

$P_i = (0.75 \text{ x } 1860 \text{ x } 98.7 \text{ x } 12)/1000$
$= 1652$ kN
$\mu = 0.25$
$\alpha = 0.0073$ rad
$K = 0.007$
$X = 1$ m

So,

$\Delta P = 1652 \text{ x } [1-e^{-(0.007 \text{ x } 1+0.25 \text{ x } 0.0073)}]$
$= 14.52$ kN/m

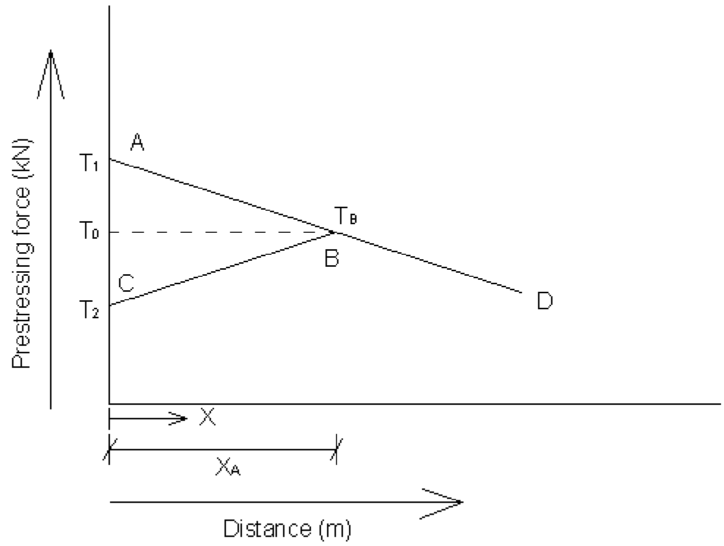


Fig 1.3.13 Loss of prestress Due to Friction and Wedge Pull-in

Note : When distance X is less than Xa the magnitude of prestress force after friction and wedge pull in losses will be the ordinate of line CB and when k is greater than Xa prestress force in the cable will be the ordinate of line BO.

Length of cable subjected to pre-stress loss due to wedge pull-in,

$$X_A = \sqrt{(\Delta wp \text{ Eps Aps})/\Delta P}$$

Here,

$\Delta wp = 8$ mm

Eps $= 197 \times 10^6$ kN/m^2

Aps $= 98.71 \times 10^{-6} \times 12 = 1184.8 \times 10^{-6}$ m^2

So,

$$X_A = \sqrt{(0.008 \times 197 \times 10^6 \times 1184.4 \times 10^{-6})/14.52} = 11.34 \text{ m}$$

Now,

$$P_{XA} = P_i\, e^{-(Kx + \mu\alpha)}$$
$$= 1652 \times e^{-(0.007 \times 11.34 + 0.25 \times 0.0829)}$$
$$= 1494.63 \text{ kN}$$

Cable force at mid,

When, $X_A = 20.325$ m

$$\alpha_V = X_A/R = 20.325/136.78 = 0.1485 \text{ rad}$$

Now,

$$P_{XA} = P_i\, e^{-(Kx + \mu\alpha)}$$
$$= 1652 \times e^{-(0.007 \times 20.325 + 0.25 \times 0.1485)}$$
$$= 1380.70 \text{ kN}$$

Table: Calculation of sag & radius of curvature

Cable no	Vertical sag, d_r (m)	Radius of curvature, R, (m)
1	0.26	794.43
2	0.96	215.16
3	1.31	157.67
4	0.610	338.61
5	1.51	136.79
6	1.71	12079

Table: Calculation of wedge pull in and friction loss

Cable no	Initial pre-stress force, P_i, (kN)	α = X/R for X=1, (rad)	Loss of pre-stress force per unit length, ΔP, (kN/m)	Distance of wedge pull-in X_A, (m)	α = X_A/R for X_A distance, (rad)	Cable force at X_A, P_{XA}, (kN)	α = X/R for X=20.325 m distance, (rad)	Cable force at X=20. 325m P_x (kN)	Loss of pre-stress %
1	1652	0.0013	12.05	12.45	0.0157	1508.2	0.026	1424	13.80
2	1652	0.0048	13.41	11.80	0.0548	1500.3	0.094	1400	15.25
3	1652	0.0031	14.11	11.50	0.0729	1496.7	0.128	1388	15.98
4	1652	0.0058	12.71	12.12	0.0358	1504.1	0.060	1412	14.52
5	1652	0.0068	14.51	11.34	0.0829	1494.7	0.149	1381	16.40
6	1652	0.0078	14.92	11.19	0.0926	1492.6	0.168	1374	16.80

$\sum P_X$ = 8998.2 kN $\sum$Loss = 92.75

Average loss of pre-stress = 92.75/6 = 15.5 %

b) Calculation of loss of prestress due to elastic shortening:

Elastic shortening, $\Delta f_{PES} = \left\{\frac{(N-1)}{2N}\right\} \times \left(\frac{E_P}{E_{ci}}\right) \times f_{cgp}$

Here,

N = number of identical pre stressing tendon

f_{cgp} = Sum of concrete stresses at the centre of gravity of pre-stressing tendon due the pre-stressing force after jacking and self weight of the member at the section of maximum moment.

fcgp value may be calculated using a steel stress reduced below the initial value by a margin depends on elastic shortening relaxation and friction effect.

$f_{cgp} = P/A+Pe^2/I-M_{Ge}/I$

P = Effective prestress force
= P_i – (% of elastic shortening + relaxation + friction effects) x P_i
= 1652-{(3+0+16) x 1652}/100
= 1338.12 kN

Assume,
Elastic shortening loss = 3%
Relaxation loss = 0%

M_G = Moment for self wt of girder = 4693.79 kN-m
e = 1167.31-165 = 1002.31 mm = 1.00 m
A = 0.984 m^2
I = 0.604 m^4
N = Number of initial pre-stressing tendon = 6

Now,
$f_{cgp} = P/A+Pe^2/I-M_Ge/I$
$= 1338.12 \times 6/0.984 + 1338.12 \times 6 \times (1)^2/0.604 – 4693.79 \times 1/0.604$
= 8159.26 +13292.58 -7771.18
= 13680 kN/m^2
= 13.68 MPa

$\Delta f_{PES} = \{(N-1)/2N\}(Ep/Eci)f_{cgp}$
$= \{(6-1)/(2 \times 6)\} \times (197 \times 10^3/24952) \times 13.68$
= 45.66 MPa

So, Elastic shortening loss = $45.66 \times 10^3 \times 100 \times 1184 \times 10^{-6}/1652$
= 3.27%

1.3.8.2 Determination of Long Term Losses:

a) Shrinkage loss:

Loss of pre stress due to shrinkage in post tensioned member is calculated by using the following equation:

$$\Delta f_{SH} = 83\gamma_h \gamma_{st}$$

Here,
$\gamma_{st} = 35/(7 + f'_{ci})$
$\gamma_h = 1.7 - 0.01H$

Here,
γ_{st} = Correction factor for specific concrete strength at time of prestress transfer to the concrete member.

γ_h = Correction factor for relative humidity of the ambient air.
H = The average annual ambient relative humidity (%)
= 60% (Assume)
$f'_{ci} = 0.75 \times 35$
$= 26.25$

So,

$\gamma_{st} = 35/(7 + 26.25) = 1.0526$
$\gamma_h = 1.7 - 0.01 \times 60 = 1.1$

$\Delta f_{SH} = 83\gamma_h \gamma_{st}$
$= 83 \times 1.1 \times 1.0526$
$= 96.1$ MPa

Shrinkage loss % $= 96.1 \times 10^3 \times 100 \times 1184.4 \times 10^{-6}/1652$
$= 6.9\%$

b) Creep loss in post tension member:

$\Delta fcr = 10.0(f_{pi} A_{ps}/A_g) \gamma_h \gamma_{sT}$

Here,

f_{pi} = Pre-stressing steel stress immediately prior to transfer (MPa)
$= 0.75 \times 1860$
$= 1395$ MPa
$A_{ps} = 98.7 \times 12 \times 6$
$= 7106.4 \text{ mm}^2$
$A_g = 1.30 \times 10^6 \text{ mm}^2$

So,

$\Delta fcr = 10.0(f_{pi} A_{ps}/A_g) \gamma_h \gamma_{sT}$
$= 10 \times (1395 \times 7106.4/1.30 \times 10^6) \times 1.1 \times 1.0526$
$= 88.30$ MPa

Creep loss % $= 88.30 \times 10^3 \times 100 \times 1184.4 \times 10^{-6}/1652$
$= 6.33\%$

c) Steel relaxation loss:

Referring from AASHTO designation M204-89,
Relaxation loss of pre-stressing steel after 100 hr = 2.5%
Relaxation loss of pre-stressing steel after 1000 hr = 3.5%

1.3.9 Schedule of Stresses:

a) Moment due to self wt. of girder,

$M_G = 4693.79$ kN-m

Stress at girder bottom, σ_b= 4693.79 x 1.167/(0.604 x 1000)
= 9.07 MPa

Stress at girder top, σ_t = 4693.79 x 1.232/(0.604 x 1000)
= 9.57 MPa

Eccentricity = 1167.31 – 167 = 1002.31 mm

b) Moment due to pre-stress = 1652 x 6 x 1.0
= 9912 kN-m

So,

Stress at girder bottom, σ_b= 9212 x 1.167/(0.604 x 1000)
= 19.15 MPa

Stress at girder top, σ_t = 9912 x 1.232/(0.604 x 1000)
= 20.22 MPa

c) Moment due to cross girder = 599.4 kN-m

So,

Stress at girder bottom, σ_b = 599.4 x 1.167/ (0.604 x 1000)
= 1.158 MPa

Stress at girder top, σ_t = 599.4 x 1.232/(0.604 x 1000)
= 1.22 MPa

d) Moment due to deck slab = 1776 kN-m

So,

Stress at girder bottom, σ_b= 1776 x 1.167/ (0.604 x 1000)
= 3.46 MPa

Stress at girder top, σ_t = 1776 x 1.232/ (0.604 x 1000)
= 3.62 MPa

e) Moment due to wearing course = 425.5 kN-m

So,

Stress at girder bottom, σ_b= 425.5 x 1.488/ (1.03 x 1000)
= 0.615 MPa

Stress at girder top, σ_t = 425.5 x 0.911/ (1.03 x 1000)
= 0.38 MPa

Here,

$S_{td} = 1.09 \times 10^9 \ mm^3$

$\sigma_{td} = 425.5 / (1.09 \times 1000)$

$= 0.39$ MPa

f) Total live load moment = 3301.58 kN-m

So,

Stress at girder bottom, $\sigma_b = 3301.58 \times 1.488/(1.03 \times 1000)$

$= 4.77$ MPa

Stress at girder top, $\sigma_t = 3301.58 \times 0.911/(1.03 \times 1000)$

$= 2.92$ MPa

Here,

$S_{td} = 1.09 \times 10^9 \ mm^3$

$\sigma_{td} = 3301.58 / (1.09 \times 1000)$

$= 3.028$ MPa

Loss of pre-stress due to creep modified differential shrinkage:

Let,

Creep modified differential shrinkage, $A_{sh} = 100 \times 10^{-6}$

Force due to creep modified differential shrinkage,

$P_{diff} = 100 \times 10^{-6} \times 1.56 \times 0.2 \times 28.397 \times 10^6$

$= 886$ kN

Stress at deck top, $= [886/1.30 + 886 \times 1.007/1.09 – 100 \times 10^{-6} \times 28.397 \times 10^6] \times 1/1000$

$= -1.33 \ N/mm^2$

Stress at girder bottom, $\sigma_b = [886/1.30 - (886 \times 1.007 \times 1.488)/1.03] \times 1/1000$

$= -0.607 \ N/mm^2$

Stress at girder top, $\sigma_t = [886/1.30 + (886 \times 1.007 \times 0.911)/1.03] \times 1/1000$

$= 1.48 \ N/mm^2$

Table: Schedule of Stresses

Sl no.	Description of item	Axial force kN	Bending moment (kN-m)	Stress at girder bottm (N/mm2)	Stress at girder top (N/mm2)	Stress at deck top (N/mm2)
01	Moment due to self wt of girder		4693.79	-9.07	+9.57	
02	Axial force due to pre stressing(1652x6) Moment due to prestressing (1652 x 6 x 1.00)	9912	 9912	+10.07 +19.15	+10.07 -20.22	
03	Loss of pre stress due to friction = 16 %			-4.67	+1.62	

04	Loss of prestress due to elastic shortening (3.27 %)			-0.967	+0.332	
Sub Total 1: Allowable compression,0.6f'ci = 15.75 N/mm2 Allowable tension -0.25√f'ci = -1.28 N/mm2				14.51 OK	1.372 OK	
05	Loss of prestress due to relaxation loss 0 %			-	-	
06	Loss of prestress due to shrinkage 6.9%			-2.016	+0.700	
Sub total 2				12.5 OK	2.03 OK	
07	Moment due to X girder		599.4	-1.158	+1.22	
08	Moment due to deck		1776	-3.46	+3.62	
09	Moment due to wc		425.5	-0.615	+0.38	+0.39
10	Moment due to LL		3301.58	-4.77	+2.92	+3.028
11	Final relaxation (3.5%)			-1.022	+0.355	
12	Stress due to creep modified diff, shrinkage			-0.607	+1.48	-1.33
13	Creep loss (6.33%)			-1.85	+0.642	
Total stress at service Load condition Allowable compression, 0.45f'c = 15.75N/mm^2 Allowable tension, -0.5√f'c = -2.96 N/mm^2				-0.982 OK	12.65 OK	2.088 OK

1.3.10 Calculation of Moment Capacity:

Average stress in pre-stressing steel when fpe ≥.5 fpu

$f_{ps} = f_{pu}(1 - K\, c/d_p)$ [Ref. AASHTO 07, Eq. 5.7.3.1.1-1]

f_{ps} = Average stress in prestressing steel

f_{pu} = specified tensile strength of pre-stressing steel = 1860 MPa

K $= 2(1.04 - f_{py}/f_{pu})$ [Ref. AASHTO 07, Eq.5.7.3.1.1-2]
= 0 .28 for low relaxation strand [Ref. AASHTO 07, Table 5.7.3.1.1-1]

d_p = distance from extreme compression fiber to centroid of the pre stressing tendon
= $h - y_{bs}$ = 2600 – 165 = 2435 mm

[y_{bs} = distance between the c.g of the prestressing cables and bottom concrete fiber of the beam = 165]

c = distance between the neutral axis and the compression face

Assuming rectangular behavior:

$c = (A_{ps}\, f_{pu} + A_s f_y - A_s' f_y') / (0.85 fc' \beta_1 b + kA_{ps}\, f_{pu}/d_p)$

$= (12 \text{ x} 98.71 \text{x } 6 \text{x } 1860 + 0 - 0) / \{(0.85 \text{x} 25 \text{x} 0.85 \text{x} 1850) + (0.28 \text{ x } 12 \text{ x } 98.71 \text{ x } 6 \text{ x } 1860/2435)\}$

$= 13217904/(33415.63 + 1519.92) = 378.35 > h_f \text{ mm}$

$= 378.35 > 200 \text{ mm}$ **Not ok**

Assuming T-section behavior:

So,

$c = (A_{ps}\, f_{pu} + A_s f_s - A_s' f_s' - 0.85 f'_c (b - b_w) h_f) / (0.85 f_c' \beta_1 b_w + kA_{ps} f_{pu}/d_p)$

$$= \frac{(12 \text{ x } 98.71 \text{x } 6) \text{ x } 1860 + 0 - 0 - 0.85 \text{ x } 35(1564 - 300) \text{ x } 200}{0.85 \text{ x } 35 \text{x } 0.85 \text{ x } 300 + 0.28(12 \text{ x } 6 \text{ x } 98.71) \text{ x} 1860/2435}$$

$= 625.63$ mm

$a = \beta_1 \text{x } c = 0.85 \text{ x } 625.63 = 531.78$ mm

$f_{ps} = f_{pu}(1 - k\, c/d_p)$

$= 1860(1 - 0.28 \text{ x } 625.63/2435) = 1726.19$ MPa

$M_r = \Phi M_n$ where M_n = Nominal resisting moment

$\Phi = 1$ [Ref. AASHTO 07, Art.5.5.4.2]

$M_n = A_{ps} f_{ps}(d_p - a/2) + A_s f_s(d_s - a/2) + A_s' f_s' (d_s' - a/2) + 0.85 fc' (b - b_w) h_f (a/2 - h_f/2)$

$= (12 \text{x} 6 \text{x} 98.71) \text{x} 1726.19(2435 - 531.78/2) + 0 + 0 + 0.85 \text{x} 35(1564 - 300) \text{x} 200\, (531.78/2 - 200/2)$

$= (2.66 \text{x} 10^{10} + 1.25 \text{x} 10^9)/10^6$

$= 27850$ kN-m

$M_r > M_u$

OK

1.3.11 Calculation of Non-prestressing Reinforcement:

Surface reinforcement = 0.003bt [Ref. AASHTO 07, Art. 5.6.3.6]

= 0.003 x 1000 x 300/2 = 450 mm^2 / m

Providing T-10

Spacing = 1000 x 78.5/450 = 175 mm

Total no. of surface bar needed = 2300/175 x 2 = 26

1.3.12 Calculation of Shear Reinforcement: (Ref. AASHTO'07, Art. 5.8.3.3)

The nominal shear resistance Vn shall be determined as lesser of

$$V_n = V_c + V_s + V_p \ \ldots\ldots\ldots\ldots(1)$$

$$V_n = 0.25\ fc'b_v\, d_v + V_p \ \ldots\ldots(2)$$

Where, $Vc = 0.083\beta\sqrt{fc'}b_v d_v$

b_v = effective web width as the minimum web width within The depth dv
= 300 mm

d_v = effective shear depth taken as the distance measured perpendicular to neutral axis between the resultant of the tensile and compressive forces due to flexure it need not be taken to be lesser than the greater of 0.9 de or 0.72h(mm)

(i) $d_v = M_n/A_sf_y = A_sf_y(d\text{-}a/2)/A_sf_y = (d\text{-}a/2) = (2435\text{-}531.78/2) = 2169.11$ mm

(ii) 0.72h = 0.72 x 2600 = 1872 mm

(iii) 0.9de = 0.9 x 2435 = 2191.5 mm

0.9de > 2169.11, Hence dv = 2191.5 mm

1.3.12.1 Calculation of β & Θ:

Here, β = factor indicating ability of diagonally cracked concrete to transmit tension as specified in

AASHTO '07 Article 5.8.3.4

θ = Angle of inclination of diagonal compressive stresses as determined in AASHTO'07 Article 5.8.3.4

v_u = Total factored shear / Area

$= [1445.74 \times 10^3 / 1.30 \times 10^6] \times 1/1000$

$= 1.112 \times 10^{-3}$

$_u/f'c = 1.112 \times 10^{-3}/35 = 3.18 \times 10^{-5} m^2$

Referred from AASHTO 07 Table: 5.8.3.4.2

θ = 36.4, β=2.23

$\therefore V_c = 0.083\beta\sqrt{f'c}\ b_vd_v$ [Ref. AASHTO 07, Art. 5.8.3.3]

$= 0.083 \times 2.23 \times \sqrt{35} \times 300 \times 2191.5$

$= 719912.48$ N

$\therefore V_s = A_vf_yd_v\mathrm{Cot}\theta/S$

Here, S = 125 mm C/C

$A_v = 113 \times 2 = 226\ mm^2$

$V_s = 226 \times 415 \times 2191.5 \times 1.36/125$

$= 2236283.74$ N

1.3.12.2 Calculation of Prestress Force Component:

Effective prestress force of Cable No. 1	P_1	=	1652-2(1652-1508.1) =1362.20 kN
Effective prestress force of Cable No. 2	P_2	=	1692-2(1652-1500.3) =1348.60 kN
Effective prestress force of Cable No. 3	P_3	=	1652-2(1652-1496.7) =1341.38 kN
Effective prestress force of Cable No. 4	P_4	=	1652-2(1652-1504.1) =1356.20 kN
Effective prestress force of Cable No. 5	P_5	=	1652-2(1652-1494.6) =1337.26 kN
Effective prestress force of Cable No. 6	P_6	=	1652-2(1652-1492.6) =1333.16 kN

V_{P1} =	1362.2 x 0.0256	= 34.87 kN	θ_1=1.47
V_{P2} =	1348.6 x 0.0941	= 126.92 kN	θ_2=5.40
V_{P3} =	1341.4 x 0.1279	= 171.60 kN	θ_3=7.35
V_{P4} =	1356.2 x 0.060	= 81.37 kN	θ_4=3.44
V_{P5} =	1337.26 x 0.1469	= 196.44 kN	θ_5=8.45
V_{P6} =	1333.16 x 0.1659	= 221.17 kN	θ_6=9.55

Total V_P = 832.37 kN

V_n = (719912.48+2236283.74+832370)/1000

= 3788.56 kN

Again V_n = (0.25 x 35 x 2191.5 x 300+832370)/1000

= 6585 kN

Hnce V_n = 3788.56 kN>1445.74 kN, Hence OK.

1.3.12 Calculation of Deflection:

Uniformly distributed load due to prestressing force at transfer,

c.g of cables at end = (1x350+1x700+1x1050+1x1400+1x1750+1x2100)/6

= 1225mm

c.g of cables at mid = 165mm

$w = 8Fh/L^2$

$= 8 \times 1652 \times 6 \times 1.060/ 40.65^2$; Here, h = 1225-165 = 1060mm

= 51 kN/m

Deflection due to prestressing force at transfer,

$\Delta_P = 5wL^4/384EI$

E_{ci} = modulus of elasticity of concrete at transfer = 24.59 x 10^6 kN/m^2

I = moment of inertia of non-composite girder = 0.604 m^4

$\Delta_P = (5 \times 51 \times 40.65^4 / 384 \times 24.59 \times 10^6 \times 0604) \times 1000 = 122$ mm

Net deflection due to pretress = 122- [(0.058 x 1652 x 6) x 40.65^2 / 8 x 24.59 x 10^6 x 0.604] x 1000

= 114 (↑)

Deflection due to beam self wt,

$\Delta_g = 5wL^4/ (384E_{ci}I)$

w = beam self wt. = 27.86 kN/m

$\Delta_g = (5x27.86 \text{ x } (40.65)^4) / (384x24.59x10^6x0.604)$

$= 0.067$ m = 67 mm (↓)

Deflection after transfer = 114-67 = 47mm

Deflection due to X-girder,

$\Delta_{xg} = (90 \text{ x } 40.65^3 / 48 \text{ x } 28.39 \text{ x } 10^6 \text{ x } 0.604) \text{ x } 1000 = 7.34$mm

Deflection due to slab wt,

$\Delta_s = 5wL^4/(384E_cI)$

E_c = modulus of elasticity of concrete at service stage = $28.39x10^6$ kN/m^2

w = slab wt. = 8.88 kN/m

$\Delta_s = [(5x8.88x\ (40.65)^4) / (384x28.39x10^6x1.03)] = 10.8$mm (↓)

Deflection due to wearing course,

$\Delta_{wc} = 5wL^4/(384E_cI_c)$

w = wearing course wt. = 2.13 kN/m

$\Delta_{wc} = (5x2.13x\ (40.65)^4)/\ (384x28.39x10^6x1.03) = 2.62$ mm (↓)

Deflection due to lane load,

$\Delta_{LL} = 5wL^4/(384E_cI_c)$

w = lane load = 5.45 kN/m

$\Delta_{LL} = (5x5.45x\ (40.65)^4)/\ (384x28.39x10^6x1.03)$

= 6.15 mm (↓)

Deflection due to truck & impact load,

$\Delta_{WL} = PL^3/\ (48E_cI_c)$

P = wheel load = (145x2+35) x DFM x Dynamic allowance

= 325x0.586x1.33

= 253.3 kN

$\Delta_{WL} = (253.3 \text{ x } (40)^3x1000)/(48 \text{ x } 28.395 \text{ x } 10^6 \text{ x } 1.03)$

= 11 mm (↓)

Allowable deflection for live load = L/800 (Ref. AASHTO 07 Art. 2.5.2.6.2)

= (40.65) x 103/800 = 50.82 mm

Total live load deflection = 11+6.15 = 17.15 mm<50.82 mm

Hence OK.

1.3.14 Design of End block

Taking bursting force = 30% of jacking force

[Ref. AASHTO 07, Art.5.10.9.6]

$T_{burst} = (.30 \text{ x } 12 \text{ x } 98.7 \text{ x } 1860)/100$

= 661kN

Yield stress of steel, f_y = 415 Mpa

Allowable stress, fs = 207.5 Mpa

$A_t = 661x\ 10^3/207.5 = 3186$ mm^2 [Ref. AASHTO 07, Art.10.9.3.2]

Reinforcement is distributed in 1.5x (lateral dimension) = 1.5(550+650)/2 = 900 mm
Hence 32-T12-125 vertical bar is provided.

1.3.15 Cable Profile Details

12K-13 Cable requires duct size of minimum 55mm (inner). Here, 65 mm of duct size is used for cable profiling.

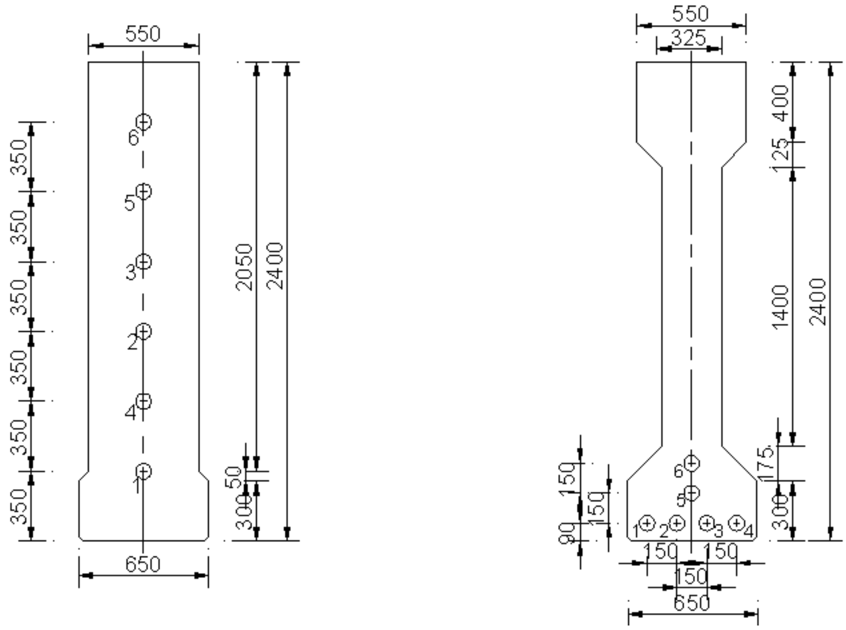


Fig 1.3.14 Cable Profile of End and Mid-section of PC girder

Distance of (65+50) =115 is required as minimum distances between the ducts. In this case, 150mm is provided between the ducts in horizontal direction. Parabola equations are used for profiling the cables. Vertical coordinates of the cables at different length are located using the following equation

$$Y = 4 \text{ x } X \text{ x } H \text{ x } (L-X)/L^2$$

Here, X = Distance in the long direction
H = Vertical distance of the cables in mid and end section
L = Total Length of Girder

And, Horizontal Co-ordinates are calculated using the following equation

Z = Z coordinate of end section+ 4 x (L-2x End Block) x Z coordinate of mid section x {(L-2x End Block-(X coordinate at desired section-end block)}/ (L-2x End Block) ^{2}

The table is provided below where the cable profile ordinates of Cable no 1 are calculated.

Table: Cable Profile Ordinates of Cable 1.

Ordinates	**X=0**	**X=L/20**	**X=L/10**	**X=3L/20**	**X=L/5**	X is measured from End section.
X	0	2032.5	4065	6097.5	8130	
Y	350	300.60	256.40	217.40	183.60	
Z	0	0	-39.85	-83.25	-120.85	
Ordinates	**X=L/4**	**X=3L/10**	**X=7L/20**	**X=2L/5**	**X=9L/20**	**X=L/2**
X	10162.5	12195	14227.5	16260	18292.5	20325
Y	155.00	131.6	113.40	100.40	92.60	90.00
Z	-152.67	-178.71	-198.96	-213.42	-222.10	-225

CHAPTER-2
ELASTOMERIC BEARING DESIGN
(SI UNITS)

METHOD B - STEEL - REINFORCED ELASTOMERIC BEARINGS
(AASHTO 2007 ART. NO. 14 .7.5)

2.1 Initial Design Data

Span = 25.0 m
Dead Load, P_D = 434 kN (due to self wt. of deck, girder, x-girder & wearing course at service condition)
Live Load, P_{LL} = 340 kN (due to truck load without impact & lane load at service condition)
Horizontal Movement of Bridge Superstructure,Δ_0 = 7 mm [Detailed calculations are provided hereafter]

Detailed Calculation of Horizontal Movement of Bridge Superstructure, Δ_0

Δ_T = 0.5 x L x α x Δt
= (25000/2) x 12 x 10^{-6} x 30
= 4.50 mm

Δ_{SH} = ε_{SH} x L/2
= 200 x 10^{-6} x 25000/2
=2.50 mm

Δ_o = 4.5 + 2.5
= 7.0 mm

Axis of Pad Rotation: Transverse
Calculated Rotation = 0.005 Radians
Rotation Construction Tolerance = 0.005 Radians (Ref. AASHTO 2007, Art. 14.4.2.1)
Design Rotation, θs = 0.008 Radians (Obtained from STAAD analysis)
Bearing Shape: Rectangular

Note:

1. Bearing is Subject to Shear Deformation.
2. Bridge Deck is Not Fixed against Horizontal Translation

2.2 Bearing Geometry

Flange Width = 460 mm
Bearing Width, W = 350 mm
Flange Width ≥ W
460 mm ≥ 350 mm

OK

Total Unfactored Compressive Load, P_T = (434+340) x 1000
= 774000 N.
Minimum Required Area of Bearing, A_{min} = 774000/11 = 70364 mm^2 (subject to shear deformation)
(Ref. AASHTO 2007, Art. 14.7.5.3.2)
Minimum Bearing Length, L_{min} = 70634/350 = 201 mm.
Bearing Length, L = 250 mm.

Bearing Length, L ≥ Minimum Bearing Length, L_{min}
250 mm ≥ 201 mm

OK

Bearing Provided, A = 350 x 250
= 87500 mm^2.

2.3 Shear Deformation: (REF.AASHTO 2007, ART. 14.7.5.3.4)

Maximum Total Shear Deformation of Elastomer at Service Limit,
$\Delta s = \Delta_0 = 7.0$ mm.
Factored shear deformation $2\Delta s = 2 \times 7.0 = 14.0$ mm
Elastomeric Layer Thickness, $h_{ri} = 10.0$ mm
Thickness of top and Bottom Cover Layers (each),
$h_{cover} = 5.0$ mm

$h_{cover} \leq 0.7\ h_{ri}$
(Ref. AASHTO 2007, Art. 14.7.5.1)

$5.0 \leq 0.7 \times 10.0 = 5.0 \leq 7.0$

OK

Number of Interior Elastomeric Layers, $n_{int} = 4$
(Excluding Exterior Layer Allowance)

Total Elastomer Thickness = $h_{rt} = 2h_{cover} + n_{int}h_{ri} = 2 \times 5 + 4 \times 10 = 50$ mm.
$h_{rt} \geq 2\Delta s$ (Ref. AASHTO 2007, Art.14.7.5.3.4-1)
$50 \geq 2 \times 7.0 = 50 \geq 14.0$

OK

2.4 **Compressive Stress:** (REF. AASHTO 2007, ART. 14.7.5.3.2)

Service Average Compressive Stress (Total Load), $\sigma_s = P_T/A$
$= 774000/87500$
$= 8.85$ MPa

Service Average Compressive Stress (Live Load), $\sigma_L = P_{LL}/A$
$= 340/87500$
$= 0.004$ MPa

Rectangular Shape Factor, $S_i = LW/2h_{ri}\,(L+W)$
(Ref. AASHTO 2007, Art. 14.7.5.1-1)
$= 250 \times 350/\{2 \times 10 \times (250+350)\}$
$= 7.29$

Shear Modulus of Elastomer, G = 0.9 MPa

Now, $0.9 \leq G \leq 1.38$ (Ref. AASHTO 2007, Art. 14.7.5.2)
$= 0.9 \leq 0.9 \leq 1.38$

OK

For Bearings Subject to Shear Deformation:

$\sigma_s \leq 1.66GS$ (Ref. AASHTO 2007, Art.14.7.5.3.2-1)
$= 8.85 \leq 1.66 \times 0.9 \times 7.29$
$= 8.85 \leq 10.89$

OK

Now, $\sigma_s \leq 11$ (Ref. AASHTO 2007, Art.14.7.5.3.2-1)
$= 8.85 \leq 11$

OK

Now, $\sigma_L \leq 0.66GS$ (Ref. AASHTO 2007, Art.14.7.5.3.2-2)
$= 0.004 \leq 0.66 \times 0.9 \times 7.29$
$= 0.004 \leq 4.33$

OK

2.5 **Combined Compression and Rotation:**

(REF. AASHTO 2007, ART. 14.7.5.3.5)

RECTANGULAR BEARINGS:

B = Length of Pad = 250 mm
Exterior Layer Allowance, $n_{ext} = 0$ (Ref. AASHTO 2007, Art. 14.7.5.2)

Equivalent Number of Interior Elastomeric Layers, n = $n_{int} + n_{ext}$
= 4.0 + 0 = 4.0

$$\sigma_s > 1.0GS\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2$$ (Ref. AASHTO 2007, Art. 14.7.5.2)

= 8.85 > 1.0 x 0.9 x 7.29 x (0.008/4) x $(250/10)^2$
= 8.85 > 8.20

OK

Check shear deformation: (Ref. AASHTO 2007, Art. 14.7.5.3.5-2)

$$\sigma_s < 1.875GS\left[1 - 0.200\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$$

= 8.85 < 1.875 x 0.9 x 7.29 [1-{0.2 x (0.008/4) x $(250/10)^2$}]
= 8.85 < 9.23

OK

2.6 Stability: (REF. AASHTO 2007, ART. 14.7.5.3.6)

For free horizontal translation: 2A ≤ B

$$A = \frac{1.92\frac{h_{rt}}{L}}{\sqrt{1+\frac{2L}{W}}} = 1.92 \text{ x } (50/250)/\sqrt{\{1+ (2 \text{ x } 250/350)\}} = 0.246$$
(Ref. AASHTO 2007, Art. 14.7.5.3.5-2)

2A = 2 x 0.246 = 0.493

$$B = \frac{2.67}{(S+2.0)\left(1+\frac{L}{4.0W}\right)} = 2.67/ [(7.29+2.0) \text{ x } \{1+250/(4.0 \text{ x } 350)\}]$$

= 0.244
(Ref. AASHTO 2007, Art. 14.7.5.3.5-3)

Now, 2A ≤ B
0.493 ≤ 0.244

NOT SATISFIED

As, this condition is not satisfied, the following equation is followed from AASHTO 2007.

Bridge Deck Free to Translate Horizontally:

$$\sigma_s \leq \frac{GS}{2A - B}$$ (Ref. AASHTO 2007, Art. 14.7.5.3.6-4)

$= 8.85 \leq 0.9 \times 7.29/(0.493-0.244)$
$= 8.85 \leq 26.36$ MPa

OK

REINFORCEMENT: (REF. AASHTO 2007, ART. 14.7.5.3.7)

Service Limit State:

Min. Yield Strength of Steel Reinforcement, $f_y = 250$ MPa
Thickness of Steel Reinforcement = h_s

$$h_{s\,min} = \frac{3.0 h_{max} \sigma_s}{F_y} = \quad 3.0 \times 10 \times 8.85/250 = 1.061 \text{ mm}$$

(Ref. AASHTO 2007, Art. 14.7.5.3.7-1)

Fatigue Limit State:

Constant Amplitude Fatigue Threshold = ΔF_{TH} = 165 MPa

$$h_{s\,min} = \frac{2.0 h_{max} \sigma_L}{\Delta F_{TH}} = \times 0.004 \times 10/165 = 0.0005 \text{ mm}$$

(Ref. AASHTO 2007, Art. 14.7.5.3.7-2)

Required Minimum Reinforcement Thickness, $h_{s\,min}$ = 1.061 mm
Reinforcement Thickness, h_s = 3.0 mm
Now, $h_s \geq h_{s\,min}$
$= 3.0 \geq 1.061$

OK

2.7 Final Design Summary:

Bearing Width, W	= 350 mm
Bearing Length, L	= 250 mm
Elastomeric Layer Thickness, h_{ri}	= 10 mm
Thickness of top and Bottom cover Layers (each), h_{cover}	= 5.0 mm
Number of Interior Elastomeric Layers, n_{int} (Excluding Exterior Layer Allowance)	= 4
Total Elastomer Thickness, h_{rt}	= 50 mm
Reinforcement Thickness, hs	= 3 mm
Total Bearing Thickness	= h_{rt} + hs(n_{int}+1)
	= 50 + (4+1) x 3.0
	= 65 mm

CHAPTER 3

SUBSTRUCTURE & FOUNDATION

3.0 STRUCTURAL DESIGN OF ABUTMENT-WING WALL AND PILE CAP

3.1 Model Details

3.1.1 General

STAAD.Pro 2006 has been used as a data processing tool for design purpose. Full view of 8.0m height abutment structural model is presented here along with load case details, pile reactions and node displacement summary reports. Detailed partial models of abutment wall and wing wall are described with plate no's, orientation of axis and stress distribution of plates by contour map and tabular forms. Node displacement summary from the STAAD.Pro analysis is attached with the rest of the report. AASHTO 07 codes have been followed for detailing of reinforcement for all the elements of substructure.

Abutment height 8.0m model with 30 piles and counterfort at wing wall with superstructure load of 25.0m (c/c bearing) is chosen for presenting the structural design example of Abutment-Wing wall and Pile Cap.

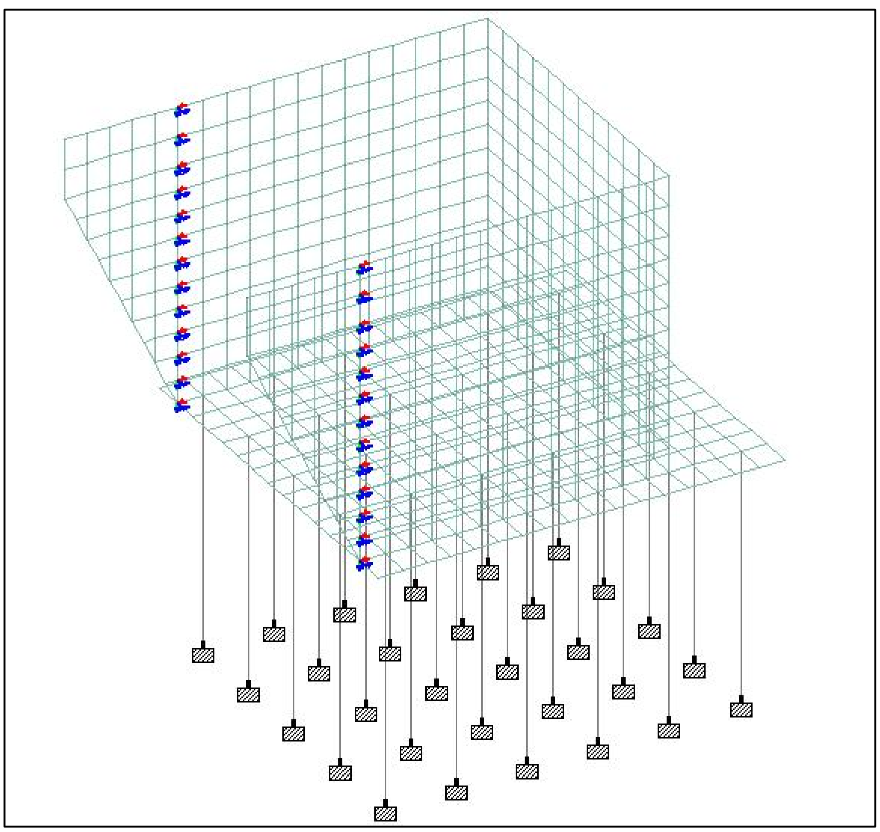


Fig 3.1 Full View of Substructure Model from STAAD.Pro

3.1.2 Load Case Details & Node Displacement Summary

Load case details with basic and combination cases and node displacement summary are provided in Appendix-"A" obtained from STAAD.Pro model.

3.1.3 Pile Reaction

Pile reactions at nodes of 30 piles are given below which are based on inelastic analysis and thus shows flexible distribution of pile reaction.

Table 3.1 Pile Reactions at Nodes for Service Load from STAAD.Pro Model

Node	F_Y, kN	Node	F_Y, kN
76	763.274	73	567.934
79	756.742	72	540.008
69	756.528	82	539.989
75	735.689	77	522.149
85	723.402	78	511.538
66	723.311	68	511.356
80	721.816	63	483.051
70	721.807	83	482.976
65	688.46	86	470.502
57	688.237	67	470.335
74	655.032	60	442.406
71	631.374	59	418.607
81	631.303	61	418.531
64	587.836	58	367.685
84	587.57	62	367.525

Elastic analysis of pile load distribution along with stability analysis is performed using Program DPM-EX. Different components of loading on substructure and foundation are calculated for vertical load calculation.

The following table provides vertical load calculations of the abutment. The following diagram is the reference drawing for the calculations following.

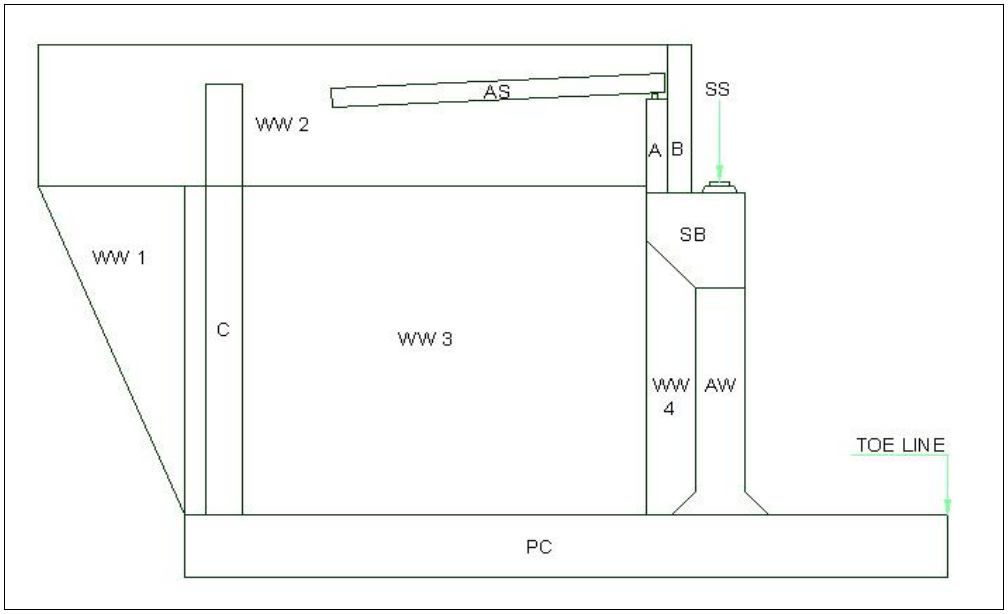


Fig 3.2 Wedges of Substructure for Calculation

Table: Vertical Load Calculation

Restoring Moment about Toe line		Volume (m^3)	Unit weight (KN/m^3)	Service Load (kN)	Lever Arm, m	Moment, kN-m
Wedge No	Name of Wedges					
AW	Abutment Wall	17.10	24.00	410.36	2.80	1149.00
SB	Seat Beam	11.89	24.00	285.47	3.10	884.94
B	Back Wall	5.33	24.00	127.86	3.30	421.95
A	Approach Slab Wall	2.48	24.00	59.47	3.58	212.61
AS	Approach Slab	8.26	24.00	198.24	5.58	1105.19
ASS	Approach Slab Support	0.05	24.00	1.12	3.58	3.99
WW1	Wing Wall, Region 1	5.41	24.00	129.74	11.15	1446.65
WW2	Wing Wall, Region 2	10.98	24.00	263.52	8.28	2180.63
WW3	Wing Wall, Region 3	27.98	24.00	671.62	7.00	4701.31
WW4	Wing Wall, Region 4	1.66	24.00	39.74	3.40	135.13
C	Counterfort	8.84	24.00	212.22	9.25	1963.04
PC	Pile Cap	118.66	24.00	2847.74	5.15	14665.88
S	Soil	430.85	18.00	7755.26	6.70	51960.27
SU	Surcharge	38.65	18.00	695.69	6.70	4661.14
SS	Superstructure			3100.00	2.83	8757.50
	TOTAL		$\sum V$	**16798**	$\sum M_R$	**85491.73**

Calculation of Lateral Forces:

Pressure due to Surcharge, P1	= 0.61 x 0.33 x 18 = 3.62	kN/m^2
Pressure due to Soil, P2	= 7.10 x 0.33 x 18 = 42.17	kN/m^2
Force due to Surcharge, F1	= 7.10 x 3.62 = 25.73	kN/m
Force due to Soil, F2	= ½ x 42.61 x 7.10 = 149.72	kN/m
Total Force	= 25.73 + 149.72 = 175.44	kN/m
$\sum H$	= 175.44 x 9.60 = 1684.26	kN

Overturning moment, $\sum M_{OT} = M_{OT}$ due to soil + M_{OT} due to surcharge

$= 149.72 \times 9.60 \times 6.80/3 \times 1.5 + 25.73 \times 9.60 \times 6.80/2$

$= 6356.25$ kN-m

Pile Load/Reaction Calculation,

C.G of Load, $x' = (\sum M_R - \sum M_{OT})/ \sum V = 4.71$ m

C.G of Pile, $y' = (5x0.65+5x2.45+5x4.25+5x6.05+5x7.85+5x9.65)/30 = 5.15$ m [N=30]

Eccentricity, e $= y' - x' = 0.44$ m

$\sum X^2 = [(X1)^2 \times 5 + (X2)^2 \times 5 + (X3)^2 \times 5\ ] \times 2$

$= [(5.15-0.65)^2 \times 5 + (5.15-0.65-1.8)^2 \times 5 + 5.15-0.65-1.8-1.8)^2 \times 5\ ] \times 2 = 283.5$

m^2

Pile Load, P $= \sum V/ \sum N \pm (\sum V \times e \times X)/ \sum X^2$

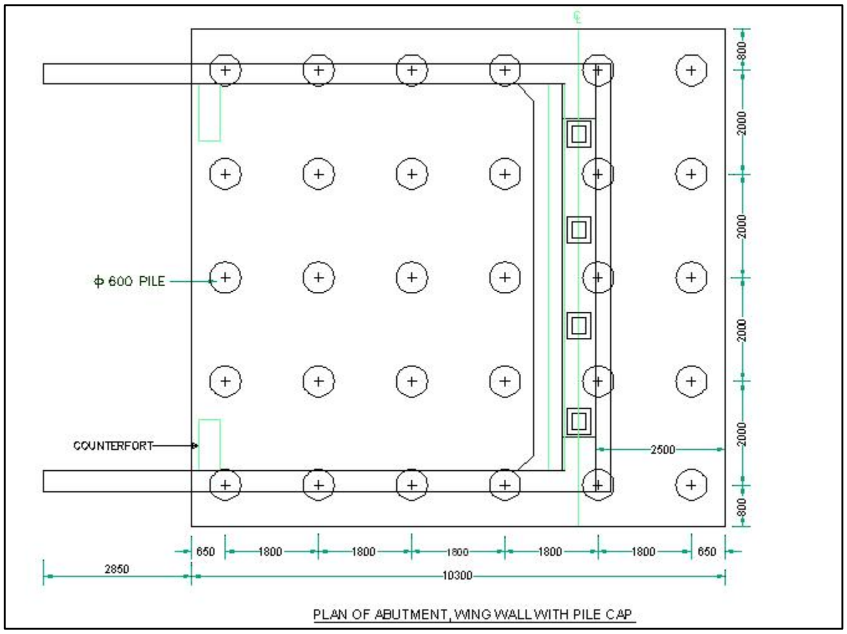


Fig 3.3 Plan of Abutment, Wing Wall with Pile Cap

At X1= 680 kN (row1 pile from toe line)

At X2= 630 kN (row2 pile from toe line)

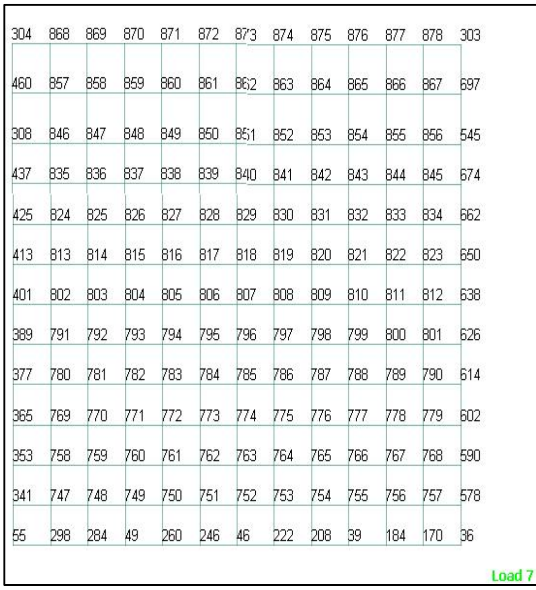
At X3= 585 kN (row3 pile from toe line)

At X4= 536 kN (row4 pile from toe line)

At X5= 490 kN (row5 pile from toe line)

At X6= 445 kN (row6 pile from toe line)

3.2 Abutment Wall Design



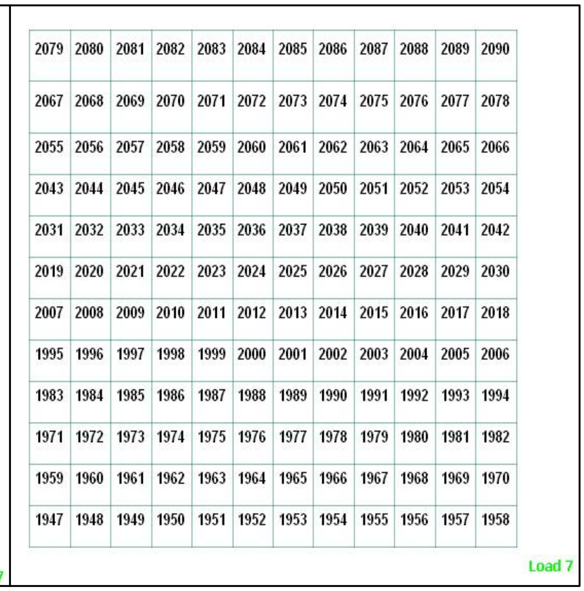


Fig 3 .4 Node no's of Abutment Wall **Fig 3.5** Plate no's of Abutment Wall

Abutment wall is designed considering plate properties with thickness 600mm. Plate and node no's of abutment wall are provided with the following figures. Moment contour map of plate according to plate orientations are also attached here. From the contour maps, higher concentration is found at the earth face or compression face.

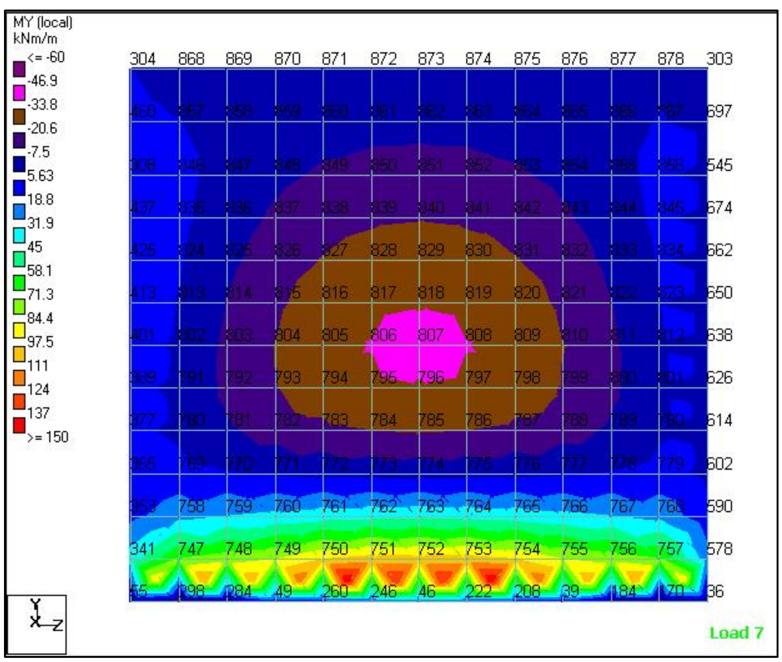


Fig 3.6 Bending Moment Contour of Abutment Wall (with node nos), M_y, kN-m

From the plate stress results, maximum flexural moment, My is found at plate no 1951 of value 146 kN-m/m along vertical direction which is found at the bottom section. With progression in height of 0.6m, certain change in moment is observed of value 65 kN-m/m. This value continues to decrease with increasing height of wing wall. For this reason, extra reinforcement is provided at the bottom only for design moment of 146 kN-m/m and for the full height of wing wall minimum reinforcement is provided following AASHTO 07 requirements. For water face or compression face, maximum moment 36 kN-m/m is considered for design reinforcement calculations and checked against minimum reinforcement requirements. The detailed calculations are provided below.

Reinforcement Design Calculation

Vertical Reinforcement For water face (W/F),

Design Moment = 36 kN-m/m

Compressive strength of Concrete, fc´= 20 MPa

Thickness of abutment wall = 0.6 m

Modulus of Rupture = 0.52 x √ fc´= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.018 m4

Cracking Moment= 2.32 x 1000 x 0.018/ (0.6/2) = 140 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 167 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 50 kN-m/m

Minimum reinforcement required is for 50 kN-m/m which is greater than 36 kN-m/m. So, design moment finally considered is 50 kN-m/m.

R16-200 is provided for design moment of 50 kN-m/m. Checking against minimum surface reinforcement and design controlling moment, it is found that R16-250 is required. However, spacing of 200 mm is provided instead of 250 mm as spacing of main/horizontal reinforcement of top bar of pile cap required is 200 mm and thus, spacing of 200 mm is chosen for convenience of working condition.

Vertical Reinforcement For earth face (E/F),

Design Moment = 146 kN-m/m

Compressive strength of Concrete, fc´= 20 MPa

Thickness of Wing Wall = 0.4 m

Modulus of Rupture = 0.5 2 x √ fc´= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.018m^4

Cracking Moment= 2.32 x 1000 x 0.018/ (0.6/2) = 140 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 167 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 195 kN-m/m

Minimum reinforcement required is for 195kN-m/m which is greater than 146 kN-m/m. So, design moment finally considered is 195 kN-m/m.

Reinforcement area required for 195 kN-m/m is 1450 mm^2. For this reason, R16-200 and R16-200 are provided. Here, curtailment has been done for one R16-200 at 2500 mm distance from top of pile cap. Maximum moment 105 kN-m/m is found for distance of about 600 mm from top of pile cap (distance of plate). Extra Reinforcement is provided for (600 + 40 times bar dia + Pile cap depth-175) mm= 2500mm. The other R16-200 has been continued for full height of wall.

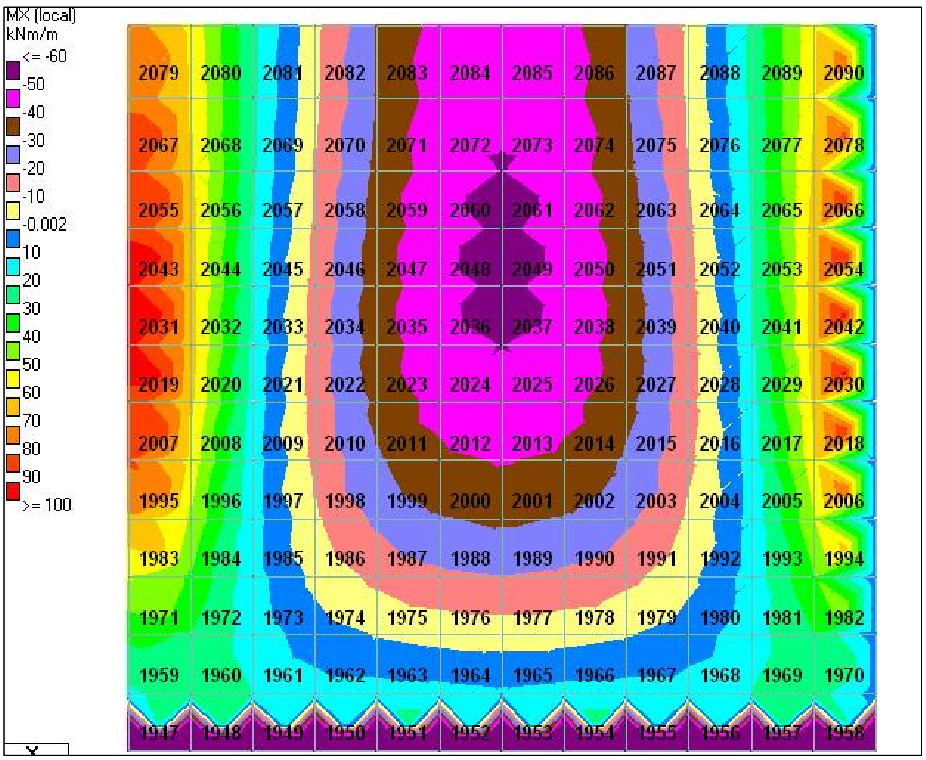


Fig 3.7 Bending Moment Contour of Abutment Wall (with plate nos), M_x, kN-m

It can be observed that maximum tensile stress is observed at plate no 2031 wherever, maximum compressive stress at plate no 2049. It is observed that tension face reinforcements are required near adjoining section between abutment wall and wing wall. Thus, extra reinforcements are provided near these two places and for the whole length minimum reinforcement is provided. Maximum compression face reinforcement is required for 59 kN-m/m.

The detailed calculations are provided below.

Reinforcement Design Calculation

Horizontal Reinforcement For water face (W/F),

Design Moment = 59 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Wall = 0.6 m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa
Moment of Inertia = b x $h^3/12 = 0.018m^4$

Cracking Moment= 2.32 x 1000 x 0.018/ (0.6/2) = 140 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 168 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 78.47 kN-m/m

Minimum reinforcement required is for 78.47 kN-m/m which is greater than 59 kN-m/m So, design moment finally considered is 78 kN-m/m. R16-200 is provided for design moment of 78 kN-m/m.

Horizontal Reinforcement For earth face (E/F),

Design Moment = 100 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Wall = 0.6 m

Modulus of Rupture = 0.62 x √ fc′= 2.32 MPa
Moment of Inertia = b x $h^3/12 = 0.018m^4$

Cracking Moment= 2.77 x 1000 x 0.018/ (0.6/2) = 140 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 168 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 133 kN-m/m

Minimum reinforcement required is for 133 kN-m/m which is greater than 100 kN-m/m. So, design moment finally considered is 133 kN-m/m.

Reinforcement area required for 133 kN-m/m is 1000 mm^2. For this calculation, R16-200 and R12-200 are provided. Here, 40 times bar diameter of length is considered in addition to the required length as curtailment length obtained from STAAD.PRO model. At the adjoining part of abutment wall and wing wall extra reinforcement of R12-200 is provided with the continuing reinforcement of R16-200 as, center stress results from STAAD.PRO shows lesser value of Mx for other plates in horizontal direction. So, T16-200 is continued for the total length of wall in horizontal direction.

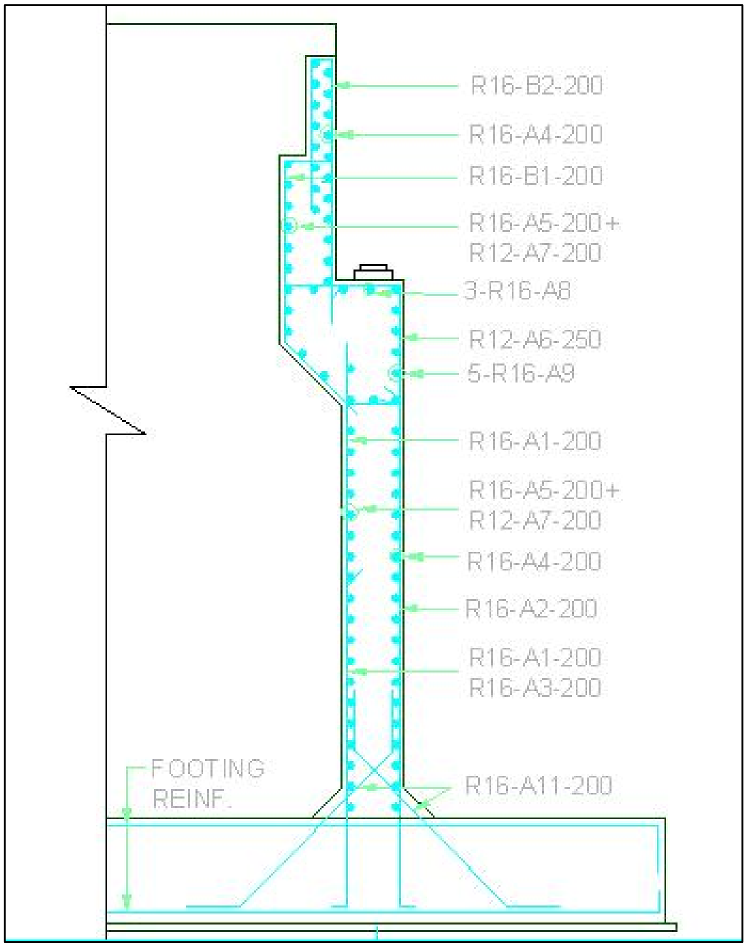


Fig 3.8 Reinforcement Details of Abutment Wall

3.3 Wing Wall Design

Wing wall is designed considering 400mm thickness. Plate and node no's of wing wall are provided with the following figures. Moment contour map of plate according to plate orientations are also attached here.

From the contour maps, higher concentration is found at the earth face or tension face. Detailed Moment according to plate no of a particular section for both x and y axis are attached here with reinforcement calculations and curtailment.

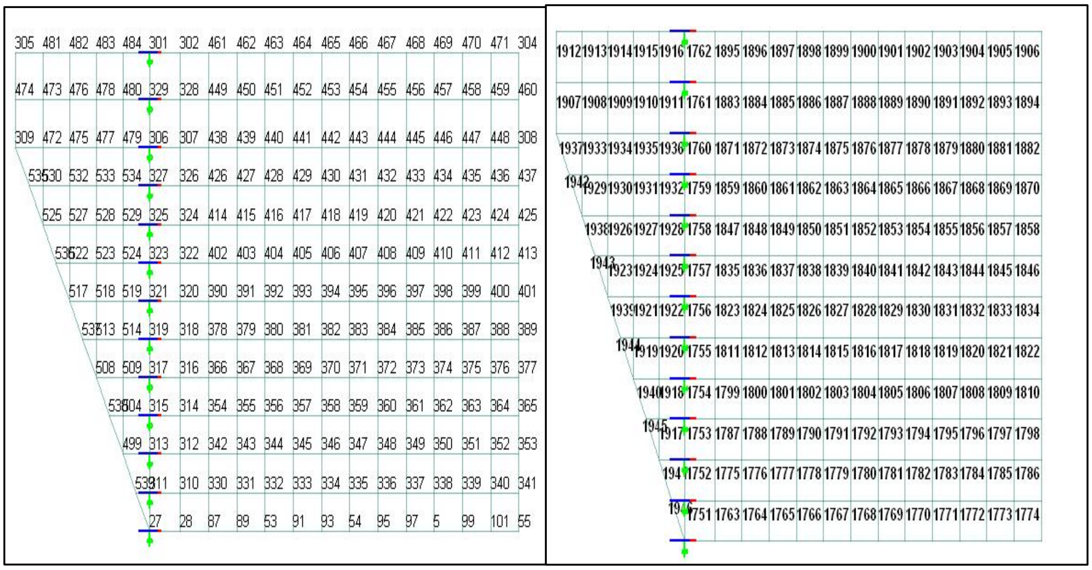
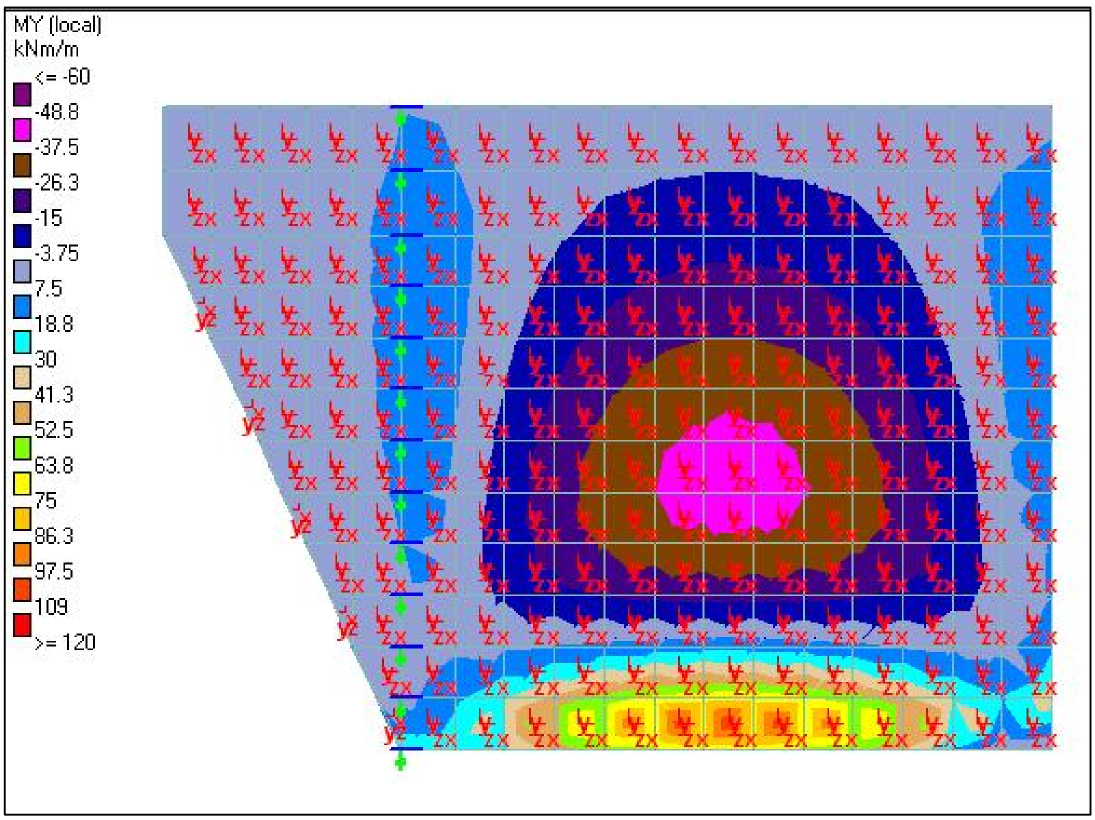


Fig 3.9 Node numbers and Plate numbers



of Wing Wall

Fig 3.10 Bending Moment Contour of Wing Wall, M_y, kN-m

Table: Bending Moment, My along section mid-section for Vertical Reinforcement Calculation

SL NO	PLATE NO(from downward to upward)	My (kN-m/m)
1	1781	-105
2	1780	-35
3	1792	-8
4	1804	+31
5	1816	+41
6	1828	+42
7	1840	+38
8	1852	+31
9	1864	+23
10	1876	+15
11	1888	+7
12	1900	+1

Here, +ve moment is for water face and –ve moment is for earth face.

It can be observed that maximum tensile stress is observed at plate no 1781 and maximum compressive stress at plate no 1828. Then again, the maximum value of tensile stress 105 kN-m/m is only found at the bottom section which continues to decrease with increasing height of wing wall. For this reason, extra reinforcement is provided at the bottom with design moment of 105 kN-m/m and for the full height of wing wall minimum reinforcement is provided following AASTO 07 requirements. For water face, maximum moment 42 kN-m/m is considered for design reinforcement calculations and checked against minimum reinforcement requirements. The detailed calculations are provided below.

Reinforcement Design Calculation

Vertical Reinforcement For water face (W/F),

Design Moment = 42 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Wing Wall = 0.4 m

Modulus of Rupture = 0.52 x √ fc´= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.00533 m^4

Cracking Moment= 2.32 x 1000 x 0.00533/ (0.4/2) = 62 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 75 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 55.86 kN-m/m

Minimum reinforcement required is for 55.86 kN-m/m which is greater than 42 kN-m/m So, design moment finally considered is 55.86 kN-m/m.

R16-250 is provided for design moment of 55.86 kN-m/m. Checking against minimum surface reinforcement and design controlling moment, it is found that R16-307 is required. However, spacing of 250 mm is provided instead of 307 mm as spacing of main/horizontal reinforcement of top bar of pile cap required is 250 mm and thus spacing of 250mm is chosen for convenience of working condition.

Vertical Reinforcement For earth face (E/F),

Design Moment = 105 kN-m/m

Compressive strength of Concrete, fc´= 20 MPa

Thickness of Wing Wall = 0.4 m

Modulus of Rupture = 0.62 x √ fc´= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.00533 m^4

Cracking Moment= 2.32 x 1000 x 0.00533/ (0.4/2)= 62 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 75 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 139.65 kN-m/m

Minimum reinforcement required is for 75 kN-m/m which is less than 105 kN-m/m. So, design moment finally considered is 105 kN-m/m.

Reinforcement area required for 105 kN-m/m is 1250 mm^2. For this reason, R16-250 and R16-250 are provided. Here, curtailment has been done for one R16-150 at 2500mm distance. Extra Reinforcement is provided for 40 times bar diameter in addition to required length for moment concentration as obtained from STAAD.Pro analysis. R16-250 has been continued for the full height as requirement of minimum reinforcement from AASHTO 07.

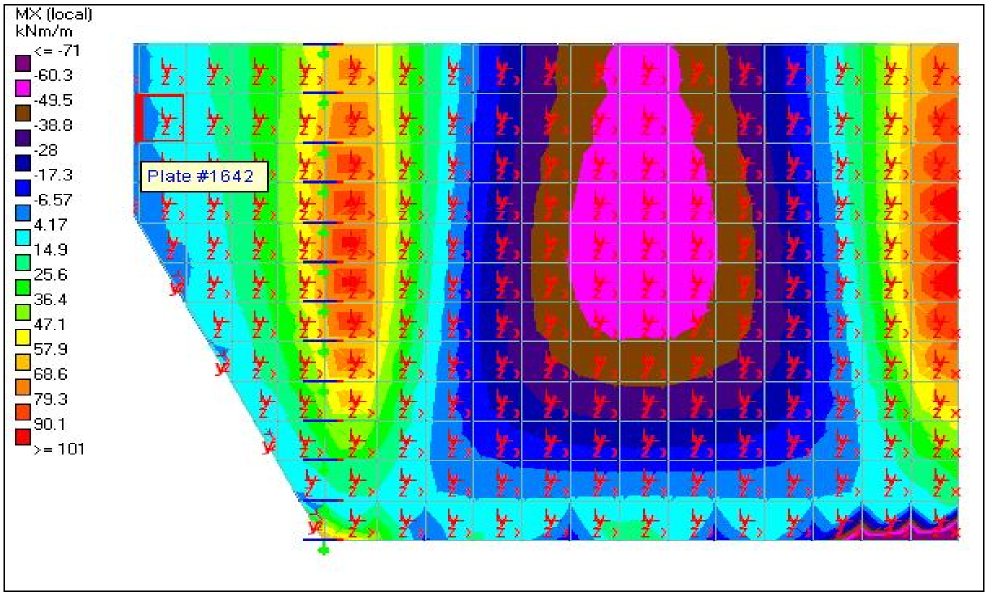


Fig 3.11 Bending Moment Contour of Wing Wall, M_x, kN-m

Table: Bending Moment, Mx along mid horizontal section for Horizontal Reinforcement Calculation

SL NO	PLATE NO	My (kN-m/m)	SL NO	PLATE NO	My (kN-m/m)
1	1926	-9.5	9	1851	+56
2	1927	-22	10	1852	+59
3	1928	-46	11	1853	+56
4	1758	-96	12	1854	+45
5	1847	-40	13	1855	+27
6	1848	-2	14	1856	-1.2
7	1849	+27	15	1857	-40
8	1850	+45	16	1858	-94

It can be observed that maximum tensile stress is observed at plate no 1754 and 1858 wherever, maximum compressive stress at plate no 1852. It is observed that tension face reinforcements are required near counterfort and support near abutment wall. Thus, extra reinforcements are provided near these two places and for the whole length minimum reinforcement is provided. Maximum compression face reinforcement is required for 59 kN-m/m.

R12-200 is provided as extra reinforcement near counterfort and abutment wall support. The detailed calculations are provided below.

Reinforcement Design Calculation

Horizontal Reinforcement For water face (W/F),

Design Moment = 59 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Wing Wall = 0.4 m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.00533 m4

Cracking Moment= 2.32 x 1000 x 0.00533/ (0.4/2) = 68 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 75 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 78.47 kN-m/m

Minimum reinforcement required is for 75 kN-m/m which is greater than 59kN-m/m. So, design moment finally considered is 75 kN-m/m. R16-200 is provided for design moment of 78 kN-m/m.

Horizontal Reinforcement For earth face (E/F),

Design Moment = 105 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Wing Wall = 0.4 m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa
Moment of Inertia = b x h^3/12 = 0.00533m4

Cracking Moment = 2.32 x 1000 x 0.00533/ (0.4/2)= 68 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 75 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 139.65 kN-m/m

Minimum reinforcement required is for 75 kN-m/m which is less than 105 kN-m/m. So, design moment finally considered is 105 kN-m/m.

Reinforcement area required for 105 kN-m/m is $1250mm^2$. For this reason, R16-200 and R12-200 are provided near counterfort for length of 2500 mm. Here, 40 times bar diameter of length is considered in addition to the required length obtained from STAAD.PRO model. At the adjoining part of abutment wall and wing wall extra reinforcement of R12-200 is provided with the continuing reinforcement of T16-200 as, center stress results from STAAD.PRO shows lesser value of Mx for other plates in horizontal direction. 40 times bar diameter equivalent to 500 mm is considered in calculating curtailment length in addition to required length obtained from STAAD.Pro analysis. So, R16-200 is continued for the total length of wing wall in horizontal direction.

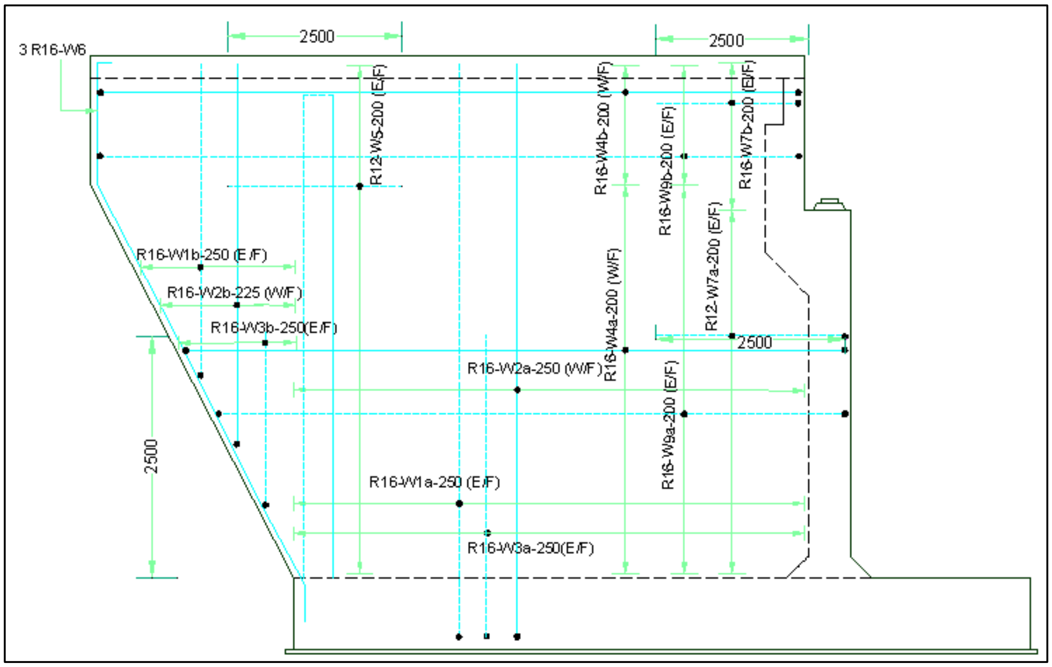


Fig 3.12 Reinforcement Details of Wing Wall

3.4 Pile Cap Design

Pile cap is designed considering plate properties with thickness varying from 700mm to 1000mm Plate and node no's of pile cap are provided with the following figures. Moment contour map of plate according to plate orientations are also attached here.

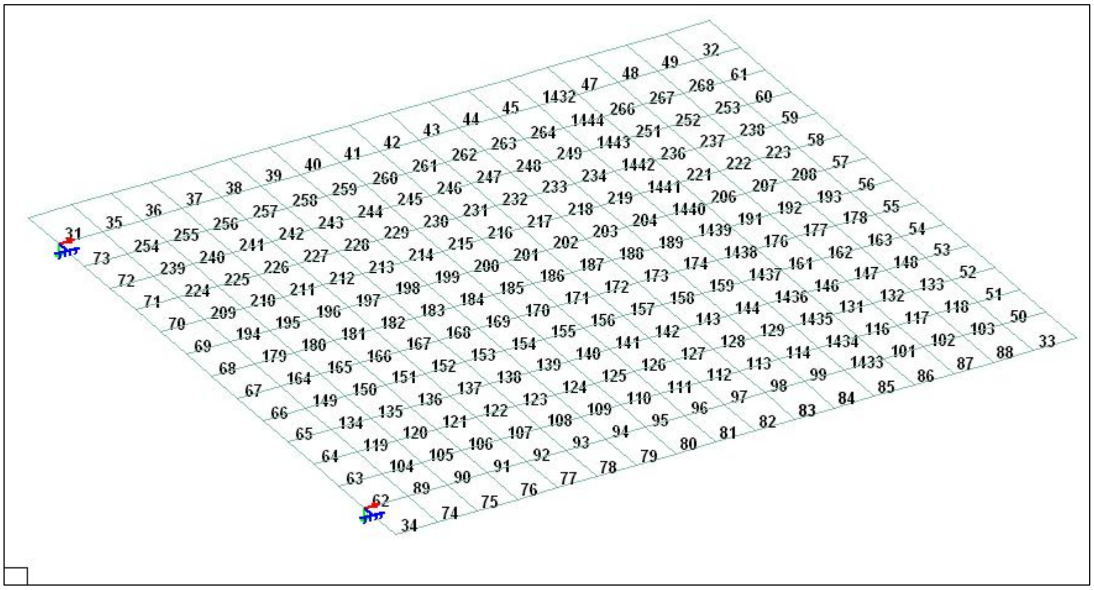


Fig 3.13 Plate Numbers of Pile Cap from STAAD.Pro Model

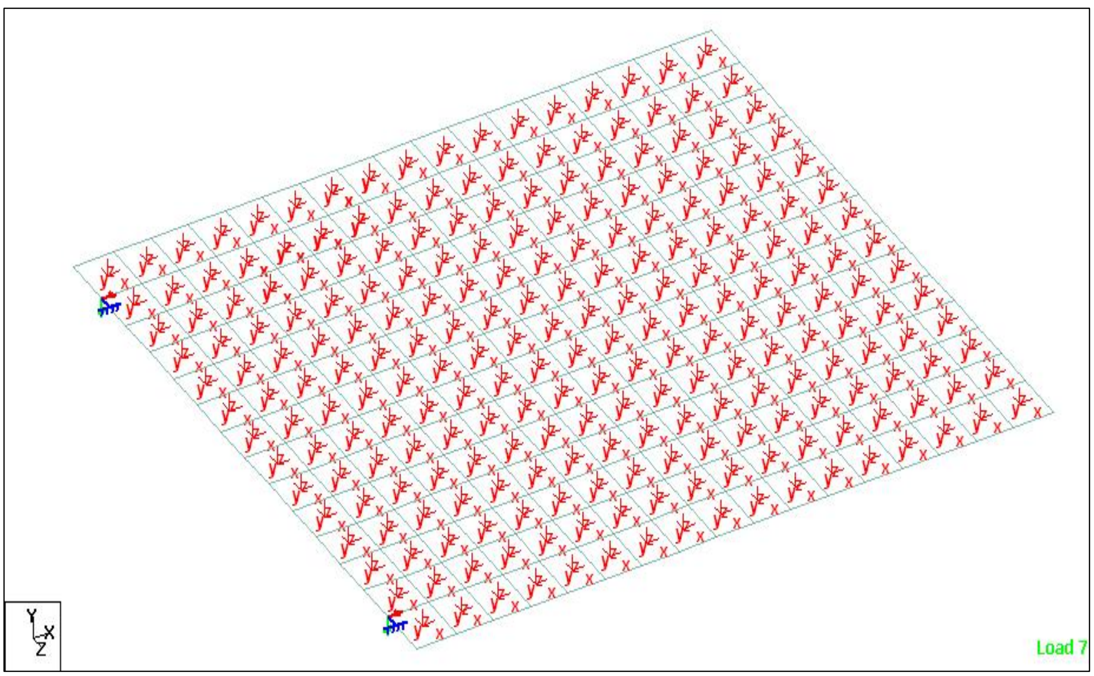


Fig 3.14 Axis Orientation of Pile Cap from STAAD.Pro Model

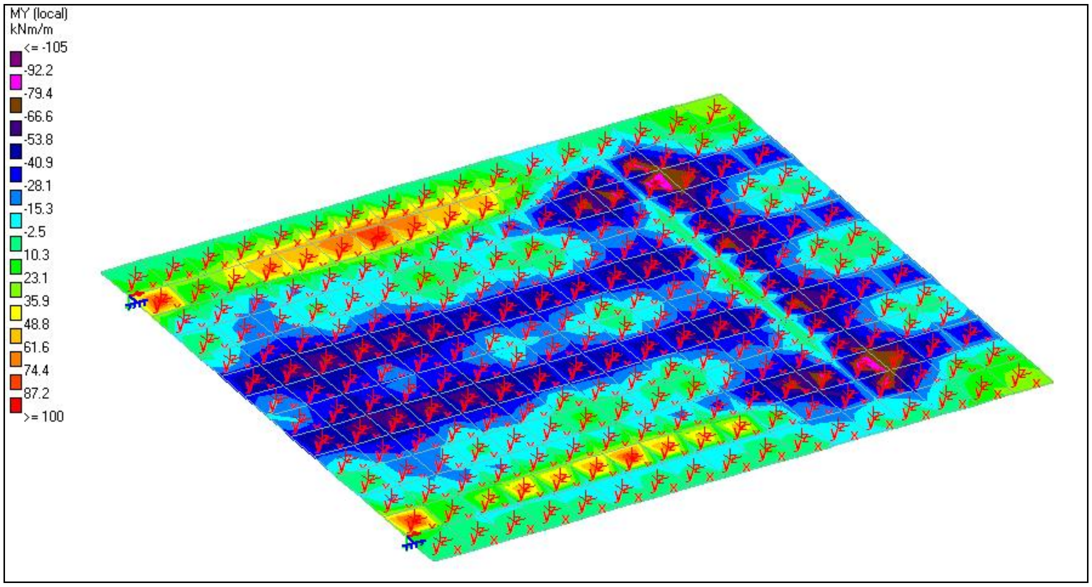


Fig 3.15 Bending Moment Contour along Y axis, M_y of Pile Cap

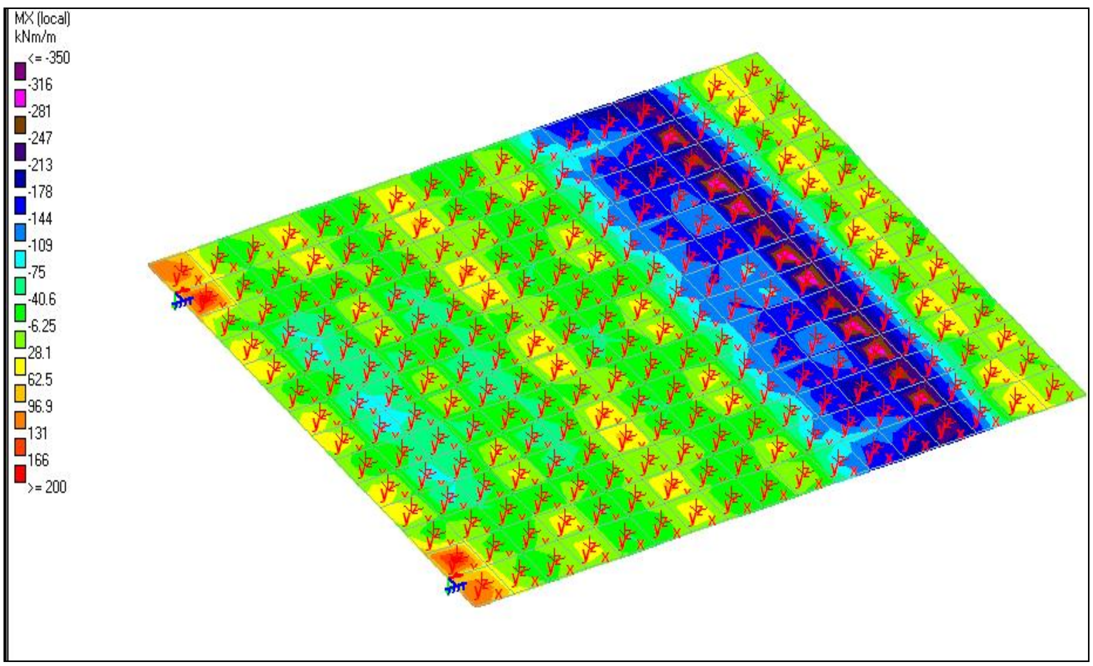


Fig 3.16 Bending Moment Contour along X axis, M_x of Pile Cap

Reinforcement Design along Traffic Direction

Bottom Face

Bending Moment Calculation considering abutment wall as fixed support for abutment height of 8.0m is done to check moment along x axis obtained from STAAD.Pro. Toe side moment for direction along traffic is calculated as follows:

M_{LL}= (1.5x680)x1.85/1.55 + (1.5x630)x 0.05/1.55

= 1250 kN-m/m

M_{DL} = 0.9x24x1.2x2.5^2/2

= 81 kN-m/m

Design Moment = 1250-81=1170 kN-m/m

Maximum bending moment M_x from STAAD.PRO is 315kN-m/m as observed from Moment contour, which is less than moment obtained from manual calculation. As, tension face reinforcement is critical, thus, this pile cap is designed for toe side bottom face reinforcement along traffic direction with design moment of 1170 kN-m/m.

Reinforcement Design

Design Moment = 1170 kN-m/m

Compressive strength of Concrete, fc′= 20 MPa

Thickness of Pile Cap= 1.2m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa

Moment of Inertia = b x h^3/12 = 1000 x 1.2^3 /12 =0.144m^4

Cracking Moment= 2.32 x 1000 x 0.144/ (1.2/2)= 558 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 670 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 1556 kN-m/m

So, minimum reinforcement required is for 798.55 kN-m/m which is less than design moment 1170 kN-m/m. So, design moment finally considered is 1170 kN-m/m.

R16-150 and R25-150 is provided for design moment of 1170 kN-m/m as bottom reinforcement for the cantilever section which has been curtailed at the distance of 4025 mm from toe side. 25 x 40+300(distance from c/l of abutment wall) = 1300mm. Distance of pile cap edge and abutment wall c/l is 2800 mm. So, (2800-75) = 2725mm of reinforcement is provided at the right side. So, extra reinforcement of R25-150 is curtailed at the distance of (1300+2725) = 4025 mm from the toe side.

Top Face

Maximum moment at the compression face is 200 kN-m/m which requires R16 bar at 250 mm spacing. Again, vertical reinforcement of wing wall is R16-250 and thus R16-250 is provided as main reinforcement at top face of pile cap.

Reinforcement Design perpendicular to Traffic Direction

Bottom face

Flexural moment of 105kN-m/m is found at the bottom face observed from STAAD.PRO moment contour. Thus, according to previous calculation R16-250 is provided as bottom face y direction reinforcement.

Top face

Flexural moment of 100 kN-m/m is found at the top face observed from STAAD.PRO moment contour. Checking against vertical reinforcement of abutment wall thus, R16-250 is provided as top face y direction reinforcement.

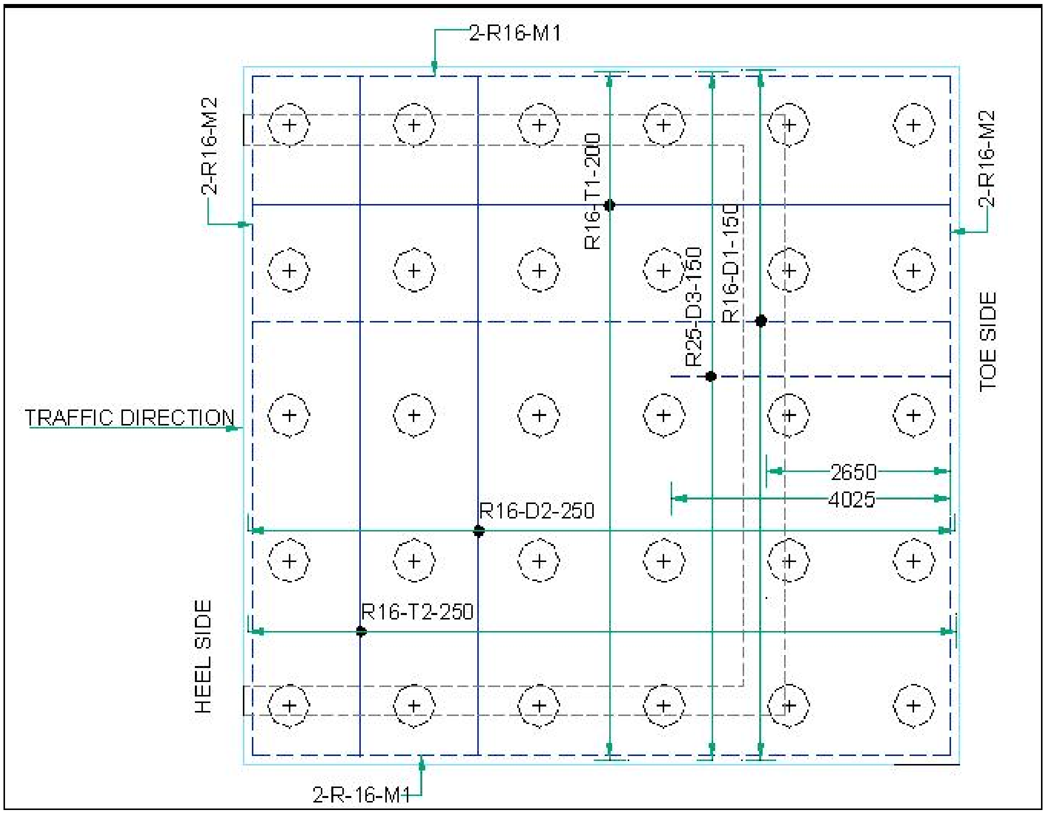


Fig 3.17 Pile Cap Reinforcement Details

3.5 Structural Design of Pile

Material Properties

Concrete Cylinder Strength at 28 Days, $f'c= 25$ MPa

Yield Strength of reinforcing steel, $fy= 400$ MPa

General Data

Unit Weight of Soil, $\gamma = 18\ kN/m^3$

Maximum Horizontal Load in Pile (factored), $P_x= 70.1$ kN

Maximum Axial Load in Pile (factored), $P_y = 713$ kN

Moment Applied at the Pile Head (factored), M_T= 0 kN-m
Coefficient. of Subgrade Modulus, n_h = 700 kN/m^3
Pile Diameter D = 0.6 m
Modulus of Elasticity, E = 24000000 kN-m^2

Soil Data

Soil	**n_h (kN/m^3)**
Medium Dense Soil	2500
Stiff Clay	1500
Soft Clay	700
Very Soft Clay	350
Organic Silt	150

NB: For loose submerged cohesion less soil n_h =1400 kN/m^3
Allowable Compressive Stress in pile = 6.25 N/mm2 (W. C. Teng, 1962)
(Considering Structural Strength of Pile only)

Proposed Reinforcement:

No. of Bar	20
Bar Diameter	25mm
Clear Cover	75mm

Analysis:
Axial Stress on Pile, σx= 2.53 N/mm2
Allowable stress of Pile, σallowable= 6.25 N/mm2

Here, the allowable compressive stress is greater than applied stress. So the lateral restrain required to prevent pile buckling will be very small. There is no need to check the buckling capacity.

No of Longitudinal bar, n = 16
Total Steel Provided, As= n x Area of Rebar= 5024 mm^2
Gross Area of Concrete, Ag= 3.14 x (D^2)/4= 282600 mm^2
% Steel provided= As/Ag= 1.77
For the Circular pile:
Moment of Inertia, I = 3.14 x (D^4)/64 = 3.14x (0.6^4)/64 = 0.006 m^4
Stiffness Factor, T= $(EI/\acute{\eta}_h)^{0.2}$= 2.936m
Minimum Length of pile, L = 4xT= 12m
Level of Fixity, L'= 1.8xT = 5.29m

From Reese and Matlock (6.13) Graph:
Bending Moment Coefficient. Am= 0.8
Bending Moment Coefficient. Bm= 1.0
Bending moment, M= A_m x P_xx T + B_m x M_t= 164.6 kN-m

Input for PCA COL:

Level of fixity, L'= 5.29m
Axial Load, P= 713 kN (Factored Load)
Moment, M= 164.63 kN-m (Factored Moment)

Notes: - Supporting strength of soil is not considered in evaluating the Capacity of Single Pile

3.6 Geotechnical Design of Pile

AASHTO 4.6.5.1
Geotechnical Design

Project : RSM JICA

Date : 16/11/2008

Bore Hole No.	1	Bore Hole Level	1.500 m	Unit Weight of Soil	18.00 kN/m^3
Pile Size	0.60 m	Ground Water Level	4.000 m	Unit Weight of Concrete	24.00 kN/m^3
Pile Spacing	1.8 m	Scour Level/Liqueafaction zone	1.000 m		
		Pilecap Bottom Level	1.000 m		
Factor of Safety against :		Interval of SPT	1.50 m		
Side Resistance	2.5				
End Bearing	2.5				

No. of Piles in X direction	3 nos.
No. of Piles in Y direction	5 nos.
Total no. of Piles	15 nos.
Overall Width, X (m)	4.2 m
Overall Length ,Y(m)	7.8 m
Perimeter of pile group	24 m
Area of pile group	32.76 m^2
Reduction factor for group action	0.67

Depth below Liquefac. Level	SPT N	Soil Code #	Soil Density / Consistency	Cor. SPT (N'_{70})	Lab s_{ui} (MPa)	s_{ui} for Analysis (MPa)	Adhesion Factor α	Effective Vertical Stress $\sigma'_{vi}=\gamma'_i Z_i$	Bearing Capacity Factor Nc(ind.)	Load Transfer Fractor βi	Ultimate unit side Friction (MPa)	Ultimate Friction at Depth (kN)	Ultimate End Bearing (kN)	Allowable Pile Capacity (kN)	Bearing Capacity Factor Nc(Gr.)	Ult. Group Side Friction (kN)	Ult. Group End Bearing (kN)	Ultimate Group Capacity (kN)	Allowable Group Capacity (kN)	Depth below Pile cap (m)
								0.00												
1.5	1	0	Very Soft Silt/Clay	1		0.01	0.40	8.000	9.00		0.003	7	17	6	5.80	75.06	169	243.9	86	1.0
3.0	1	0	Very Soft Silt/Clay	1		0.01	0.40	20.000	9.00		0.003	15	17	3	6.20	150.12	169	319.0	116	2.5
4.5	3	0	Very Soft Silt/Clay	3		0.02	0.40	32.000	9.00		0.008	37	50	19	6.59	375.30	507	882.0	319	4.0
6.0	12	1	Medium Dense Sand	12		0.00	0.00	44.000		0.94	0.041	154	193	**117**	NA	1488.81	1944	3432.4	**1243**	5.5
7.5	12	1	Medium Dense Sand	12		0.00	0.00	56.000		0.86	0.048	291	193	**- 166**	NA	1743.67	1944	3687.3	**1345**	7.0
9.0	20	1	Medium Dense Sand	18		0.00	0.00	68.000		0.80	0.054	445	322	**273**	NA	1953.23	3239	5192.6	**1861**	8.5
10.5	21	1	Medium Dense Sand	18		0.00	0.00	80.000		0.74	0.059	611	338	**340**	NA	2121.76	3401	5523.1	**1982**	10.0
12.0	27	1	Medium Dense Sand	21		0.00	0.00	92.000		0.68	0.063	788	435	**444**	NA	2252.52	4373	6625.7	**2359**	11.5
13.5	50	1	Dense Sand	33		0.00	0.00	104.000		0.63	0.065	973	806	**660**	NA	2348.09	8098	10446.6	**3639**	13.0
15.0	50	1	Dense Sand	33		0.00	0.00	116.000		0.58	0.067	1162	806	**730**	NA	2410.57	8098	10509.0	**3664**	14.5
16.5	50	1	Dense Sand	33		0.00	0.00	128.000		0.53	0.068	1354	806	**800**	NA	2441.75	8098	10540.2	**3676**	16.0
18.0	50	1	Dense Sand	33		0.00	0.00	140.000		0.48	0.068	1546	806	**871**	NA	2443.13	8098	10541.6	**3677**	17.5
19.5	50	1	Dense Sand	33		0.00	0.00	152.000		0.44	0.067	1735	806	**941**	NA	2416.03	8098	10514.5	**3666**	19.0
21.0	50	1	Dense Sand	33		0.00	0.00	164.000		0.40	0.066	1921	806	**1010**	NA	2361.61	8098	10460.1	**3644**	20.5
22.5	50	1	Dense Sand	33		0.00	0.00	176.000		0.36	0.063	2100	806	**1075**	NA	2280.89	8098	10379.4	**3612**	22.0
24.0	50	1	Dense Sand	33		0.00	0.00	188.000		0.32	0.060	2271	806	**1138**	NA	2174.78	8098	10273.3	**3569**	23.5
25.5	50	1	Dense Sand	33		0.00	0.00	200.000		0.28	0.057	2431	806	**1196**	NA	2044.12	8098	10142.6	**3517**	25.0
27.0	50	1	Dense Sand	33		0.00	0.00	212.000		0.25	0.053	2581	806	**1250**	NA	1908.00	8098	10006.5	**3463**	26.5
28.5	50	1	Dense Sand	33		0.00	0.00	224.000		0.25	0.056	2740	806	**1307**	NA	2016.00	8098	10114.5	**3506**	28.0
30.0	50	1	Dense Sand	33		0.00	0.00	236.000		0.25	0.059	2906	806	**1368**	NA	2124.00	8098	10222.5	**3549**	29.5

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	Part		
Job Title Double Lane Bridge Manual	Ref		
	By	Date 16-11-08	Chd
Client LGED-JICA	File ABUT 8.0 m COUNTER.s	Date/Time	16-Nov-2008 12:59

Job Information

	Engineer	Checked	Approved
Name:			
Date:	16-11-08		

Structure Type	SPACE FRAME

Number of Nodes	848	Highest Node	878
Number of Elements	30	Highest Beam	30
Number of Plates	774	Highest Plate	2090

Number of Basic Load Cases	7
Number of Combination Load Cases	2

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DEAD LOAD
Primary	2	SUPERSTRUCTURE DL
Primary	5	V E P ON PILE CAP
Primary	3	H E P ON WINGWALL 1
Primary	4	H E P ON WINGWALL 2
Primary	8	H E P ON ABUTMENTWALL
Primary	9	SUPERSTRUCTURE LL
Combination	6	COMBINATION SERVICE
Combination	7	COMBINATION FACTORED

Basic Load Cases

Number	Name
1	DEAD LOAD
2	SUPERSTRUCTURE DL
5	V E P ON PILE CAP
3	H E P ON WINGWALL 1
4	H E P ON WINGWALL 2
8	H E P ON ABUTMENTWALL
9	SUPERSTRUCTURE LL

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Software licensed to DPM Consultants Ltd.	Job No APPENDIX-"A'	Sheet No 2	Rev
	Part		
Job Title Double Lane Bridge Manual	Ref		
	By	Date 16-11-08	Chd
Client LGED-JICA	File ABUT 8.0 m COUNTER.s	Date/Time 16-Nov-2008 12:59	

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
6	COMBINATION SERVICE	1	DEAD LOAD	1.00
		2	SUPERSTRUCTURE DL	1.00
		5	V E P ON PILE CAP	1.00
		3	H E P ON WINGWALL 1	1.00
		4	H E P ON WINGWALL 2	1.00
		8	H E P ON ABUTMENTWALL	1.00
		9	SUPERSTRUCTURE LL	1.00
7	COMBINATION FACTORED	1	DEAD LOAD	1.25
		2	SUPERSTRUCTURE DL	1.25
		5	V E P ON PILE CAP	1.35
		3	H E P ON WINGWALL 1	1.50
		4	H E P ON WINGWALL 2	1.50
		8	H E P ON ABUTMENTWALL	1.50
		9	SUPERSTRUCTURE LL	1.75

Node Displacement Summary

	Node	L/C	X (mm)	Y (mm)	Z (mm)	Resultant (mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	873	7:COMBINATI(	**12.404**	-1.342	0.002	12.477	-0.000	0.000	-0.000
Min X	1	3:H E P ON WI	**-2.191**	-0.012	0.032	2.191	-0.000	0.000	-0.000
Max Y	309	8:H E P ON AE	7.715	**0.330**	-0.000	7.722	-0.000	-0.000	0.000
Min Y	841	7:COMBINATI(	12.171	**-1.375**	0.007	12.248	0.000	-0.000	-0.000
Max Z	11	4:H E P ON WI	0.898	0.018	**4.656**	4.742	0.000	-0.000	-0.000
Min Z	13	3:H E P ON WI	0.898	0.018	**-4.656**	4.742	-0.000	0.000	-0.000
Max rX	584	7:COMBINATI(	11.165	-0.785	0.617	11.210	**0.001**	0.000	-0.000
Min rX	347	7:COMBINATI(	11.165	-0.785	-0.617	11.210	**-0.001**	-0.000	-0.000
Max rY	416	3:H E P ON WI	-1.740	-0.037	-1.762	2.477	0.000	**0.001**	-0.000
Min rY	653	4:H E P ON WI	-1.740	-0.037	1.762	2.477	-0.000	**-0.001**	-0.000
Max rZ	16	7:COMBINATI(	11.051	-0.636	0.006	11.070	-0.000	-0.000	**0.000**
Min rZ	774	8:H E P ON AE	7.942	-0.068	-0.000	7.943	-0.000	0.000	**-0.000**
Max Rst	873	7:COMBINATI(	12.404	-1.342	0.002	**12.477**	-0.000	0.000	-0.000

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Part	
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Client LGED-JICA	File ABUT 8.0 m COUNTER.s	Date/Time 16-Nov-2008 12:59

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (N/mm^2)	Qy (N/mm^2)	Sx (N/mm^2)	Sy (N/mm^2)	Sxy (N/mm^2)	Mx (kNm/m)	My (kNm/m)	Mxy (kNm/m)
Max Qx	1751	4:H E P ON WI	**0.945**	-0.338	0.012	0.076	0.031	-74.125	-6.056	-5.728
Min Qx	1456	3:H E P ON WI	**-0.945**	0.338	0.012	0.076	0.031	74.125	6.056	5.728
Max Qy	1768	7:COMBINATI(	-0.002	**0.376**	0.011	-0.384	0.301	-14.955	-105.029	-0.430
Min Qy	1491	7:COMBINATI(	0.002	**-0.376**	0.011	-0.383	0.302	14.958	105.033	0.428
Max Sx	1946	7:COMBINATI(	-0.000	0.023	**0.386**	0.004	0.011	-0.136	-0.800	-0.198
Min Sx	1498	7:COMBINATI(	-0.052	-0.051	**-0.387**	-1.591	0.596	8.088	21.994	-5.944
Max Sy	2065	7:COMBINATI(	0.110	0.004	0.228	**0.220**	0.448	38.507	4.026	1.991
Min Sy	1947	7:COMBINATI(	0.060	-0.214	-0.295	**-1.810**	-0.362	20.430	92.219	-7.258
Max Sxy	1774	7:COMBINATI(	0.052	0.051	-0.387	-1.591	**0.599**	-8.082	-21.954	5.946
Min Sxy	2047	7:COMBINATI(	-0.036	0.019	0.040	-1.332	**-0.528**	-43.570	-21.291	1.102
Max Mx	62	7:COMBINATI(	-0.025	-0.240	-0.044	-0.070	-0.041	**186.815**	91.471	64.312
Min Mx	191	7:COMBINATI(	0.245	-0.140	-0.050	0.097	0.023	**-336.994**	-54.191	-32.271
Max My	1951	7:COMBINATI(	0.022	-0.286	-0.008	-0.708	-0.066	24.307	**146.042**	4.504
Min My	1768	7:COMBINATI(	-0.002	0.376	0.011	-0.384	0.301	-14.955	**-105.029**	-0.430
Max Mxy	62	7:COMBINATI(	-0.025	-0.240	-0.044	-0.070	-0.041	186.815	91.471	**64.312**
Min Mxy	73	7:COMBINATI(	-0.024	0.239	-0.044	-0.070	0.041	186.637	91.352	**-64.242**

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CHAPTER 4
BOX CULVERT

4.0 Structural Design of Box culvert

4.1 Model Details

4.1.1 General

STAAD.Pro 2006 has been used as a data processing tool for design purpose. Sectional view of 1 vent box culvert of box size 4.0m x 4.0m structural model is presented here along with load case details and node displacement summary reports. Node displacement summary from the STAAD.Pro analysis is attached with the rest of the report. AASHTO 07 codes have been followed for detailing of reinforcement for all the elements of box culvert.

Specific type of foundation is not required for box culvert type structure. It is placed over one layer of plain concrete under lying one layer of sand cushion. 1.0 m width of strip is analyzed which is supported by vertical spring with value of 400 kN/m. Spring support is used to represent properties of soil where allowable bearing capacity is 80kN/m^2 considering normally consolidated clay/lower bound medium dense sand. Springs are provided at every 0.36 m.

4.1.2 Calculations for Equivalent Strip Width and Loading

Equivalent strip width has been calculated using the following formula from AASHTO 07 Table 4.6.2.1.3-1

660+ 0.55 x S = 2860 mm = 2.86 m

Where, S = Spacing of supporting components = 4000 mm

Considering HL-97 live loading, only rear axle load of truck of 145kN is placed as truck load.

Truck load	= 145/2.86 = 50.7 kN	
Lane Load	= 9.3/3 = 3.1 kN/m	[Lane load = 9.3 kN/m Ref: AASHTO 07 Art No3.6.1.2.4]
Earth Pressure on vertical walls	= 0.4 x 18 x 4 = 28.8 kN/m^2	[Co-efficient is chosen for cases in between active and at rest earth pressure]
Surcharge Pressure on vertical walls	= 0.4 x 18 x 0.61 = 4.39 kN/m^2	[Co-efficient is chosen for cases in between active and at rest earth pressure]

Load for Wearing course = 50/1000 x 23 = 1.15 kN/m^2 [Unit of weight of Wearing Course =23 kN/m^3

Self weight and wearing course are applied as different load cases for this analysis.

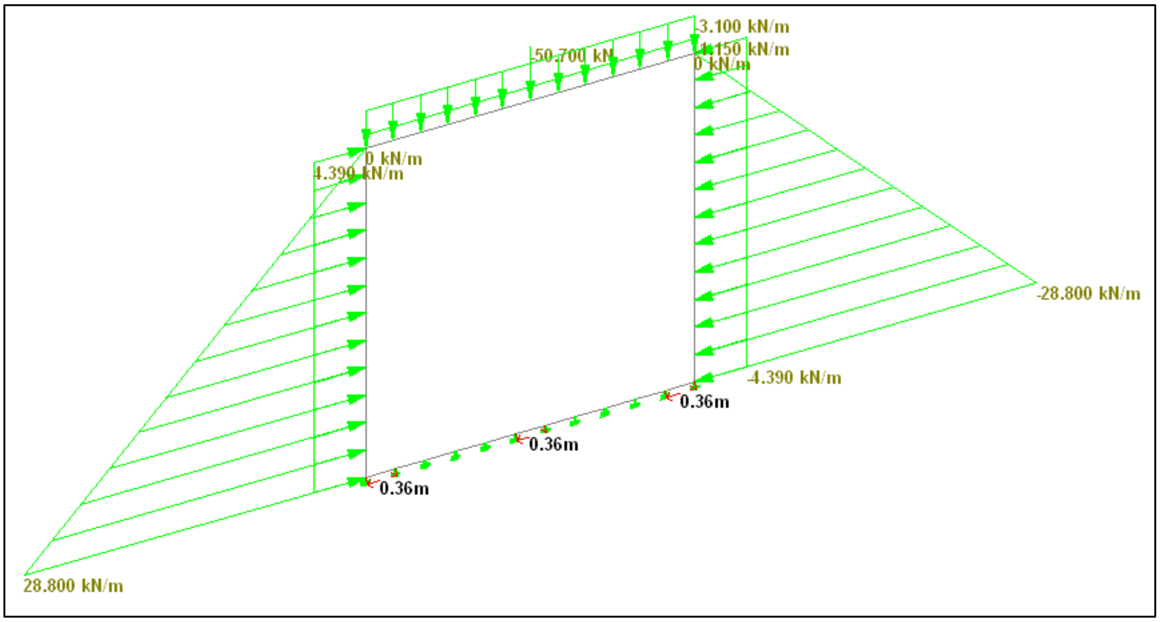


FIG 4.1 Unfactored combination of loading with Spring Support at 0.36m distance

4.1.3 Section Properties

Thickness of Vertical Wall	=	0.35 m
Thickness of Bottom Slab	=	0.30 m
Thickness of Top Slab	=	0.30 m

4.2 Flexural Moment

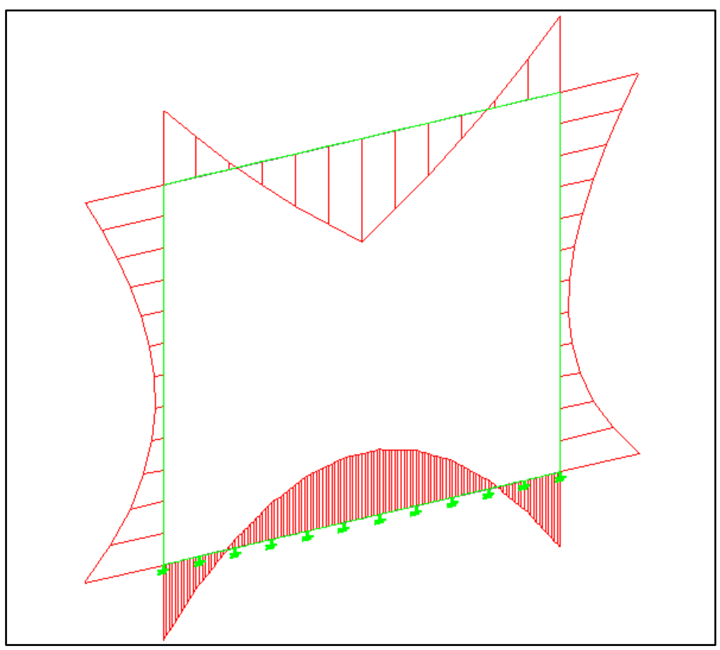


FIG 4.2 Flexural Moment Diagram of 1 vent Box Culvert

4.3 Calculation of Reinforcement

a) Top Slab

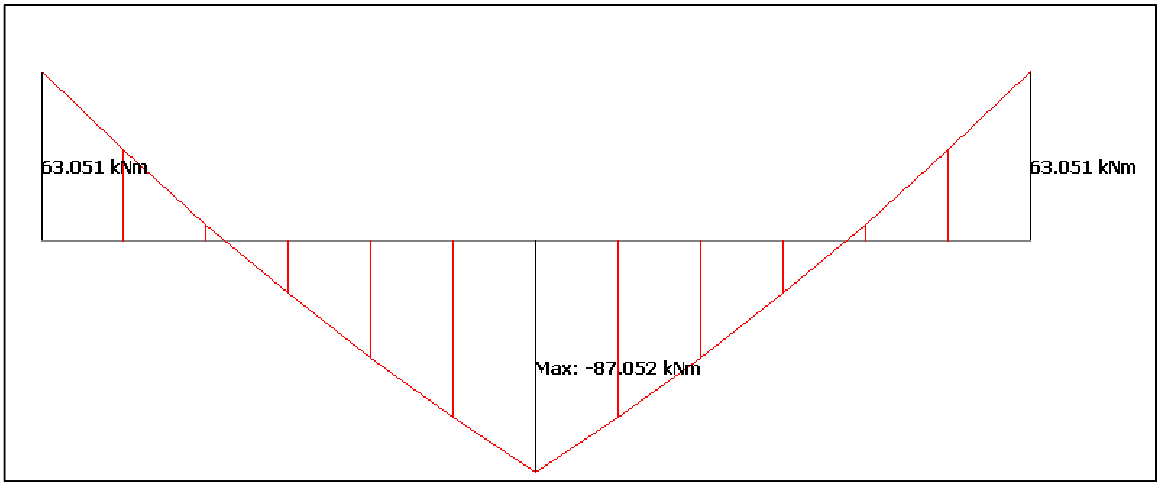


FIG 4.3 Flexural Moment(factored) Diagram of Top Slab

Reinforcement Design

Considering 1 m strip in

Design Moment = 87 kN-m/m (In top slab & in Y- direction)

Compressive strength of Concrete, fc′= 20 MPa

Thickness top slab = 0.3 m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa

Moment of Inertia = b x h^3/12 = 1 x 0.3^3 /12 =0.00225 m^4

Cracking Moment= 2.32 x 1000 x 0.00225/ (0.3/2)= 35 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 42 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 116 kN-m/m

So, minimum reinforcement required is for 42 kN-m/m which is less than design moment 87 kN-m/m. So, design moment finally considered is 87 kN-m/m.

R16-125 is provided at the bottom of top slab for design moment of 87 kN-m/m. For top face of top slab maximum flexural moment is found for both corners. Minimum reinforcement of R16-250 is continued whereas extra reinforcement of R12-150 is provided for protecting the corners. Extra reinforcement is provided for one fourth of length in vertical and horizontal direction from outer edge of fillet.

b) Bottom Slab

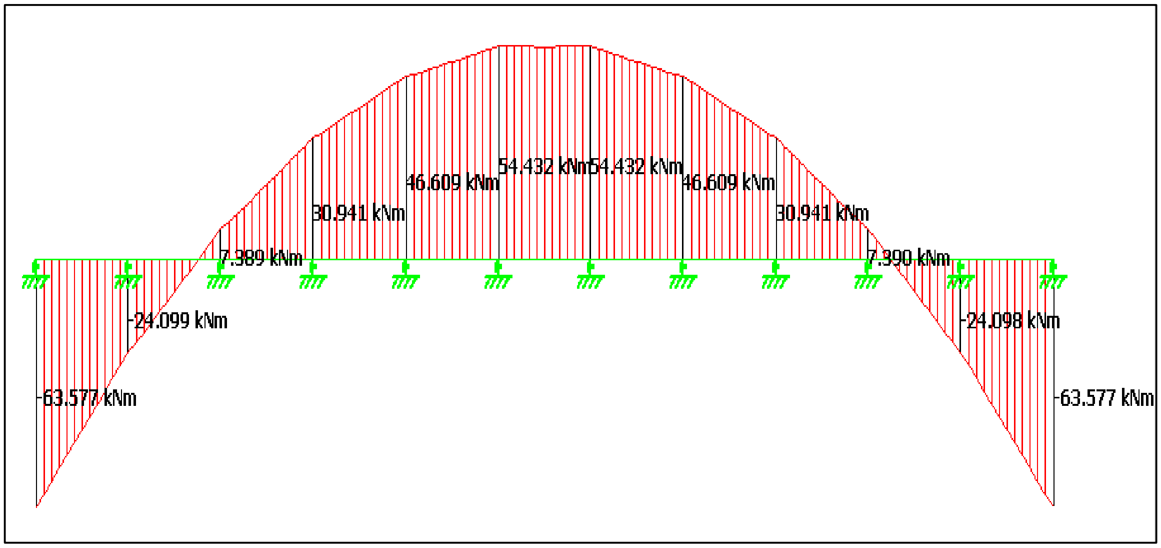


FIG 4.3 Flexural Moment (factored) Diagram of Bottom Slab

Reinforcement Design

Considering 1 m strip in

Design Moment = 55 kN-m/m (Top face of Bottom Slab)

Compressive strength of Concrete, fc´= 20 MPa

Thickness top slab = 0.3 m

Modulus of Rupture = 0.52 x √ fc´= 2.32 MPa

Moment of Inertia = b x h^3/12 = 1 x 0.3^3 /12 =0.00225 m^4

Cracking Moment= 2.77 x 1000 x 0.00225/ (0.3/2) = 35 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 42 kN-m/m

Criteria 2, 1.33 x Flexural Moment = 73 kN-m/m

So, minimum reinforcement required is for 42 kN-m/m which is less than design moment 55 kN-m/m. So, design moment finally considered is 55 kN-m/m.

R16-250 and R10-250 are provided at the top of bottom slab for design moment of 70 kN-m/m. As observed from moment diagram above, at 560mm distance from left support only minimum reinforcement is required and thus, R10-250 is continued for required length as observed from diagram in addition to another 40 times bar diameter.

c) Vertical Wall

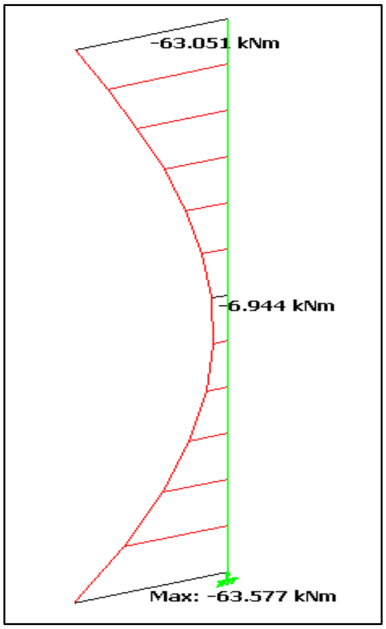


FIG 4.4 Flexural Moment Diagram (factored) of Vertical Wall

Reinforcement Design

Considering 1 m strip in

Design Moment = 64 kN-m/m (Vertical Wall)

Compressive strength of Concrete, fc′= 20 MPa

Thickness top slab = 0.3 m

Modulus of Rupture = 0.52 x √ fc′= 2.32 MPa

Moment of Inertia = b x h^3/12 = 1 x 0.3^3 /12 =0.0036 m^4

Cracking Moment= 2.32 x 1000 x 0.0036/ (0.35/2) = 48 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 57 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 85 kN-m/m

So, minimum reinforcement required is for 57 kN-m/m which is less than design moment 64 kN-m/m. So, design moment finally considered is 64 kN-m/m.

R16-250 is provided at inner and outer faces of vertical wall for design moment as minimum reinforcement. In the corner R12-250 is provided as extra reinforcement along with R16-250. R12-150 is provided for protecting the corners. Extra reinforcement is provided for one fourth of length in vertical and horizontal direction from outer edge of fillet.

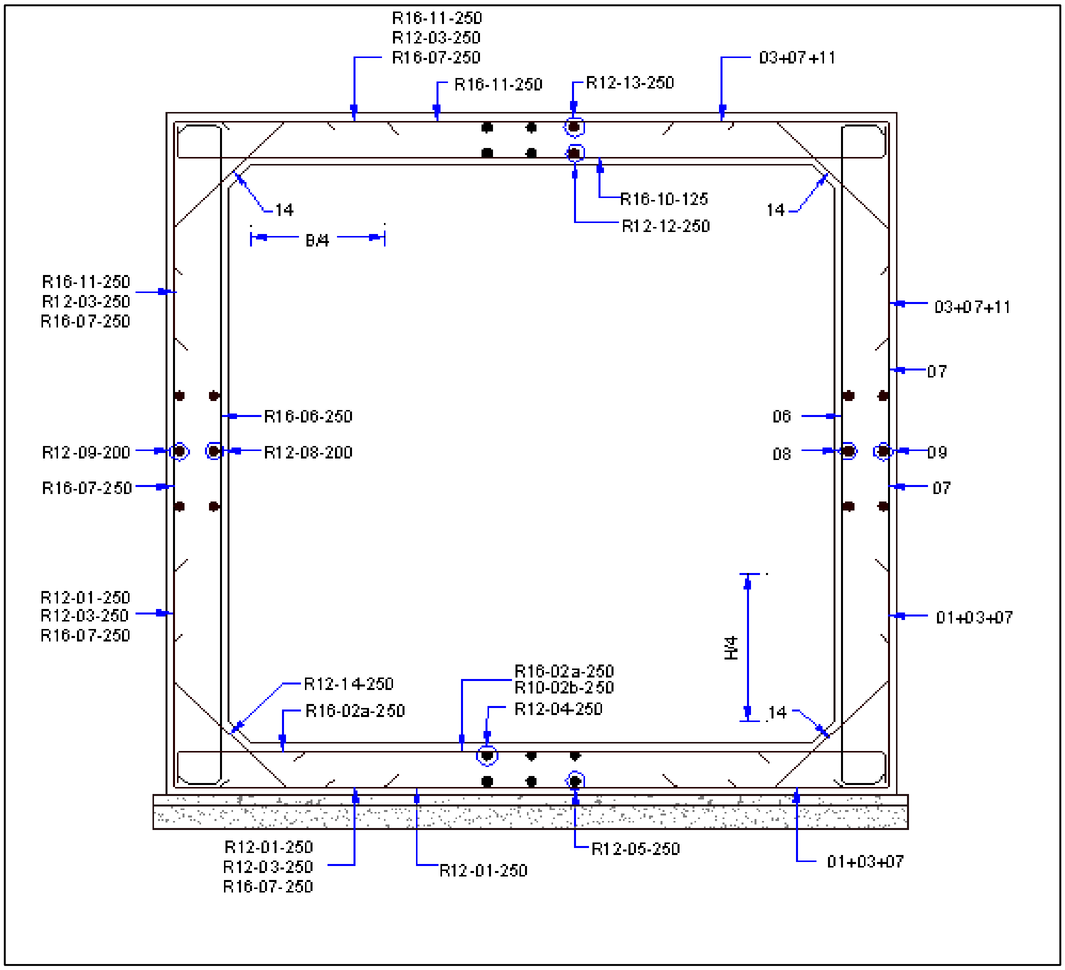


FIG 4.5 Reinforcement Details of One Vent Box Culvert

CHAPTER 5

SLOPE PROTECTION WORK

5.0 DESIGN OF PROTECTIVE WORKS BY LAUNCHING APRON

5.1 General

The calculation of slope protection work of Dharala River at Kurigram is provided here as the typical design example of protective work. The protection work is done on the right bank of the river and extended from 3.04 Km to 3.64 Km.

5.2 Design Calculation

Design Data:

Discharge	:	6093	m^3/sec	=>	Supplied by field
High Water Level	:	31.88	m(PWD)	=>	Supplied by field
Average Low Water Level	:	27.49	m(PWD)	=>	Supplied by field
Average Flow Velocity, V	:	2	m/sec	=>	Supplied by field (Assumed)
Revetment Material					
1) For pitching		CC Blocks			
2) For dumping	:	CC Blocks			
Wind Velocity	:	70 km./h		=19.44m/sec =>	Assumed
Wind Duration	:	1	hour	=>	Assumed
Fetch Length	:	0.35	km.	=>	Supplied by field
Slope of Bank, θ	:	26.56^0			
Specific Gravity, S_s	:	2.3			
Angale of Repose of Revetment Material,θ	:	40^0			
Ratio of water depth and Revetment size, h/D	:	5			
Existing River Bed Level	:	24.30	m(PWD)	=>	Supplied by field
River Bend Condition	:	Moderate bend		=>	Supplied by field
Multiplying factor for Scour depth F	:	1.5			
Gravitational acceleration, g	:	9.81	m/sec2		
Strength Co-eff. for revet. material. β — CC Block	:	3			
Significant wave height, H_s	:	0.7	m		
Wave period, T	:	2.8	sec		
Silt factor f	:	0.8			
Slope of Protection — Horizontal	:	2			
Vertical	:	1			

Design for Size of Revetment Material
i) Against Velocity

a) Using Neill's Method

$D = 0.034 \times V^2$
$= 0.136$ m
$= 136$ mm

b) Using JMBA Equation

$$D_n = \frac{0.7\,V^2}{2(S_s-1)\,g} \times \frac{2}{\mathrm{Log}(6h/D)^2} \times \frac{1}{[1-(\mathrm{Sinq}/\mathrm{sinf})^2]^{0.5}}$$

$0.110 \times 0.917 \times 1.39201$
0.140 m
140 mm

ii) Against Wave

From Table 4.1 of Design Manual for wind speed 19.44 m/sec with 1 hour duration and 0.25km Fetch

Wave breaking parameter, $E = 1.25\ T/H_s^{0.5} \tan\theta$
$= 2.09$

Using Pilarczyk equation,

$D = H_s \times 1 \times E^{1/2} / (S_s-1) \times b \times \cos\theta$

$= 290$mm

Accepted size of revetment material = 290 mm

= 58.70 Kg

Provide CC Blocks with Geotextile filter of size, L = 400 mm
B = 400 mm
T = 200 mm

Equivalent weight = 74 kg O.K

Thickness of Rip-Rap

a) Using English Formula

$$T = 0.06\ Q^{1/3}$$

	=	1.096	m
	=	1096	mm

b) Based on stone size according to ESCAP

T	=	1.5 D	
	=	435	mm
Average thickness of riprap	=	544	mm
Provide thickness of riprap	=	750	mm

According to T.S.N Rao & Spring average thickness of sloping apron is 1.25T

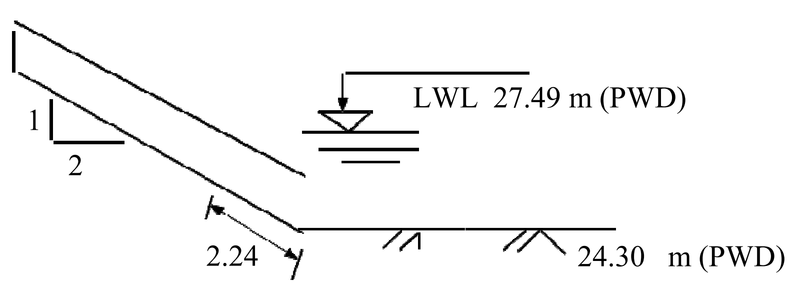
Computation of scour depth

The apron is assumed to launch at 1V:2H slope

Scour depth, R	=	$0.47(Q/f)^{1/3}$	
	=	9.25	m
Design Scour Depth	=	F x R	
	=	13.87	m
Scour Level	=	(Design Water Level)-(Design Scour Depth)	
	=	18.01	m.PWD
Existing bed level	=	24.30	m.PWD

Since existing bed level is higher than the scour level, scour level will be used for computation which is = 18.07 m (PWD)

Depth of scour from Existing bed level	=	6.29	m



Length of Protecton with 1:2 slope = 21.20 m

Use average thickness of riprap		750.00 mm
Volum of dumping blocks	=	Length of slopping Apron x Average thickness of Apron
	=	16.00 m^3/m required

Launching Apron

Since existing bed level is above the scour level, Launching apron is to be provided

Length of Launching Apron L as recommended by Inglis (1949) is 1.5 D, where d is the depth of Scour below the existing bed level

D = 6.29 m

L = 9.44 m

For adjustment of thickness with the specified blocks the dimensions are readjusted as follows :

Provide the launching apron as follows :

Thickness of launching apron at start	:	1100.00	mm
Thickness of launching apron at end	:	1500.00	mm
Length of launching apron	:	8.19	m