GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF LOCAL GOVERNMENT, RURAL DEVELOPMENT & CO-OPERATIVES
LOCAL GOVERNMENT ENGINEERING DEPARTMENT (LGED)

ROAD STRUCTURES MANUAL FOR SINGLE LANE BRIDGES

PART-E DESIGN EXAMPLES OF BRIDGES

FUNDED BY:
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PREPARED BY:
PURAKAUSHAL PROJUKTI LIMITED

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Manual on RC Girder & PC Girder Bridges

Part E- Design Examples

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CHAPTER-1

SUPERSTRUCTURE DESIGN

1.1 RC DECK OF RC GIRDER BRIDGES

The design example comprises of 5.35m wide deck with 2-girder arrangement. The span of the girders is 23.35m(c/c brg), and the overall girder length is 24.0m. The girders are spaced at 2.5m (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 = 413 MPa.

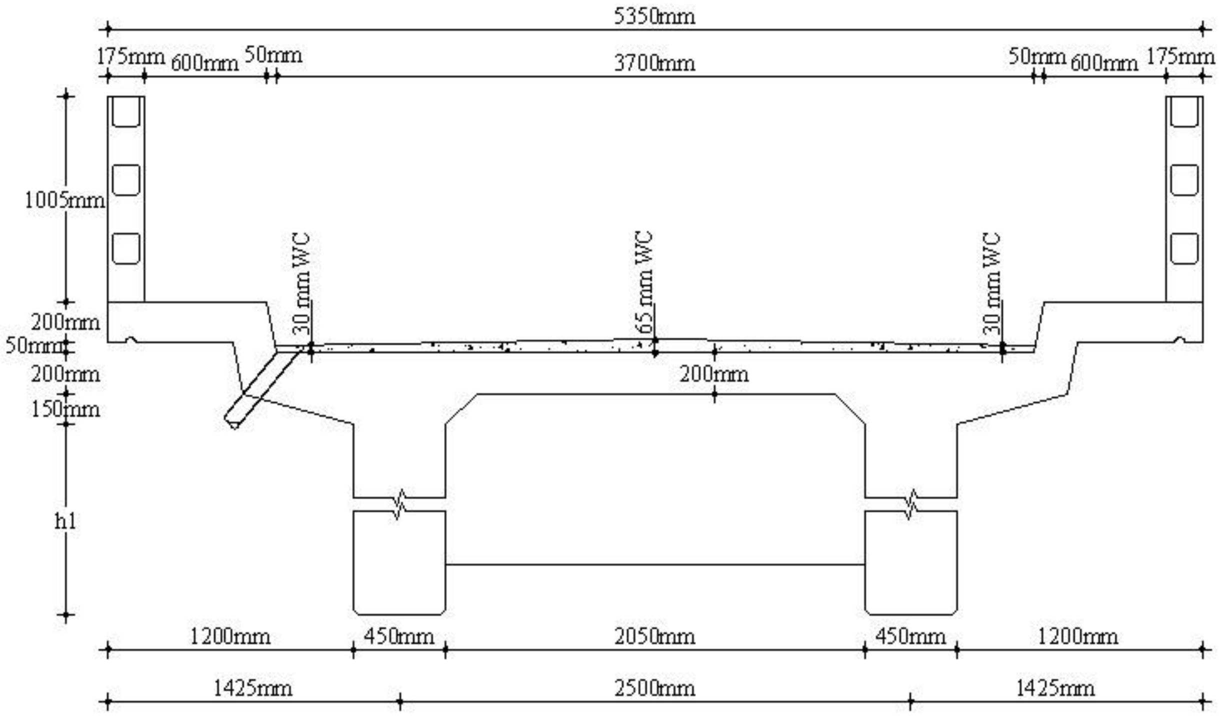


Fig 1.1.1 *Cross section of deck slab*

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*)

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	= 200mm
	Thickness of WC	= 50mm
	Side walk width	= 600mm
	Overall girder length	= 24000mm
Railing	Cross section of rail post	= 175mm x 175mm
	Height of rail post above deck	= 1005mm
	Cross section of rail bar	= 150mm x 125mm
	Rail post spacing	= 1590mm c/c
	Number of rail posts	= 16
Girder	c/c girder spacing	= 2500mm
	Height of Girder web	= 1800mm
	Girder width	= 450mm
Material Properties	Concrete strength. fc′	= 25MPa
	Yield strength of steel, fy	= 413MPa
	Unit wt. of concrete	= 24kN/m^3
	Unit wt. of wearing course	= 23kN/m^3

1.1.2 Structural analysis

1.1.2.1 Slab

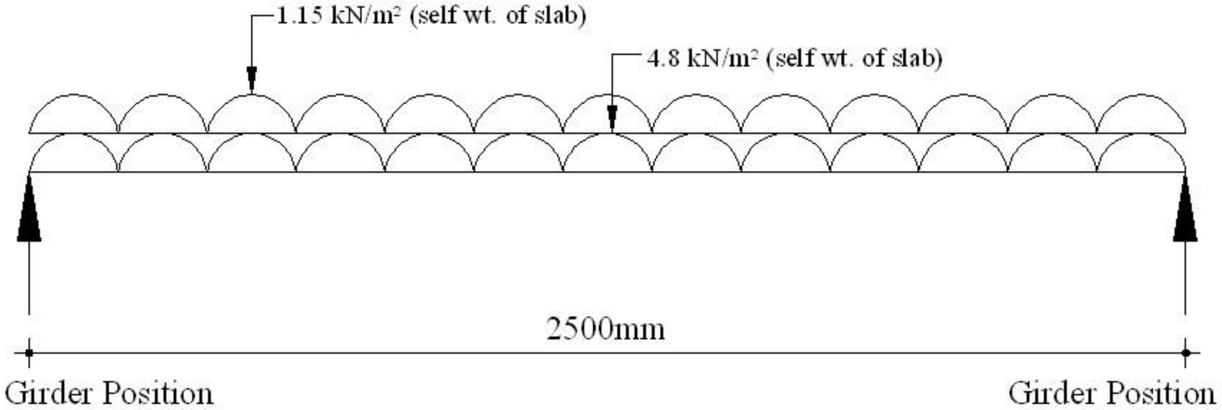


Fig 1.1.2.1 *c/c Girder Spacing Loading Diagram*

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 MDL1 (due to deck slab) = 4.8 x 2.05^2 /8 = 2.52 kN-m/m.

+ve MDL2 (due to WC) = 1.15 x 2.05^2 / 8 = 0.60 kN-m/m.

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 width. 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 x S

(*Ref. AASHTO 07, Art.4.6.2.1*)

Strip width (for -ve moment) = 1220 + 0.25 x S

(*Ref. AASHTO 07, Art.4.6.2.1*)

Here, S = clear spacing of girder = 2050mm

Distribution width (for +ve moment) = 660 + 0.55 x 2050 = 1787mm

Strip width (for -ve moment) = 1220 + 0.25 x 2050 = 1732mm

Dynamic load allowance, IM = 33% = 0.33

(*Ref. AASHTO 07, Art.4.6.2.1*)

+ve moment due to live load:

MLL = wheel (+) 17.6 kN-m/m (from STAAD)

Lane (+) 0.85 kN-m/m (from STAAD)

Total factored moment (Strength-I):

M_{pos} = (1.25 x 2.52) + (1.5 x 0.6) + 1.75 x 18.45 (1+0.33)

= 47 kN-m/m

1.1.2.2 *Deck overhang*

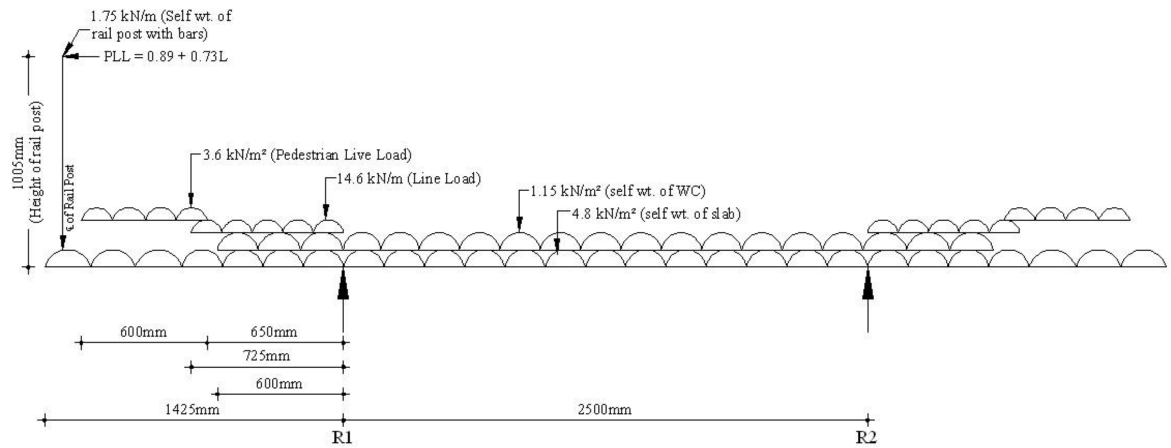


Fig 1.1.2.2 *Loading Diagram Deck Overhang*

Note: 600 mm = Side walk Width

725 mm = Distributed Overhang Line Load Width

1113 mm = Distance between CL of Rail Post and edge of Girder

1425 mm = Distance between center of Girder and edge of Sidewalk

Dead load: Self wt. of rail post with rail bar = (24 x 17 x 0.175 x 0.175 x 1.005)/24 + (3 x 0.15 x 0.125 x 21.2 x 24) / 24
= 1.75 kN/m.

-ve moment due to dead load at overhang part:

Here,

Distance from edge of girder to edge of sidewalk = 1200 mm = 1.2 m
Distance from edge of girder to CL of rail post = 1113 mm = 1.113m

Now,

-ve MDL1 = 6.7 kN-m/m (from STAAD analysis)
-ve MDL2 = 0.12 kN-m/m (from STAAD analysis)

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 = 1490 mm
   $P_{LL} = 0.89 + 0.73 \times 1.49 = 2.0$ 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 live load at overhang part:

Here,

Rail post height = 1005 mm.
Slab thickness = 200 mm.
Curb height = 250 mm.
Girder top to rail post top = 1005 + 200 + 250 = 1455 mm.
Distance of distributed line load on overhang = 725 mm
Side walk width = 600 mm
-ve MLL overhang

$$= (14.6 \times 0.725^2)/2 + 3.6 \times 0.60 \times (1.2 - 0.175)/2 + 2.0 \times 1.46$$
$$= 7.94 \text{ kN-m/m.}$$

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

M_{neg} = 1.25 x (-ve MDL1) + 1.5 x (-ve MDL2) + (-ve MLL overhang)
= 1.25 x 6.7 + 1.5 x 0.12 + 1.75 x 7.94
= 22.45 kN-m/m.

Design moment: +ve moment, M_{pos} = 47 kN-m/m.
-ve moment, M_{neg} = 22.45 kN-m/m.

1.1.3 Provision of reinforcement:

1.1.3.1 +ve reinforcement:

Here,

Slab thickness = 200 mm.

Clear cover = 50 mm.

Distance of reinforcement CG = 8 mm.

It is proposed to use R16-150 as +ve reinforcement.

Here,

Area of reinforcement = 200.96 mm^2.

A_S = 200.96 x 1000/150 = 1339.73 mm^2.

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

= (1339.73 x 413) / (0.85 x 25 x 1000)

= 26.00 mm = 0.26 m

Effective depth, d =200-50-8 = 142 mm =0.142 m.

Moment capacity, φM_n = 0.9 A_sf_y (d-a/2)

= 0.9 x 1339.73 x 413 x {142-(26/2)}x 10^{-6}

= 64.24 kN-m/m.

Ultimate M_{pos}, 47 kN-m/m. < φM_n, 64.24 kN-m/m. **OK**

1.1.3.2 -ve reinforcement:

Here,

M_{neg} = 22.45 kN-m/m.

Ultimate M_{neg}, 22.45 kN-m/m. < φM_n, 64.24 kN-m/m. **OK**

So, R16-150 is provided as –ve reinforcement.

1.1.3.3 Cracking moment:

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

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

(*Ref. AASHTO 07, Art.5.4.2.6*)

Moment of inertia, $I_g = bh^3/12 = 1 \times 0.2^3/12 = 0.0007$ mm^4.

Here,

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

Cracking moment, M_{cr} = f_rI_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**

1.1.3.4 Temperature & shrinkage reinforcement:

Here,

minimum reinforcement required is

A_{st} = 0.003bh /2 = 0.003 x 1000 x 200/2 = 300 mm^2.

(*Ref. AASHTO 07, Art.5.6.3.6*)

Assumed Temp Reinf. is R12-200 = 0.003 x 1000 x 200/2 = 300 mm^2.

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

R12-200 provided as temperature & shrinkage reinforcement on the top layer is OK.

1.1.3.5 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 = 2500-450 = 2050 mm = 2.05 m.

Required percentage of distribution reinforcement, $A_{sd} = 3840/\sqrt{2050} = 84.8\ \%$

So, A_{sd} = 67% of A_s will be provided.

Here, As = R16-150 = 1340 mm^2/m

Then, 67% of A_s = 898 mm^2/m

Using R12 for distribution bar required spacing = (113 x 1000)/898

= 155 mm.

So, R12-150 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 24.0 m (23.35 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 girders. The design is accomplished in accordance with the AASHTO LRFD Bridge Design Specification 07.

Vehicular live loading on the road ways of bridges as 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 1100N axles spaced 1.2 m 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-07, Article 3.6.1.2.1)*

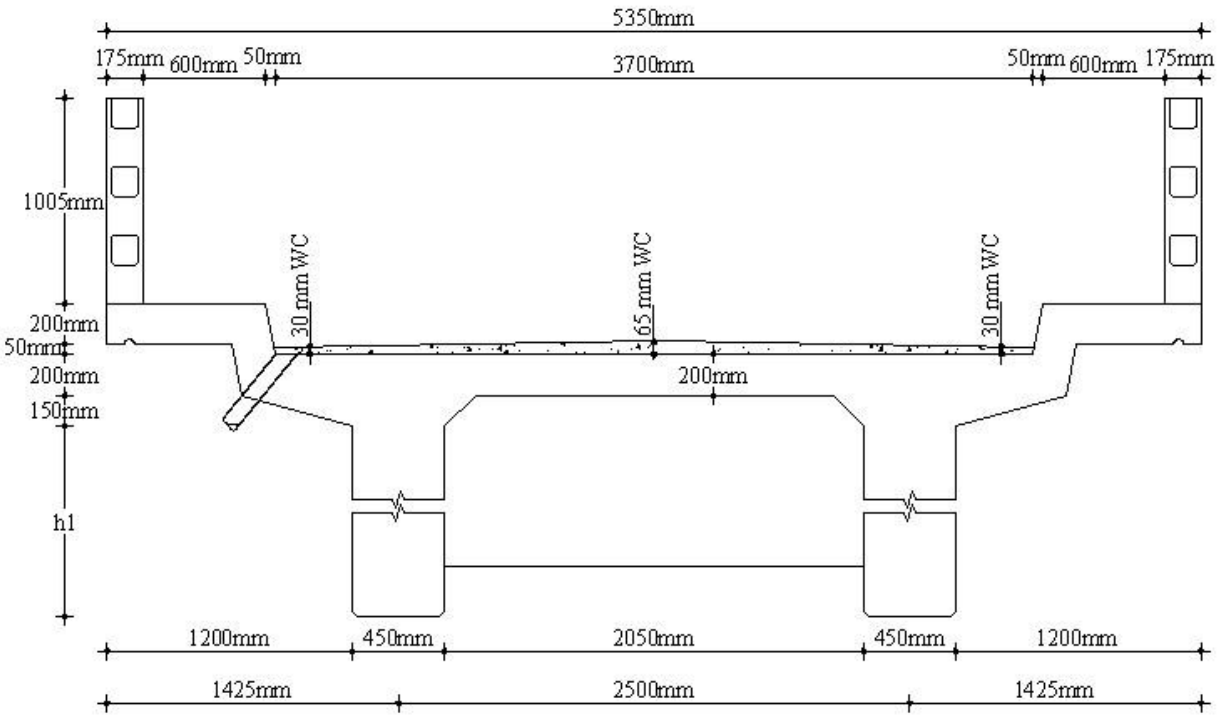


Fig 1.2.1 *Cross Section of Deck Slab*

1.2.2 Geometrical data

Span length	:	24.0 m
Thickness	:	0.2 m
Wearing course, WC	:	0.05 m
Rail Post height	:	1.005 m
Rail Post Width	:	0.175 m
c/c Rail Post spacing	:	1.49 m
Rail bar x-section	:	0.15 m x 0.125 m
Side walk thickness	:	0.2 m
Side walk width	:	0.6 m
Curb height	:	0.25 m
Girder height	:	1.8 m
Girder Width	:	0.45 m
C/C Girder Spacing	:	2.5 m
Diaphragm height	:	1.4 m
Exterior Diaphragm Width	:	0.35 m
Interior Diaphragm Width	:	0.3 m

1.2.3 Material Specifications:

Concrete strength, f'c = 25 MPa
Yield Strength of Reinforcing steel, fy = 413 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

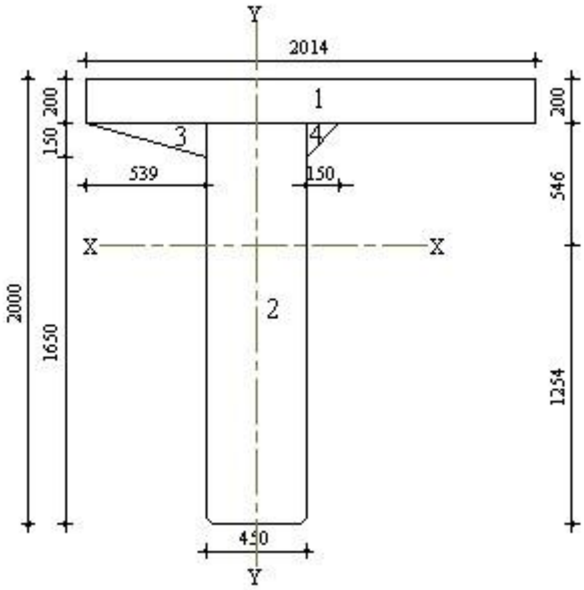


Fig 1.2.2 *Centroid about X-X axis*

Table: Showing Section Properties of Girder

Comp-onent	b	d	A	y	Ay	y_bNA	I	ybar	A(ybar)^2
	mm	mm	mm^2	mm	mm^3	mm	mm^4	mm	mm^4
1	2014	200	4.028E+05	1900	7.653E+08	1253.75	1.343E+09	646.25	1.682E+11
2	450	1800	8.1E+05	900	7.29E+08		2.187E+11	353.75	1.014E+11
3	539	150	4.043E+04	1750	7.075E+07		5.053E+07	496.25	9.956E+09
4	150	150	1.125E+04	1750	1.969E+07		1.406E+07	496.25	2.770E+09
			$\sum$A = 1.264E+06		$\sum$ Ay = 1.585E+09		$\sum$ I = 2.201E+11		$\sum$A(ybar)^2 = 2.823E+11

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

$y_b = \sum Ay / \sum A = 1253.75$ mm

y_t = distance from centroid to extreme top fiber of girder = 746.25 mm

I = moment of inertia of composite section $= \sum I + \sum A(ybar)^2$

$= 5.024E+11 \text{ mm}^4$

S_b = section modulus for the extreme bottom fiber = $I / y_b = 4.007 \times 10^8 \text{ mm}^3$

S_t = section modulus for the extreme top fiber = $I / y_t = 6.732 \times 10^8 \text{ mm}^3$

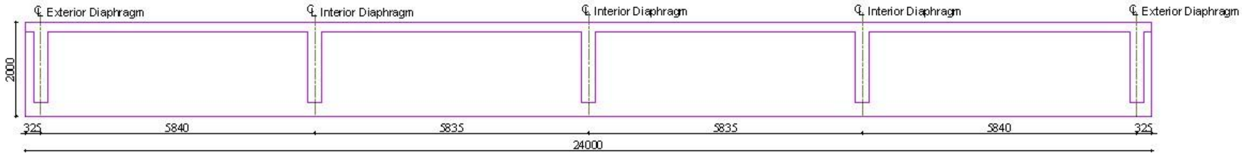


Fig 1.2.3 *Long Section of Girder*

1.2.5 Live load distribution factor

Appling Lever Rule Method the distribution factor for bending moment and share force is obtained 1.28.

1.2.6 Dynamic Allowance

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

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

1.2.7 Girder Design Details

1.2.7.1 Calculation of Weight

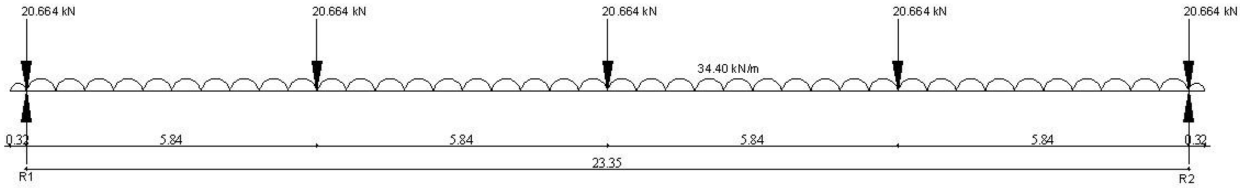
Self wt of girder = 0.45 x 1.8 x 24 = 19.44 kN/m
Diaphragm = 0.35 x 1.4 x 2.05 x 24 = 20.664 kN
Deck Slab = 0.2 x 2.675 x 24 = 12.84 kN/lm
Wearing course = 0.05 x 1.85 x 23 = 2.13 kN/lm

1.2.7.2 Bending Moment Due to Dead Load

Girder Self wt = 19.44 x $23.35^2/8$ = 1324.89 kN-m
Deck Slab = 12.84 x $23.35^2/8$ = 875.10 kN-m
Wearing Course = 2.13 x $23.35^2/8$ = 145.17 kN-m
Weight of Fillets = 2.12 x $23.35^2/8$ = 144.48 kN-m
Weight of Diaphragm = 51.66 x 11.83 – 20.664 x 11.83 – 20.664 x 5.913
= 244.50 kN-m

Dead load moment (Girder, Deck & Cross Girder), M_{DL1}
= 1324.81 + 244.5 + 875.10 + 144.48 = 2589 kN-m
Dead load moment (Wearing Course) = 145.17 kN-m

1.2.7.3 Dead Load Shear Force Calculation



R1 = [34.4 x 23.35 x 11.675 – 34.4 x 0.325 x 0.325/2 + 20.664 (23.35 + 17.512 + 11.675 + 5.8375) + 0.325 x 34.4 x 23.5125] / 23.35
= 464.46 kN.

Note: For the Shear force calculation all the dead loads have been considered as uniform load = 34.4 kN/lm & weight of cross girders have been considered as concentrated load of 20.664 kN.
Maximum shear at support = 464.46 – 20.664 = 443.796 kN
Shear force due to wearing course = 2.13 x 24/2 = 25.56 kN

1.2.7.4 Bending Moment Due to Lane Load

For obtaining maximum moment as single span bridge the middle wheel of truck is placed at the centre of the bridge. *(Ref. AASHTO 2007, Table 3.6.2.1-1)*

DFM = 1.28
P_1 = Rear end wheel load of truck = (72.5 x DFM) = 92.8 kN
P_2 = Middle wheel load of truck = (72.5 x DFM) = 92.8 kN
P_3 = Front wheel load of truck = 17.2 x DFM = 22.775 kN
W = lane load = 9.3 x 0.5 x DFM *(Ref. AASHTO 07 Article No 3.6.1.2.4)*
= 4.70 kN/m
R_1 = (92.8x15.935 + 92.8 x 11.675 + 22.775 x 7.415)/23.35 = 116.96 kN

Maximum moment at mid span due to truck load
= 116.96 x 11.675 – 92.8 x 4.26 = 970.21 kN-m

Total truck load moment with impact moment
= 970.21 (1+IM) = 970.21 (1+0.33) = 1290.39 kN-m

1.2.7.5 Bending Moment Due to Lane Load

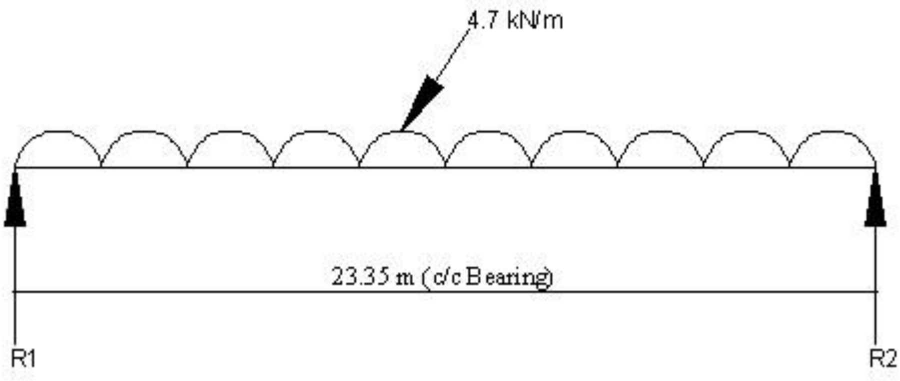


Fig 1.2.7.5 *Loading Diagram of Girder with Lane Load*

Maximum moment due to lane load at mid span = 4.7 x $(23.35)^2$/8 Kn-m
= 320.32 Kn-m

1.2.7.6 Shear Force due to Truck Load

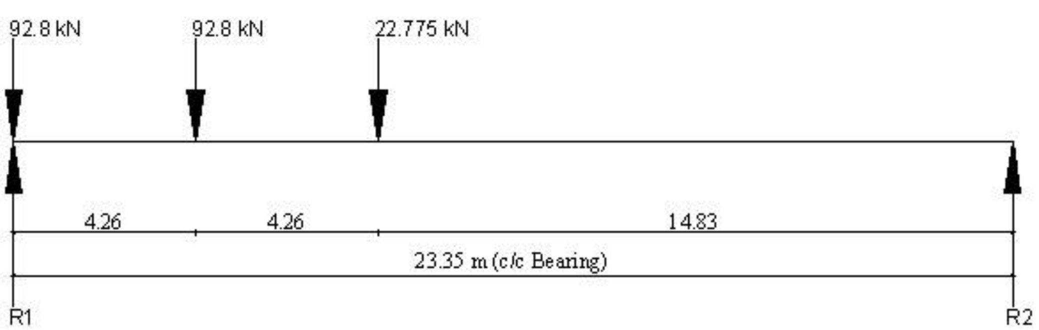


Fig 1.2.7.6 *Loading Diagram of Truck Load for Shear Force Calculation*

R_1 = (92.8 x 23.35 + 92.8 x 19.09 + 22.775 x 14.83)/23.35 = 183.13 kN

Maximum shear at support = 183.13 – 92.8
= 90.33 kN

Maximum shear with impact = 90.33(1 +IM)
= 90.33(1 +0.33)
= 120.14 kN

1.2.7.8 Shear Force due to Lane Load

Lane load = 0.5 x 9.3 = 4.7 kN/m

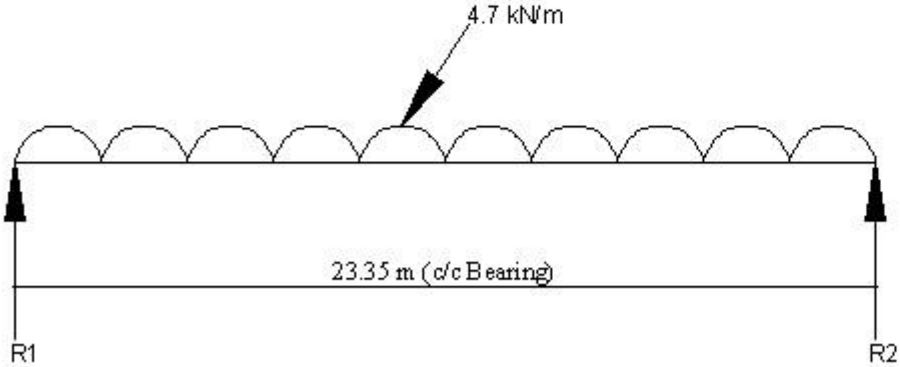


Fig 1.2.7.8 *Loading Diagram of Lane Load for Shear Force Calculation*

$\mathrm{R1} = (4.7 \times 23.35 \times 23.35 \div 2) \div 23.35 = 55\ \mathrm{kN}$

Maximum shear at support $= 1281.27 \div 23.35 = 55\ \mathrm{kN}$

1.2.7.9 Bending Moment from Simplified Conventional Method for Truck Load

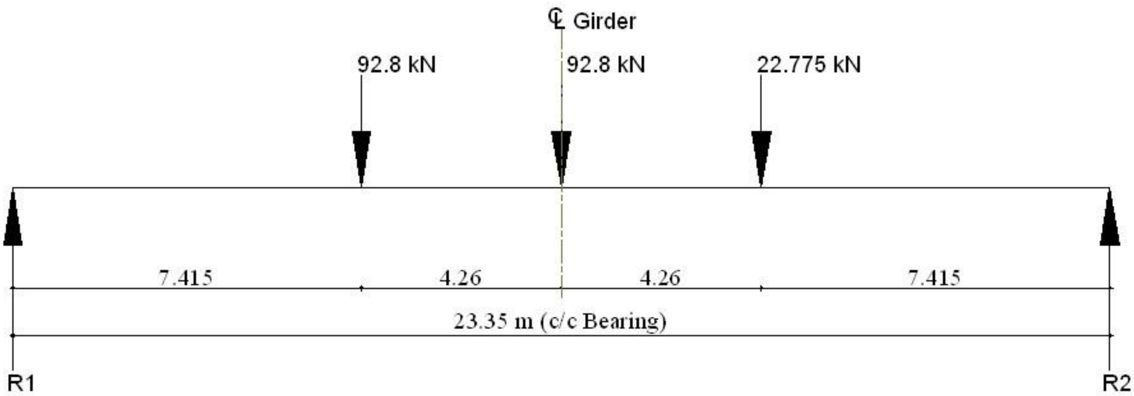


Fig 1.2.7.9 *Loading Diagram of Truck Load for Bending Moment*

Moment at mid section = 970.21 kN-m
Moment with impact factor = 970.21 (1 + 0.33) = 1290.39 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 0.5) = 4.7 kN/m

here, c/c girder distance = 2.5 m

Total lane load moment at mid section

M = 4.7 x 23.35^2/8 = 320.32 kN-m

1.2.7.10 Summary of design moment and shear factored

Total factored moment (Simplified Conventional Method)

= 1.25 x 2588.97 + 1.5 x 145.17 + 1.75 x 1290.39 + 1.75 x 320.32
= 6272.71 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 443.796) + (1.5 x 25.56) + (1.75 x 120.14) + (1.75 x 55)
= 899.58 kN ≈ 900 kN

Design Flexural Moment = 6272.71 kN-m
For calculation of reinforcement, refer to 1.2.8.1.11 below.

1.2.7.11 Crack Width Calculation

Bar stacing for maximum crack width of 0.22mm as per *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 T12 for shear reinforcement) = 50 + 6 = 56
$\beta_s = [\{\ 1 + d_c\ \{\ 0.7(\ h- d_c)\}] = [\ \{\ 1 + 56/\{0.7(2000\text{-}56)\}] = 1.076$

Service bending moment = 4344.85 kN-m = 4344 x10^6 N-mm

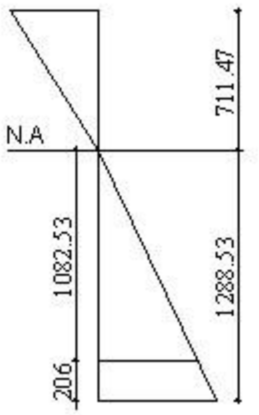


Fig 1.2.7.11 *Stress Diagram of Girder*

$f_b = M\ C_b / I = (4344 \times 10^6 \times 1288.53)/5.338 \times 10^{11} = 10.49$ N /mm^2
f_{sc} = 8.82 N/mm^2, f_{sc} = 10.49/1288.53 x 1082.53]
f_{ss} = 8082 x 8.33
= 73.47 MPa

$f_{ss} = (E_s/E_c) \times f_{sc}$
$E_c = 24 \times 10^3$ MPa
$E_s = 200 \times 10^3$ MPa
$E_s/E_c = 8.33$

$S \leq (123 \times 10^3\ \times 0.5)\ \div (73.47\ \times 1.076) - 2 \times 56$
≤ 666 mm

BS Method:

$f_t = (4344.85 \times 10^6 \times 711.47)/5.33 \times 10^{11} = 5.80\ N/mm^2$

Concrete strain = $5.80 / 24 \times 10^3 = 0.00024$

Crack width = $3\ \varepsilon_{min} \times a_{cr} = 3 \times 0.00024 \times 107.35$ [$a_{cr} = \sqrt{(100^2 + 55^2)} = 107.35$]

= 0.08 < 0.2 Maximum crack width 0.22 mm.

1.2.7.12 Deflection for 23.35m Girder (due to live load):

Deflection due to beam self wt,

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

w = beam self wt. = 19.44 kN/m

$\Delta_g = (5 \times 19.44 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338)$

= 0.0059 m = 5.9 mm (↓)

Deflection due to slab wt,

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

w = slab wt. = 14.96 kN/m

$\Delta_s = (5 \times 14.96 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338)$

= 0.0045 m = 4.5 mm (↓)

Deflection due to X-girder load,

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

P = X-girder load = 20.664 x 3 = 61.992 kN

$\Delta_{x\text{-girder}} = (61.992 \times (23.35)^3 \times 1000)/(48 \times 24 \times 10^6 \times 0.5338)$

= 1.28 mm (↓)

Deflection due to wearing course,

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

w = wearing course wt. = 2.13 kN/m

$\Delta_{wc} = (5 \times 2.13 \times (23.35)^4)/ (384 \times 24 \times 10^6 \times 0.5338) = 0.6$ mm (↓)

Deflection due to lane load,

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

w = lane load = 4.7 kN/m

$\Delta_{LL} = (5 \times 4.7 \times (23.35)^4)/ (384 \times 24 \times 10^6 \times 0.5338)$

= 1.4 mm (↓)

Deflection due to truck & impact load,

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

P = wheel load = (92.8 x 2 + 22.775) x 1.33 x Dynamic allowance

= 277 kN

$\Delta_{WL} = (277 \times (23.35)^3 \times 1000)/(48 \times 24 \times 10^6 \times 0.5338)$

= 5.73 mm (↓)

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

= (23.35) x 1000/800 = 29.19 mm

Total live load deflection = 5.73 + 1.4 = 7.13 mm < 29.19 mm **OK**

1.2.7.13 Reinforcement calculation

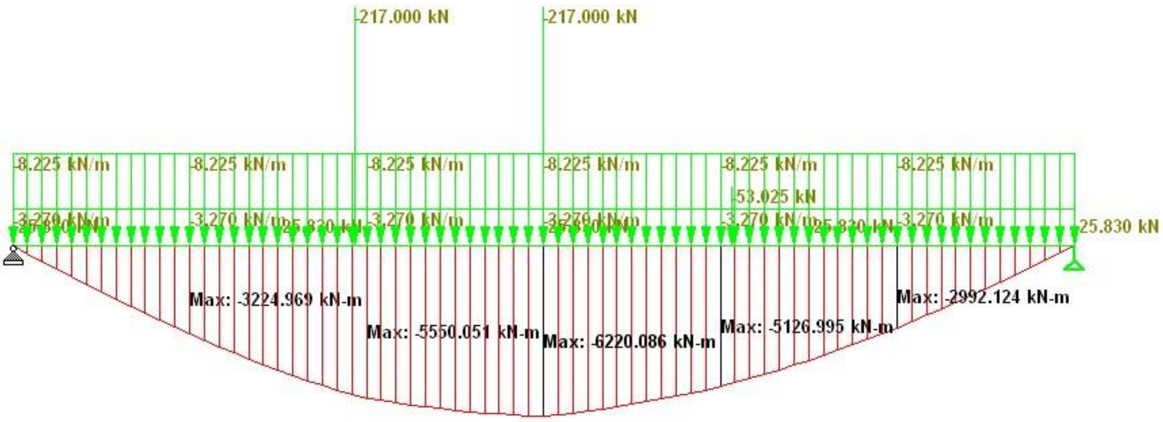


Fig 1.2.7.13.1 *Moment Diagram of Girder*

a) Flexural Moment

Total moment, M_u = 6272.71 kN-m (Simplified Conventional Method).

Using, 16 - T32 = 12862 mm^2

Effective depth of girder, d = 1626 mm

Lever arm factor, a = $A_s f_y /0.85 \times f'_c \times b$

= (12862 x 413) / (0.85 x25 x 2050)

= 122 mm

Nominal moment, ΦM_n = 0.9 x A_s x f_y (d – a/2)

= 0.9 x 12862 x 413 (16826 – 122/2) / 10^6

= 8438 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, $M_{cr} = S_c(f_r+f_{cpe})-M_{dnc}(S_c/S_{nc}-1)$

$= (f_r \times I_g)/ y_t$

Modules of rupture, fr (for normal density concrete) = 0.52 √fc′

(Ref. AASHTO 07 Art 5.4.2.6)

= 0.52 √25

= 2.6 MPa

Distance from N.A to the top fiber of the section, yt = 711.47

Moment of inertia, I = 5.338×10^{11} mm^4
Cracking moment, M_{cr} = $((2.6 \times 5.338 \times 10^{11}) / 711.47)/106$ = 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
= 0.003 x 1000 x 450 / 2
= 675 mm^2

Providing T-12,
Spacing = (1000 x 113) / 675
= 167 mm
≈ 150 mm

Effective depth = 1800 mm
Total no. of surface reinforcement = 1800 / (150x2)
= 6

d) Shear Reinforcement

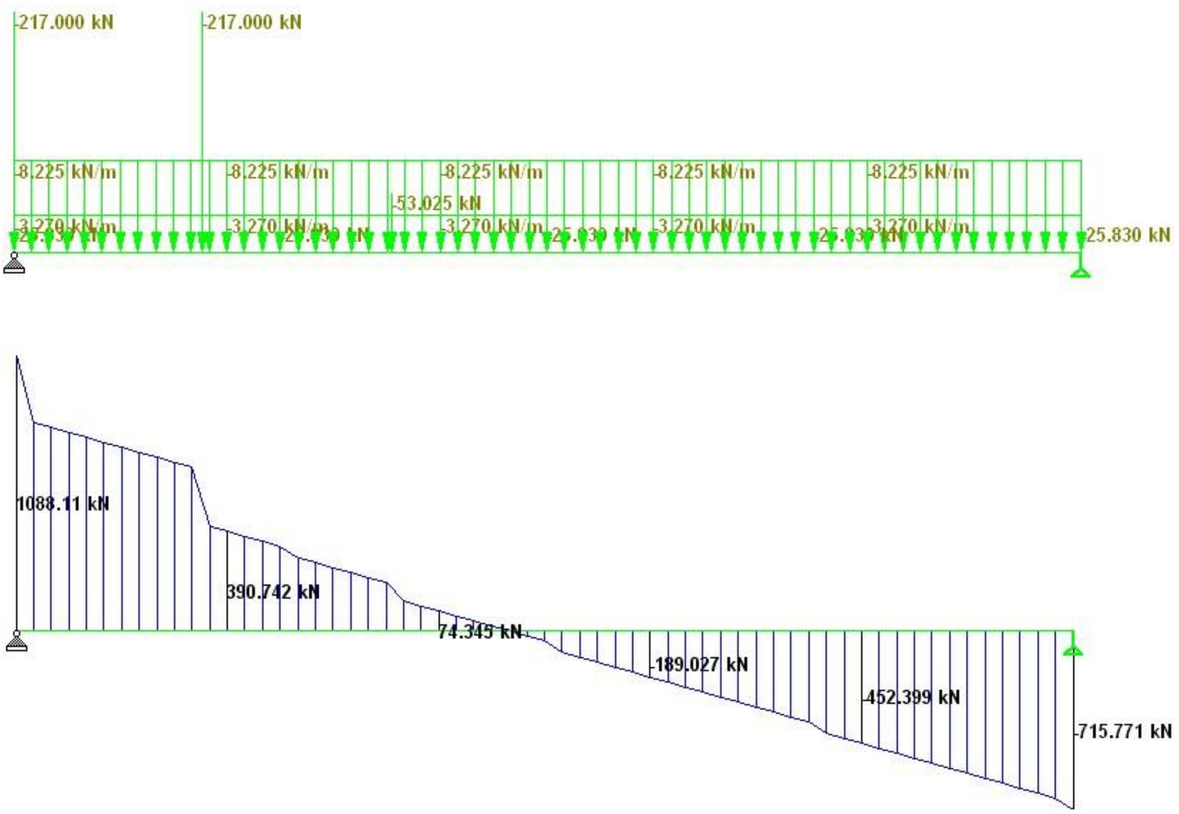


Fig 1.2.7.13.2 *Shear Force Diagram of Girder*

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

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

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

$d_v = M_n / A_s\, f_y$ *(Ref. AASHTO Art 5.8.2.9)*

$= As\ f_y(d – a/2)/ A_s\, f_y$

$= 1738$ 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

$a = (A_s\, f_y)/ (0.85\ f'_c\, b) = 125$ 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.

$v_u / f_c' = 0.69/25$

$= .028$

here, v_u = total factored shear/area

$= (900/1333 \times 10^3) \times 1000$

$= 0.675\ N/mm^2$ from AASHTO Table-5.8.3.4.2-1

Let, $\varepsilon_x = 0.001$

$\theta = 36.4$

$\beta = 2.23$

Hence,

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

$= 0.83 \times 2.23 \times \sqrt{25} \times 450 \times 1738 / 1000$

$= 724$ 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)*

$= (2 \times 113 \times 413 \times 1738 \times 1.37)/150$

$= 1482$ kN

$V_n = V_c + V_s + V_p$

$= 724 + 1482 + 0$

$= 2206$ kN

$V_n = 0.25 f_c' b_v d_v + V_p$

$= 0.25 \times 25 \times 450 \times 1738 + 0$

$= 4888.13$ kN

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

So, $V_n = 2206$ kN

Therefore,

$V_n = 2206$ kN

$V_u = 900$ kN

To insure more safely against shear stress critical shear have been taken at support.

$V_u < V_n$ so OK.

So we provide R10-150 at a distance 3m from support and then provide T12-200 after distance 3m to mid section.

iii) Checking wheather Transverse Reinforcement needed

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

$V_u = 900$ kN

$0.5 \times \varphi \times V_c = 0.5 \times 0.9 \times 724 = 326$ kN

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

iv) Checking Minimum Transverse Reinforcement

$A_v \geq 0.083\sqrt{f_c'}\, b_v s / f_y = 0.083\sqrt{25} \times 450 \times 150/413 = 68\ mm^2$

(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,

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

If $v_u \geq 0.125 \times fc'$ then,

$S_{max} = 0.4\, d_v \leq 300$ mm *(Ref. AASHTO Art. 5.8.2.7-2)*

$v_u = V_u/A = 900 \times 10^3 / 1333 \times 10^3 = 0.675\ N/mm^2$

$v_u / fc' = 0.675 / 25 = 0.027$

Therefore,

$S_{max} = 0.8 d_v \leq 600$ mm $= 0.8 \times 1738 \leq 600 = 1390 \leq 600$

Provided maximum spacing = 200 mm < 600 mm, so ok.

1.3 STRUCTURAL DESIGN OF PC GIRDER

1.3.1 Introduction

The design example demonstrates the design of a 30.0 m span prestressed concrete girder. For this example, 200mm thick RC deck slab, 50 mm wearing course are considered. This example illustrates in detail the design of single lane exterior girders. The design is accomplished in accordance with the AASHTO LRFD Bridge Design Specification 2007.

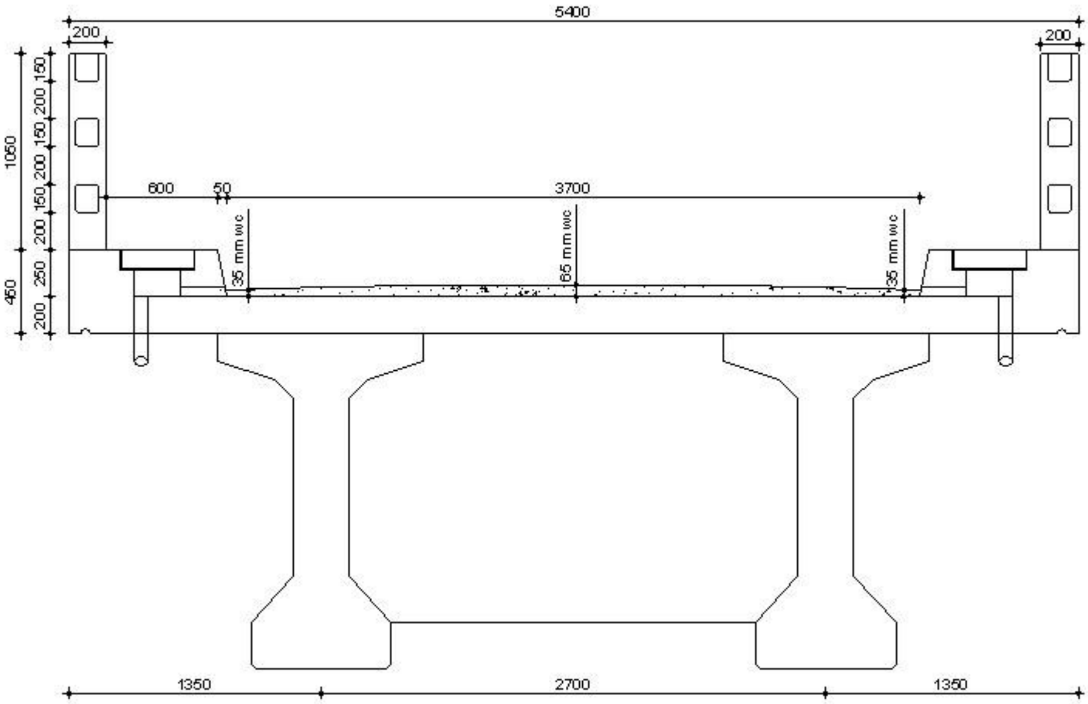


Fig 1.3.1.1 *Bridge Cross Section*

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 = 30 m

Design span = 29.30 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 = 4

Ultimate strength, f_{pu} = 1860 MPa

Yield strength, f_{py} = 0.9 f_{pu}= 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 x 1860 = 1395 Mpa

At service limit state after all Losses,

$f_{pe} \leq 0.80\ f_{py}$ = 0.80 x 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 = 413 MPa

Modulus of elasticity, E_s = 200,000 MPa

1.3.3 Section Property:

1.3.3.1 Non Composite Section at Mid Span:

For normal density concrete with, γ_c = 2400 kg/ m^3

Modulus of elasticity = $4800\sqrt{f'_c}$

Therefore, Modulus of elasticity for cast in place slab = $E_c = 4800\sqrt{25}$

= 24000 MPa

Pre cast girder at transfer, Eci = $4800\sqrt{26.25}$ = 24593 MPa

Pre cast girder at service load, Ec = $4800\sqrt{35}$ = 28397.18 MPa

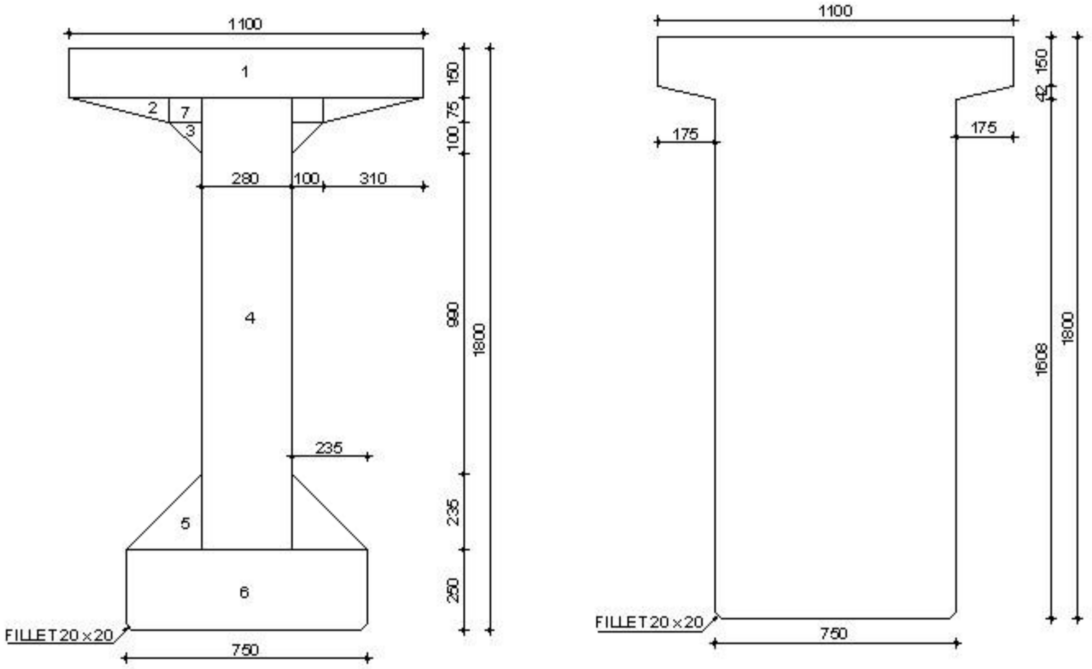


Fig 1.3.3.1.1 *Non Composite Mid & End-Section*

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

Component	b	h	A	Y	AY	Y_b	I	Ybar	A*(Ybar)^2
1	1100	150	1.65E+05	1725	2.85E+08	915.1	3.09E+08	809.9	1.07E+11
2	310	75	2.33E+04	1625	3.78E+07		7.27E+06	709.9	1.16E+10
3	100	100	1.00E+04	1541.667	1.54E+07		5.56E+06	626.567	3.88E+09
4	280	1400	3.92E+05	950	3.72E+08		6.40E+10	34.9000	3.84E+08
5	235	235	5.52E+04	328.33	1.81E+07		1.69E+08	586.70314	1.77E+10
6	750	250	1.88E+05	125	2.34E+07		9.77E+08	759.90314	1.18E+11
7	100	75	1.50E+04	1612.5	2.42E+07		7.03E+06	697.40000	7.22E+09
Sum			8.48E+05		7.76E+08		6.55E+10		2.66E+11

y_b = Distance from centroid to the extreme bottom fiber of non-composite girder
= 915.1 mm

y_t = Distance from centroid to extreme top fiber of non-composite girder
= 884.9mm

∑ I = 3.26E+11 mm4

S_b = Section modulus for the extreme bottom fiber = I/ y_b = 3.56x 10^8mm^3

S_t = I/ y_t = 3.68x 10^8mm^3

Calculation for effective flange width:

Effective flange width of exterior girder shall be taken as ½ of the girder spacing plus the lesser of

1. 1/8 x span — 3750 mm
2. 6 x ts + greater of ½ x web thickness or ¼ x beam top flange width — 1475 mm
3. Width of the over hang Deck — 1350 mm

then effective flange width — 2700 mm

1.3.3.2 Composite Section at Mid Span

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

= 0.85

Transformed flange width = n x (effective flange width) = 0.85 x 2700

= 2295mm

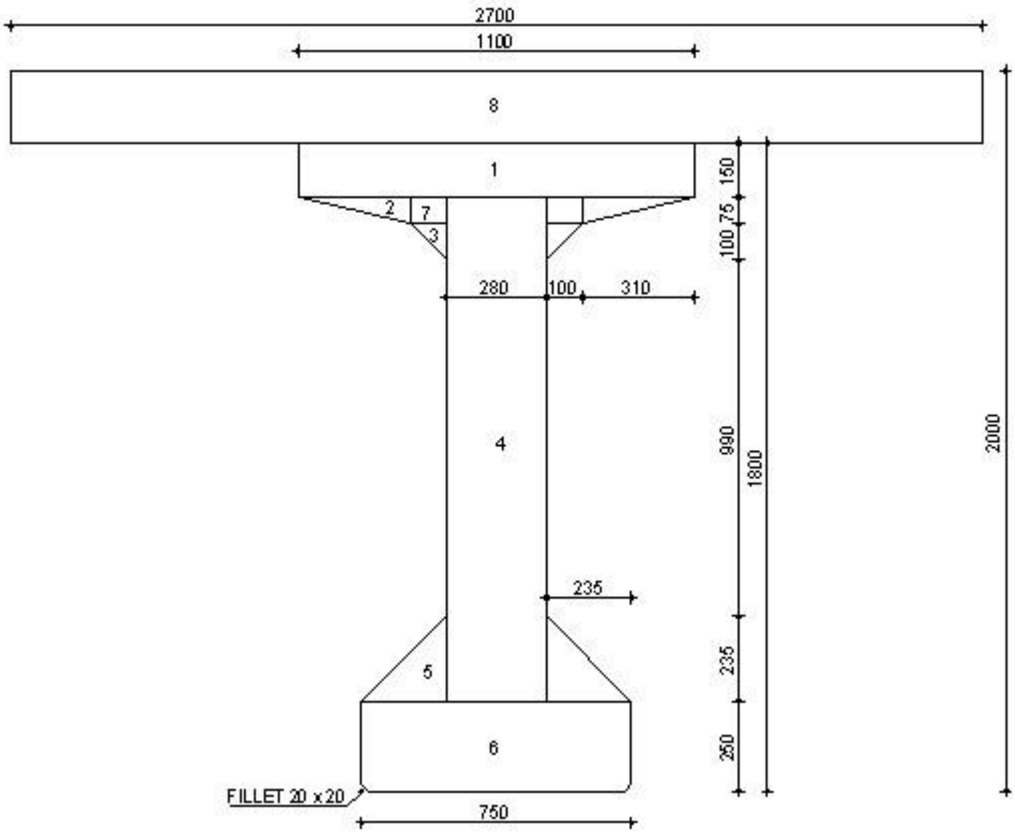


Fig 1.33.2.1 *Composite Mid-Section*

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

Component	b	h	A	Y	AY	Y_{bc}	I	Ybar	A*(Ybar)^2
1	1100	150	1.65E+05	1725	2.85E+08		3.09E+08	4.25E+02	2.98E+10
2	310	75	2.33E+04	1625	3.78E+07		7.27E+06	3.25E+02	2.46E+09
3	100	100	1.00E+04	1541.6667	1.54E+07		5.56E+06	2.42E+02	5.84E+08
4	280	1400	3.92E+05	950	3.72E+08		6.40E+10	3.50E+02	4.80E+10
5	235	210	4.94E+04	320.00	1.58E+07	1299.93	1.21E+08	9.80E+02	4.74E+10
6	750	250	1.88E+05	125	2.34E+07		9.77E+08	1.17E+03	2.59E+11
7	100	75	1.50E+04	1612.5	2.42E+07		7.03E+06	3.13E+02	1.47E+09
8	2700	200	5.40E+05	1900	1.02E+09		1.78E+09	6.00E+02	1.93E+11
Sum			1,377,100.00		1.79E+09		6.72E+10		5.81E+11

A_c = Total area of composite girder
= 1.377 E+06 mm2
h_c = overall depth of the composite section = 1800 + 200 = 2000 mm
I_c = moment of inertia of the composite section
= 6.48E+11
y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the pre-cast girder = 1299.93 mm
y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the deck
= 700.07mm
y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the pre-cast girder = 500.07mm
S_{bc} = Composite section modulus for the extreme bottom fiber of the pre cast girder
= I_c / y_{bc} = 4.98x 10^8 mm3

S_{tg} = Composite section modulus for the top fiber of the pre-cast girder = I_c / y_{tg}
= 1.29×10^{9} mm^3
S_{td} = Composite section modulus for the extreme top fiber of the deck
= $1/n \times (Ic / y_{tc})$
= 1.088×10^9 mm^3

1.3.4 Calculations for Exterior Girder:

1.3.4.1 Dead Load Moment :

Self wt. moment	= 2315.04 kN-m
Cross girder moment	= 159.22 kN-m
Deck slab moment	= 1444.5 kN-m
Wearing cource moment	= 382.95 kN-m
Sidewalk slab moment	= 523.13 kN-m
Rail Post moment	= 55.13 kN-m
Rail bar moment	= 187.31 kN-m

1.3.4.2 Total Dead Load Moment:

Total service dead load moment = 5067.3 kN/m

Total factored dead load moment = 6430 kN-m

1.3.4.3 Simplified Lever rule for live load distribution factor:

For exterior girder:

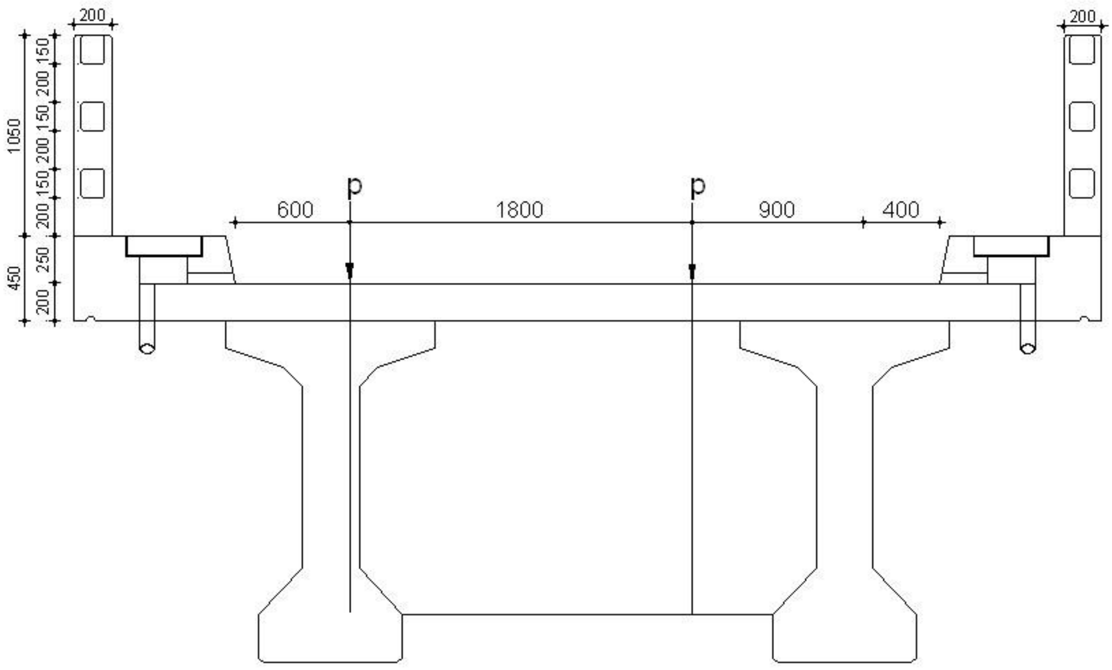


Fig 1.3.4.3.1 *Truck Load Distribution (exterior girder)*

Here p = wheel load.
Load per interior girder = P + (2700-1800)/2700 x P
= 1.33 x P

Axle load distribution factor = 1.33/2 = 0.67

1.3.4.4 Dynamic Allowance

IM = 33%

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

(Ref. AASHTO 07, Table 3.6.2.1-1)

1.3.4.5 Calculation of Live load moment for exterior girder:

1.3.4.5.1 Moment due to truck load:

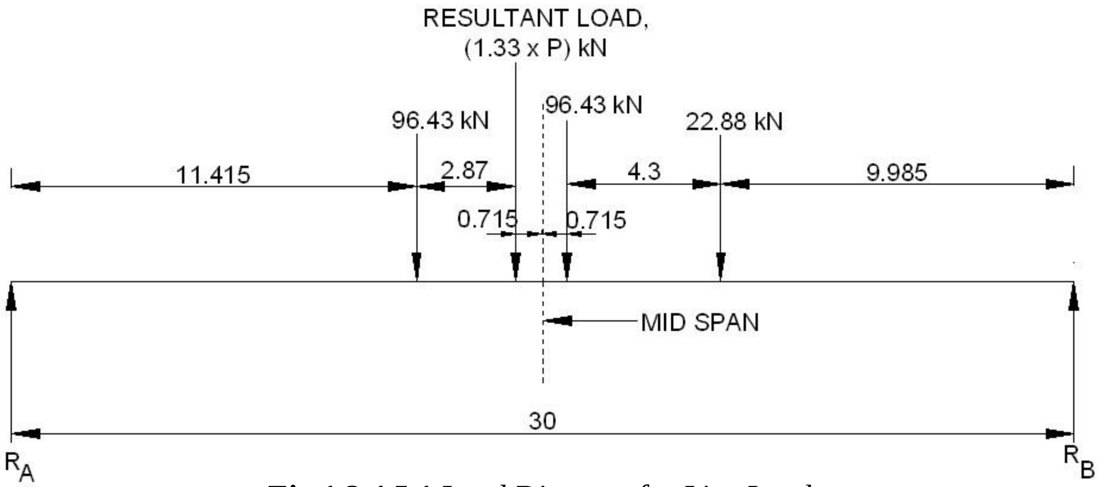


Fig 1.3.4.5.1 *Load Diagram for Live Load*

Rear wheel load	= 96.43 kN
Front wheel load	= 22.88 kN
RA	= 113.27 kN

Maximum service moment at mid span due to truck load with distribution factor = 1353.35 kN-m

For Impact Moment, 1353.35 x 1.33 = 1799.96 kN-m

1.3.4.5.2 Moment due to lane load:

Lane load, w = 5.7 kN/m *(Ref. AASHTO 07, Art. 3.6.1.2.4)*

$$M = wl^2 / 8$$
$$= 5.7 \times 30^2 / 8$$
$$= 641.25 \text{ kN-m}$$

Total live load moment = 2441.21 kN-m

1.3.4.6 Calculation of Shear Force for Exterior Girder:

1.3.4.6.1 Shear force due to truck load

Rear wheel load	96.43 kN
Front wheel load	22.88 kN
Reaction at support	195.36 kN
Shear force	98.93 kN
with impact	131.58 kN

1.3.4.6.2 Shear force due to lane load

Max shear force = 85.50kN

1.3.4.6.3 Total service dead load shear

Shear due to self wt of girder	= 335.77 kN
Shear due to X girder	= 16.28 kN
Shear due to deck slab	= 192.6 kN
Shear due to WC	= 53.28 kN
Shear due to side walk slab, railing, & rail post	= 102 kN
Total service dead load shear	= 699.93 kN
Total service live load shear	= 217.08 kN
Total factored dead load shear	= 888.3 kN
Total factored live load shear	= 379.89 kN
Total factored shear	= 1268.19 kN

1.3.5 Calculation of Losses:

(Ref. AASHTO' 07 Art. 5.9.5.1)

1.3.5.1 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:

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

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.

Radius of curvature, $R_v = L^2/8H_h$

$$R_H = L^2/8H_v$$

$\alpha = \sqrt{(\alpha_V^2 + \alpha_H^2)}$
$= X/R$

$\mu = 0.25$
K = 0.007

Length of cable subjected to pre-stress loss due to wedge pull-in,
$X_A = \sqrt{\{(\Delta wp\ Eps\ Aps)/\Delta P\}}$

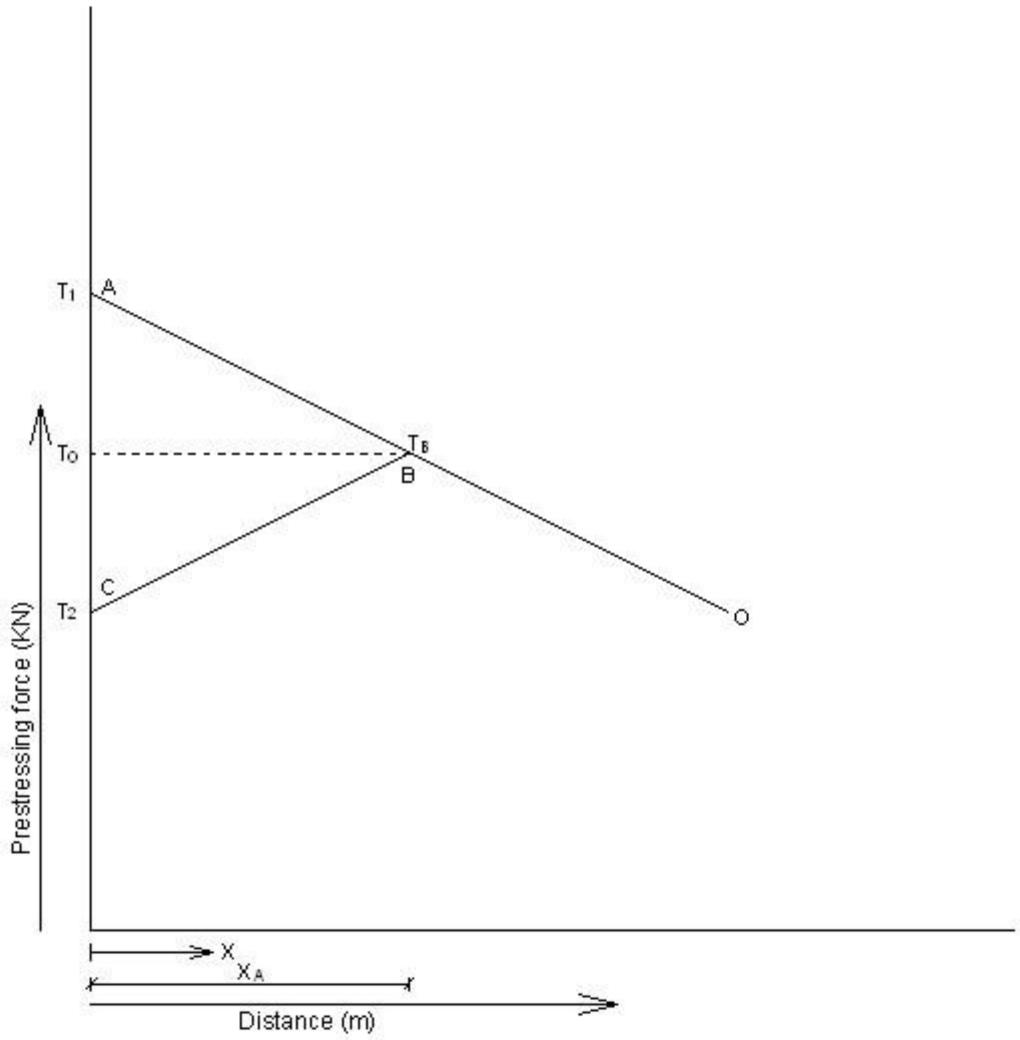


Fig 1.3.5.1.1 *Loss of prestress Due to Friction and Wedge Pull-in*

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 X is greater than Xa prestress force in the cable will be the ordinate of line BO.

Here,

Δwp = 7 mm

Eps = 197×10^{6} kN/m^2

Aps = $98.71 \times 10^{-6} \times 12 = 1184.4 \times 10^{-6}$ m^2

Table: Calculation of sag & radius of curvature

Cable no.	Vertical sag, H_v	Horizontal sag, H_h	Radius of curvature, R_v	Radius of curvature, R_h
1	330	200	340.91	562.50
2	680	200	165.44	562.50
3	1030	0	109.22	0.00
4	1230	0	91.46	0.00

Table: Calculation of wedge pull in and friction loss

Cable No.	Initial prestress force, Pi,(kN)	ά = X/R for X=1 (rad)	Loss of prestress force per unit length Δp,(kN/m)	Distance of wedge pull-in XA (m)	ά = X/R for XA distance (rad)	Cable force at XA distance (kN)	ά = X/R for X = 15m distance (rad)	Cable force at X = 15m (kN)	Loss of Prestress%
1	1652	0.003	12.93	11.24	0.039	1512.59	0.051	1468.539	11.12
2	1652	0.006	14.11	10.76	0.068	1506.61	0.095	1452.816	12.07
3	1652	0.009	15.28	10.34	0.095	1500.93	0.137	1437.345	13.01
4	1652	0.011	16.00	10.10	0.110	1497.51	0.164	1427.794	13.58
									49.78

Average loss of pre-stress = 12.44 %

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

Elastic Shortening due to Post-Tensioning,

$$\Delta f_{pES} = \frac{N-1}{2N}\frac{E_p}{E_c} f_{cgpt}$$

$$f_{cgpt} = \frac{F_{pt}}{A_g} + \frac{F_{pt} e_{pt}^2}{I_g} - \frac{M_{assembly} e_{pt}}{I_g}$$

$$F_{pt} = A_{pt}\left(f_{pj} - \Delta f_{pF} - \Delta f_{pES}\right)$$

F_{pt} (kN)	A_g (mm^2)	I_g (mm^4)	e_{pt} (mm)	$M_{assembly}$ (kN-m)	f_{cgpt} (MPa)
6137.40	842100	3.316E+11	761.20	2315.04	12.70

F_{pt} is the post-tensioning force immediately before anchor set

N	E_p (MPa)	E_c (MPa)	f_{cgpt} (MPa)	Δf_{pES3} (MPa)
4	197000	28397.18	12.70	33.03

c) Elastic Gain Due to Deck and other loads acting on the non-composite section:

$$\Delta f_{pED1} = \frac{E_p}{E_c} \Delta f_{cd1}$$

$$\Delta f_{cd1} = -\frac{M_{nc} e_{pt}}{I_g}$$

$$M_{nc} = M_{slab} + M_{diaphragm} + M_{UserDC} + M_{UserDW}$$

Here M_{nc} is used for deck and cross girder only.

E_p (MPa)	E_c (MPa)	M_{nc} (kN-m)	I_g (mm^4)	e_{pt} (mm)	Δf_{cd1} (MPa)	Δf_{pED1} (MPa)
197000	28397.18	1607.25	3.316E+11	761.203143	-3.7	-25.6

d) Elastic Gain Due to Traffic Barrier and other superimposed loads acting on the composite section:

$$\Delta f_{pED2} = \frac{E_p}{E_c} \Delta f_{cd2}$$

$$\Delta f_{cd2} = -\frac{M_c e_{cpt}}{I_c}$$

$$M_c = M_{trafficbarrier} + M_{overlay} + M_{UserDC} + M_{UserDW}$$

Here Mc is used for wearing course only (for interior girder).And for exterior girder barrier load and other superimposed load should be included.

E_p (MPa)	E_c (MPa)	M_c (kN-m)	I_c (mm^4)	e_{cpt} (mm)	Δf_{cd2} (MPa)	Δf_{pED2} (MPa)
197000	28397.18	1148.51	6.48E+11	1142.4	-2.02	-14.04

Δf_{pcd1} = -($M_{nc}e_{pt}$)/ Ig
M_{nc} = M_{slab} + $M_{diaphragm}$
e_{pt} = eccentricity of prestressing force with respect to centroid of noncomposite section
Δf_{pcd2} = -($M_c e_{cpt}$)/ Ic
M_c = $M_{barrier}$ + $M_{w/c}$
e_{cpt} = eccentricity of prestressing force with respect to centroid of composite section.
f_{cgpt} = 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
= $P/A+Pe^2/I-M_G e/I$
F_{pt} = effective jacking force.

1.3.5.2 Determination of Long Term Losses*:*

Long term loss = sum of time dependant prestress losses between transfer and deck placement + sum of time dependant prestress losses after deck placement.

Creep and Shrinkage Parameters:

$$k_s = 1.45 - 0.005\left({}^{V}\!/_{S}\right) \geq 1.0$$

$$k_{hc} = 1.56 - 0.008H$$

$$k_{hs} = 2.00 - 0.014H$$

$$k_f = \frac{37}{7 + f'_a}$$

$$k_{td} = \left(\frac{t}{61 - 0.58f'_a + t}\right)$$

$$\psi(t, t_i) = 1.9k_s k_{hc} k_f k_{td} t_i^{-0.118}$$

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} \times 0.48 \times 10^{-3}$$

k_s = factor for the effect of the volume to surface ratio of the component

k_{hs} = humidity factor for shrinkage.

k_f = factor for the effect of the concrete strength

k_{td} = time development factor

k_{hc} = humidity factor for creep

t = maturity of concrete (day) defined as the age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculation, and time being considered for analysis of creep or shrinkage effects.

t_i = age of concrete at time of load application(day)

V/S = volume to surface ratio (mm)

H = The average annual ambient relative humidity (%)

= 70% (Assume)

V/S Girder (mm)	**kvs Girder**	**V/S Deck (mm)**	**kvs Deck**	**H**	**k_{hc}**	**k_{hs}**
135	0.775	93	0.985	70	1	1.02

f'_{ci} (MPa)	**f'_c (MPa)**	**k_f Girder**	**k_f Deck**
26.25	35	1.11278	0.88095

t_i (day)	**t_{pt} (day)**	**t_d (day)**	**t_f (day)**	**k_{td} $t=t_{pt}$**	**k_{td} $t=t_d$**	**k_{td} $t=t_f$**	**k_{td} $t=t_f - t_d$ $f'_{ci}=f'_c$**
0.04	7	60	25000	0.1326	0.567242	0.998172	0.998371

t_i = Starting time of construction

t_{pt} = time counting from start time to post tensioning

t_d = time counting from start time to deck placement

t_f = time counting from start time to final time (service stage)

$\psi_b(t_{pt},t_i)$	**$\psi_b(t_d,t_i)$**	**$\psi_b(t_f,t_i)$**	**$\psi_b(t_{pt},t_d)$**	**$\psi_b(t_f,t_{pt})$**
0.318	1.359	2.391	1.041	2.074

ε_{bipt}x 1000	**ε_{bid}x 1000**	**ε_{bif}x 1000**	**ε_{ddf}x 1000**
0.0560	0.240	0.421	0.424

a) Loss due to Shrinkage of Girder Concrete:

$$\Delta f_{pSRipt} = 0$$
$$\Delta f_{pSRptd} = \varepsilon_{bptd} E_p K_{id}$$
$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$$
$$\varepsilon_{bipt} = \varepsilon_{sh}\left(t = t_{pt}\right)$$
$$\varepsilon_{bid} = \varepsilon_{sh}\left(t = t_{d}\right)$$
$$\varepsilon_{bif} = \varepsilon_{sh}\left(t = t_{f}\right)$$
$$\varepsilon_{bptd} = \varepsilon_{bid} - \varepsilon_{bipt}$$
$$\varepsilon_{bdf} = \varepsilon_{bif} - \varepsilon_{bid}$$
$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{pt}}{A_g} \left(1 + \frac{A_g e_{pt}^2}{I_g}\right)\left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]}$$
$$K_{df} = \frac{1}{1 + \frac{E_p}{E_c} \frac{A_{pt}}{A_c} \left(1 + \frac{A_c e_{cpt}^2}{I_c}\right)\left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]}$$

E_p (MPa)	E_{ci} (MPa)	A_{pt} (mm^2)	A_g (mm^2)	I_g (mm^4)	e_{pt} (mm)	$\psi_b(t_f,t_i)$	K_{id}
197000	24593	4737.6	842100	3.32E+11	761.2031429	2.391	0.771

E_p (MPa)	E_c (MPa)	A_{pt} (mm^2)	A_c (mm^2)	I_c (mm^4)	e_{cpt} (mm)	$\psi_b(t_f,t_i)$	K_{df}
197000	28397.18	4737.6	1.38E+06	6.48E+11	1142.4	2.391	0.806

ε_{bipt}x 1000	ε_{bid}x 1000	ε_{bif}x 1000	K_{id}	K_{df}	Δf_{pSRptd} (MPa)	Δf_{pSD} (MPa)
0.056004508	0.23950876	0.421462288	0.771	0.806	27.85	28.89

b) Loss due to Creep of Girder Concrete:

$$\Delta f_{pCRipt} = 0$$
$$\Delta f_{pCRptd} = \frac{E_p}{E_{ci}} f_{cgpt}\left[\psi_b\left(t_d, t_i\right) - \psi_b\left(t_{pt}, t_i\right)\right] K_{id}$$
$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgpt}\left[\psi_b\left(t_f, t_i\right) - \psi_b\left(t_d, t_i\right)\right] K_{df} + \frac{E_p}{E_c}\left(\Delta f_{cd1} + \Delta f_{cd2}\right)\psi_b\left(t_f, t_d\right) K_{df} \geq 0.0$$

E_p (MPa)	E_c (MPa)	E_{ci} (MPa)	$\psi_b(t_{pt},t_i)$	$\psi_b(t_d,t_i)$	$\psi_b(t_f,t_d)$	$\psi_b(t_f,t_i)$	K_{id}	K_{df}
197000	28397	24593	0.317754783	1.359	1.032	2.391	0.771	0.806

f_{cgpt} (MPa)	Δf_{cd1} (MPa)	Δf_{cd2} (MPa)	Δf_{pCRptd} (MPa)	Δf_{pCD} (MPa)
12.6981	-3.7	-2.024	81.602	51.660

c) Loss due to Relaxation of Tendon:

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)$$

$$f_{pt} = f_{pj} - \Delta f_{pF} - \Delta f_{pES3} - \Delta f_{pA}$$

$$\Delta f_{pR1ipt} = 0$$

$$\Delta f_{pR1ptd} = \left(\frac{t_d - t_{pt}}{t_d - t_i}\right)\Delta f_{pR1}$$

$$\Delta f_{pR2} = \Delta f_{pR1}$$

f_{pj} (MPa)	Δf_{pF} mid-span (MPa)	Δf_{pA} mid-span (MPa)	Δf_{pES3} (MPa)	f_{pt} (MPa)	f_{py} (MPa)	K_L	Δf_{pR1} (MPa)
1395	57.49	0	33.03	1304.48	1674	30	9.969

K_L = 30 for low relaxation strand
f_{pt} = stress in prestressing strands immediately after transfer, taken not less than $0.55f_{py}$.

t_i (day)	t_{pt} (day)	t_d (day)	Δf_{pR1ptd} (MPa)	Δf_{pR2} (MPa)
0.04	7	60	8.81	9.969

d) Loss due to Shrinkage of Deck:

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}\left[1+\psi_b\left(t_f,t_d\right)\right]$$

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left[1+\psi_b\left(t_f,t_d\right)\right]}\left(\frac{1}{A_c} + \frac{e_{cpt} e_d}{I_c}\right)$$

$$\varepsilon_{ddf} = \varepsilon_{sh}\left(t = t_f - t_d\right)$$

ε_{ddf} x 1000	A_d (Deck area per Girder) (mm^2)	E_{cd} (MPa)	$\psi_b(t_f,t_d)$	A_c (mm^2)	I_c (mm^4)	e_{cpt} (mm)	e_d (mm)	Δf_{cdf} (MPa)
0.424	535000	24000	1.032	1.38E+06	6.48E+11	1142.434585	-600.07	-0.887

E_p (MPa)	E_c (MPa)	Δf_{cdf} (MPa)	$\psi_b(t_f,t_d)$	K_{df}	Δf_{pSS} (MPa)
197000	28397	-0.887	1.032	0.806	-10.082

Δf_{cdf} = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete.
ε_{ddf} = shrinkage strain of deck concrete between placement and final time.
A_d = area of deck concrete.
E_{cd} = modulus of elasticity of deck concrete.
e_d = eccentricity of deck with respect to the gross composite section.

Summary of Post-Tension Losses:

Elastic Shortening/Gain	MPa
Initial Δf_{pES3}	33.03
Due to Deck Placement Δf_{pED1}	-25.59
Due to Superimposed Dead Loads Δf_{pED2}	-14.04

Shrinkage of Girder Concrete	MPa
PT to Deck Placement Δf_{pSRptd}	27.85
Deck to Final Δf_{pSD}	28.89
Due to Shrinkage of Deck Δf_{pSS}	-10.082

Creep of Girder Concrete	MPa
PT to Deck Placement Δf_{pCRptd}	81.602
Deck to Final Δf_{pCD}	51.660

Relaxation of Tendons	MPa
PT to Deck Placement Δf_{pR1ptd}	8.81
Deck to Final Δf_{pR2}	9.969

Schedule of Stress for Girder (at mid-section)

Stage 1	**Stressing of the Girder: i. Selfweight only; ii. Non-composite section**				Stress, MPa		
Sl. No.	Description		Axial Force, kN	Moment, kN-m	Girder Bottom	Girder Top	Deck top
1	Moment due to Selfweight			2315.04	-6.41	6.15	
2	Axial force due to Prestressing		6608.95		7.85	7.85	
3	Moment due to Prestressing			5031	13.94	-13.37	
4	Friction Loss	12.44	%		-2.71	0.69	
5	Elastic Shortening loss	2.37	%		-0.52	0.13	
6	Wedge pull-in loss	0.00	%		0.00	0.00	
Check 1	*Allow. Compression,0.6f'ci*	***15.75***			**12.146**	**1.449**	
	Allowable tension,0.25(f'ci)$^{1/2}$	***-1.28***					

Stage 2	**Deck casting: i. Non-composite section; ii. Full concrete strength of girder**				Stress, MPa		
7	Moment due to deck & cross-girder load			1607.25	-4.45	4.27	
8	Elastic gain due to deck & cross-girder load	-1.83	%		0.40	-0.10	
9	Shrinkage of Girder concrete	2.00	%		-0.44	0.11	
10	Creep of girder concrete	5.85	%		-1.27	0.32	
11	Relaxation of prestressing steel	0.63	%		-0.14	0.03	
Check 2	*Allow. Compression,0.45f'c*	***15.75***			**6.246**	**6.087**	
	Allowable tension,0.25(f'c)$^{1/2}$	***-1.48***					
Stage 3	**Service condition: i. Composite section; ii. Full concrete strength of girder & deck**				Stress, MPa		
12	Moment due to other dead load (barrier, overlay etc.)			1148.51	-2.30	0.53	0.89
13	Elastic gain due to other dead load	-1.01	%		0.22	-0.06	-0.07
14	Live load moment			2375.84	-4.76	1.10	1.83
15	Final Shrinkage loss of girder concrete	2.07	%		-0.45	0.11	0.15
16	Final creep loss of girder concrete	3.70	%		-0.81	0.20	0.26
17	Final Relaxation loss of prestressing steel	0.71	%		-0.16	0.04	0.05
18	Shrinkage of deck	-0.72	%		0.16	-0.04	-0.05
Check 3	*Allow. Compression,0.60f'c with LL*	***21.00***			**-1.857**	**7.981**	**3.057**
	Allow. Compression,0.45f'c without LL	***15.75***			**2.906**	**6.881**	**1.225**
	Allowable tension,0.5(f'c)$^{1/2}$	***-2.96***					

Note: -ve value indicates Tensile stress & +ve indicates Compressive

1.3.6 Calculation of Moment Capacity:

Average stress in pre-stressing steel when fpe $\geq$.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

$= h - y_{bs} = 2000 - 157 = 1843$ mm

[y_{bs} = distance between the c.g of the prestressing cables and bottom concrete fiber of the beam = 157]

c = distance between the neutral axis and the compression face

Assuming rectangular behavior:

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

$= (12 \text{ x}98.7\text{x } 4\text{x } 1860 +0\text{-}0\ )/\{(0.85\text{x}25\text{x}0.85\text{x}2675)+(0.28 \text{ x } 12 \text{ x } 98.7 \text{ x } 4 \text{ x } 1860/1843)\}$

$= 8811936/(48317.18+1338.76) = 177.45 \;< h_f$ mm

$= 177.45<200$ mm **ok**

$a \;= \beta_1 \text{x } c = 0.85 \text{ x } 177.45 = 150.84\text{mm}$

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

$= 1860(1 - 0.28 \text{ x } 177.45/1843) = 1806\text{MPa}$

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

$\Phi \;= 1$ *(Ref. AASHTO 07, Art.5.5.4.2)*

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

$= (12\text{x}4\text{x}98.7)\text{x}1806{*}(1843\text{-}161.09/2) +0+0+0.85\text{x}35(\ 2675\text{-}2675)\text{x}200{*}(161.09/2 -200/2)$

$= 15079$ kN-m

$M_r > M_u$ OK

1.3.7 Calculation of Non-prestressing Reinforcement:

Surface reinforcement = 0.003bt *(Ref. AASHTO 07, Art. 5.6.3.6)*

$= 0.003 \text{ x } 1000 \text{ x } 280/2 = 420 \text{ mm}^2 / \text{m}$

Providing T-10

Spacing = 1000 x 78.5/420 = 186 mm

Total no. of surface bar needed = 1015/186 x 2 = 11

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

The nominal shear resistance Vn shall be determined as lesser of

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

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

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 = 280 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) $d_v = M_n/A_sf_y = A_sf_y(d\text{-}a/2)/A_sf_y = (d\text{-}a/2) = (1843\text{-}150.84/2) = 1767.58$ mm

(ii) 0.72h = 0.72 x 2000 = 1440 mm

(iii) 0.9de = 0.9 x 1843 = 1658.7 mm

Hence dv = 1767.58 mm

1.3.8.1 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

$= [1260 \times 10^3 / 1.377 \times 10^6] \times 1/1000$

$= 9.1 \times 10^{-4}$

$v_u/f'c = 9.1 \times 10^{-4}/35 = 2.61 \times 10^{-5}$

Referred from AASHTO 07 Table: 5.8.3.4.2

θ = 36.4, β=2.23

$\therefore V_c = 0.083\beta\sqrt{f'c}\, b_v d_v$ *(Ref. AASHTO 07, Art. 5.8.3.3)*

$= 0.083 \times 2.23 \times \sqrt{35} \times 280 \times 1767.58$

= 541943.59N

$\therefore V_s = A_v f_y d_v \mathrm{Cot}\theta/S$

Here, S = 125 mm C/C

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

$V_s = 226 \times 420 \times 1767.58 \times 1.36/125$

= 1825432.186 N

1.3.8.2 Calculation of Prestress Force Component:

Effective prestress force of Cable No.1P_1= 1652-2*(1652-1512.59) = 1373.18kN

Effective prestress force of Cable No.2 P2 =1652-2*(1652-1506.61) =1361.22kN

Effective prestress force of Cable No.3 P_3 =1652-2* (1652-1500.93) =1349.86kN

Effective prestress force of Cable No.4 P_4 =1652-2* (1652-1497.51) =1343.02kN

V_{P1} =	1373.18 x .052	= 71.5kN	θ_1= 2.98
V_{P2} =	1361.22 x .095	= 129.32 kN	θ_2= 5.47
V_{P3} =	1349.86 x .138	= 186.28 kN	θ_3= 7.95
V_{P4} =	1343.02 x .165	= 221.59 kN	θ_4= 9.49

Total V_P = 608.69 kN

V_n = 2367.98 kN

Again V_n = (0.25 x 35 x 1767.58 x 280+608690)/1000

= 4939.26 kN

Hnce V_n = 2367.98 kN>1260 kN, Hence OK.

1.3.9 Calculation of Deflection:

Uniformly distributed load due to prestressing force at transfer,

c.g of cables at end = 975 mm

c.g of cables at mid = 157.5mm

$w = 8Fh/L^2$

$= 8 \times 1652 \times 4 \times 0.817/ 30^2$ Here, h = 817.5 mm

= 48.03 kN/m

Deflection due to prestressing force at transfer,

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

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

I = moment of inertia of non-composite girder = .332 m^4

Δ_P = 62 mm

Net deflection due to pretress = $62- [(0.04 \times 1652 \times 4) \times 30^2 / 8 \times 24.59 \times 10^6 \times 0.403] \times 1000$

= 59 (↑)

Deflection due to beam self wt,

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

w = beam self wt. = 27 kN/m

$\Delta_g = (5 \times 27 \times (30)^4) / (384 \times 24.59 \times 10^6 \times 0.332)$

= 34.88 mm (↓)

Deflection after transfer = 59-34.88 = 24.12 mm (↑)

Deflection due to X-girder,

$\Delta_{xg} = (65.1 \times 30^3 / 48 \times 28.39 \times 10^6 \times 0.332) \times 1000 = 3.88$mm

Deflection due to slab wt,

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

E_c = modulus of elasticity of concrete at service stage = 28.39×10^6 kN/m^2

w = slab wt. = 12.84 kN/m

$\Delta_s = [(5 \times 12.84 \times (30)^4) / (384 \times 28.39 \times 10^6 \times .648)] = 7.36$mm (↓)

Deflection due to wearing course,

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

w = wearing course wt. = 3.4 kN/m

$\Delta_{wc} = (5 \times 3.4 \times (30)^4)/ (384 \times 28.39 \times 10^6 \times .648) = 1.94$ mm (↓)

Deflection due to lane load,

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

w = lane load = 5.7 kN/m

$\Delta_{LL} = (5 \times 5.7 \times (30)^4)/ (384 \times 28.39 \times 10^6 \times .648)$

= 3.26 mm (↓)

Deflection due to truck & impact load,

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

P = wheel load = (145x2+35) x DFM x Dynamic allowance
= 325x0.64x1.33 = 276.64 kN
Δ_{WL} = (276.64 x $(30)^3$x1000)/(48 x 28.39 x 10^6 x .648)
= 8.45 mm (↓)
Allowable deflection for live load = L/800 (Ref. AASHTO 07 Art. 2.5.2.6.2)
= (30) x 1000/800 = 37.5 mm
Total live load deflection = 11.71mm<37.5 mm, Hence OK.

1.3.10 Design of End block

Taking bursting force = 0.25∑Pu(1-a/h)+0.5{∑(Pu sinά) }
(Ref. AASHTO 07, Art.5.10.9.6)
The design force for post tensioning, = 1.2 x jacking force
here, Pu = factored tendon force = 1652*1.2 = 1982.4 kN
a = lateral dimention of anchorage divice = 240 mm
h = lateral dimention of cross section = (1100+750)/2 = 925mm
ά = angle of inclination of tendon force with respect to the centerline of the member.
Bursting force,
Total T_{burst} = 1914.08 kN
Average T_{burst} = 1914/4 = 478.52kN
Yield stress of steel, f_y = 420 Mpa
Allowable stress, fs = 207.5 Mpa
A_t = 478.52*10^3 /207.5 = 2306.12 mm^2 *(Ref. AASHTO 07, Art.10.9.3.2)*
But according to the manufacturer specification 36-T12-120 vertical bar is provided.

1.3.11 Cable Profile Details

12K-13 Cable requires duct size of 75 mm.

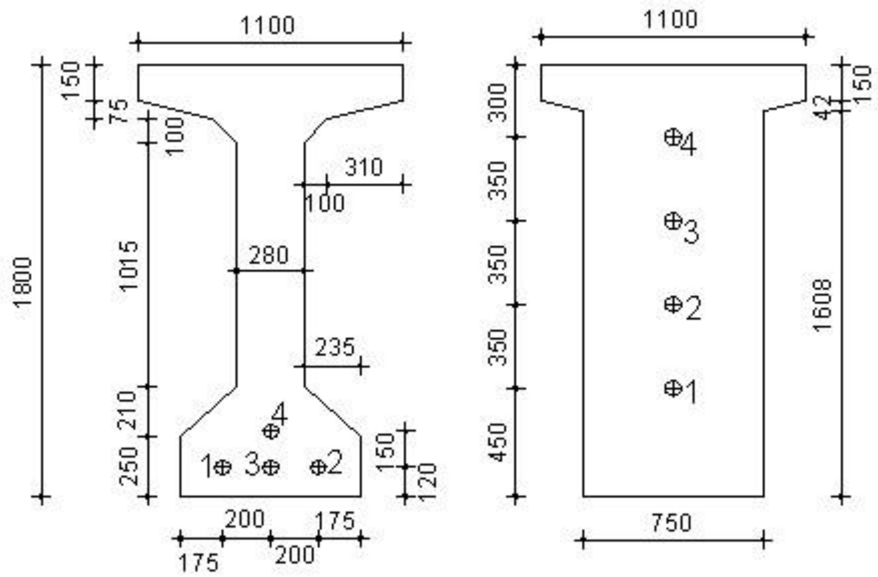


***Fig 1.3.11.1** Cable Profile of End and Mid-section of PC girder*

Table: Cable Profile Ordinates.

Cable No.	Cable Position				Cable		DISTANCE 'X' FROM END TOWARDS THE C/L IN mm																				Emergence Angle, Φ	Elongation at each end excluding grip, Δ (mm)
	End		Mid		Sag, a		**14850**		**13850**		**13000**		**12000**		**10000**		**8000**		**6000**		**4000**		**2000**		**0**			
	Y	Z	Y	Z	Y	Z	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Y**	**Z**	**Degree**	
1	**450**	**0**	**120**	**200**	330	200	450	0	407	0	373	24	335	50	270	96	216	133	174	162	144	183	126	196	120	200	2.98	99.57
2	**800**	**0**	**120**	**-200**	680	200	800	0	712	0	641	-24	564	-50	428	-96	317	-133	231	-162	169	-183	132	-196	120	-200	5.47	99.14
3	**1150**	**0**	**120**	**0**	1030	0	1150	0	1016	0	909	0	793	0	587	0	419	0	288	0	195	0	139	0	120	0	7.95	98.80
4	**1500**	**0**	**270**	**0**	1230	0	1500	0	1340	0	1213	0	1073	0	828	0	627	0	471	0	359	0	292	0	270	0	9.49	98.61

CHAPTER 2

SUBSTRUCTURE & FOUNDATION

2.0 STRUCTURAL DESIGN OF ABUTMENT-WING WALL AND PILE CAP

2.1 Model Details

2.1.1 General

STAAD.Pro 2006 has been used as a data processing tool for design purpose. Full view of 5.0m 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 5.0m model with 12 piles with superstructure load of 24.0m span (23.35c/c bearing) is chosen for presenting the structural design example of Abutment-Wing wall and Pile Cap.

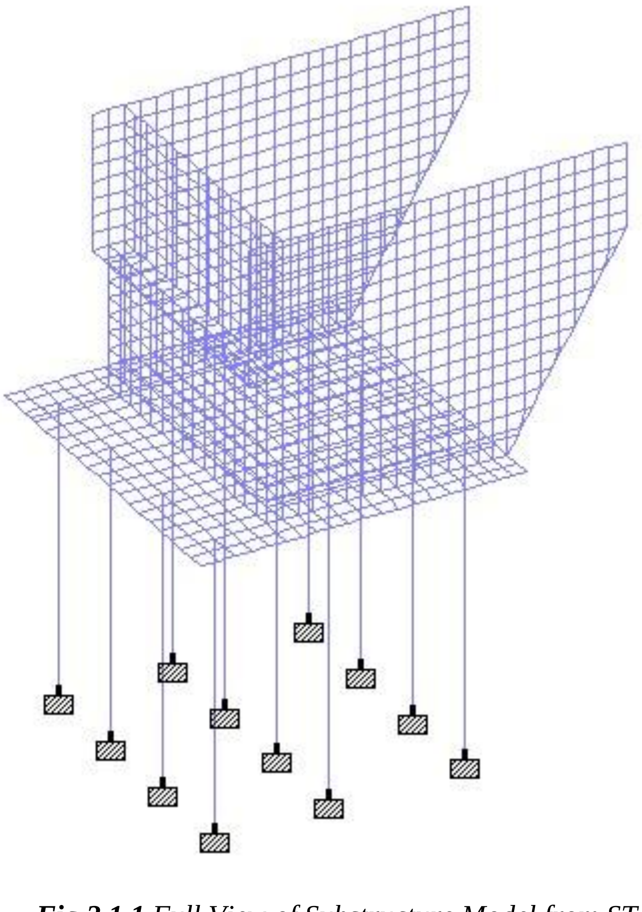


Fig 2.1.1 *Full View of Substructure Model from STAAD.Pro*

2.1.2 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.

2.1.3 Pile Reaction

Pile reactions at nodes of 12 piles are given below which are based on inelastic analysis and thus shows flexible distribution of pile reaction.

Table 2.1 Pile Reactions at Nodes for Service Load from STAAD.Pro Model

Node	**FY,** kN	Node	**FY,** kN
13	449.234	19	482.353
14	565.222	20	505.617
15	345.136	21	160.038
16	482.538	22	450.261
17	505.970	23	564.811
18	160.246	24	344.830

2.2 Abutment Wall Design

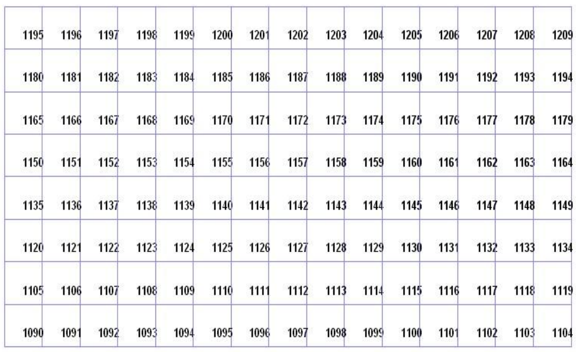


***Fig 2.2.1** Node Numbers of Abutment Wall*

***Fig 2.2.2** Plate Numbers of Abutment Wall*

Abutment wall is designed considering plate properties with variable thickness of 350mm at top to 500mm at bottom. Plate 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.

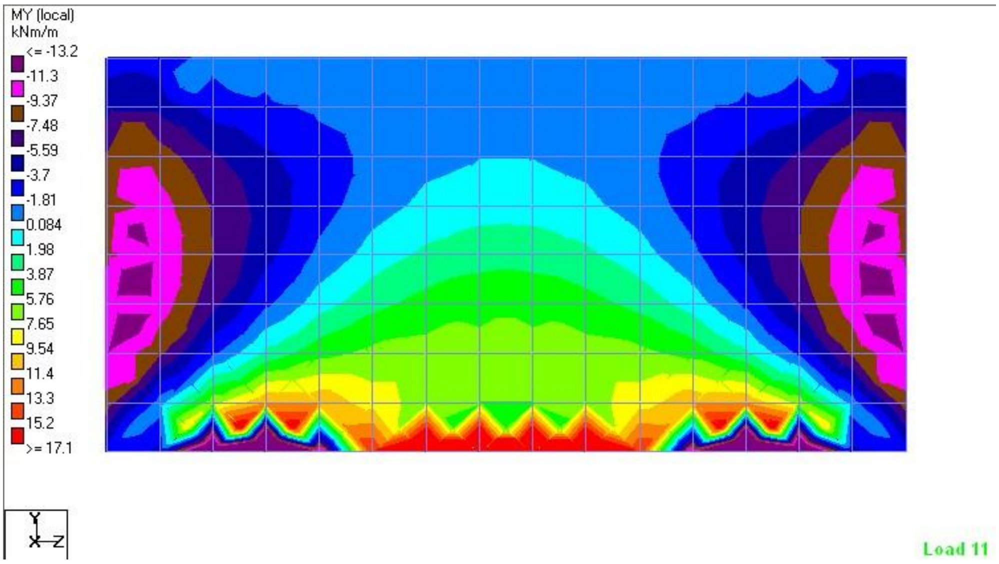


***Fig 2.2.3** Bending Moment Contour of Abutment Wall (With Plate Nos), M_y, kN-m*

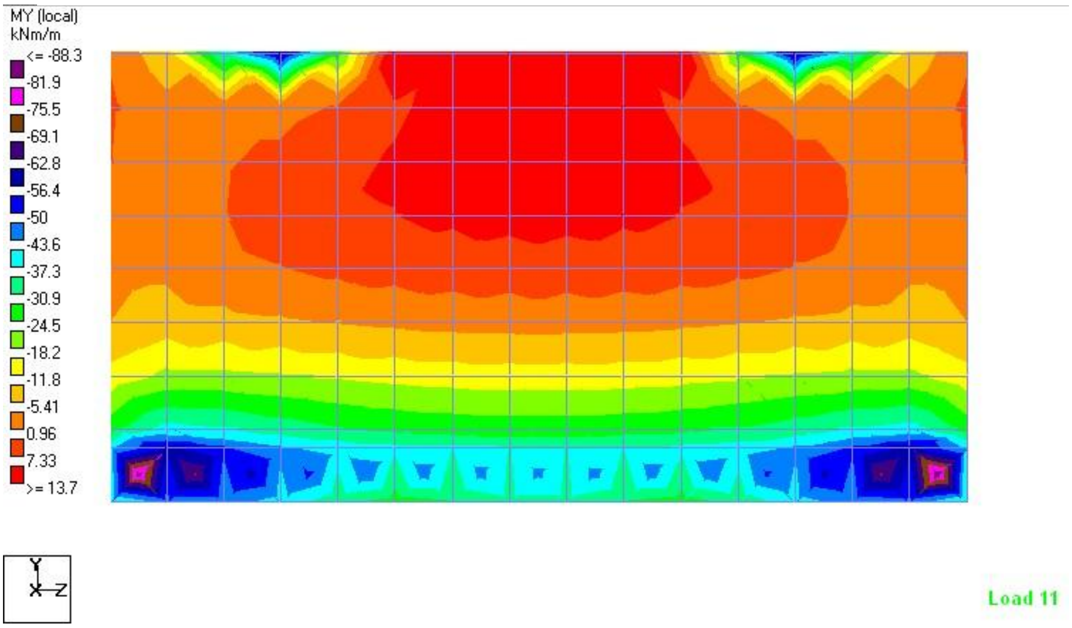


***Fig 2.2.4** Bending Moment Contour of Abutment Wall (With Plate Nos), M_y, kN-m*

From the plate stress results, maximum flexural moment, My is found at plate no 56 of value 89 kN-m/m along vertical direction which is found at the bottom section at Earth-Face(E/F).This value continues to decrease with increasing height of Abutment wall. This value seemed to be significantly less so the minimum reinforcement requirement according to AASHTO 07 is checked the detail calculation is provided below.

Reinforcement Design Calculation

Vertical Reinforcement For earth face (E/F),

Design Moment M_u = 89 kN-m/m

Compressive strength of Concrete, fc′= 25 MPa

Thickness of abutment wall = 0.5 m

Modulus of Rupture = 0.52 x √ fc′= 2.6 MPa

Moment of Inertia = b x h^3/12 = 0.01 m4 (B=1m , H=0.5m)

Cracking Moment= 2.6 x 1000 x 0.01/ (0.5/2) = 104 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment = 125 kN-m/m

Criteria 2, 1.33 x Flexural Moment M_u = 119 kN-m/m

Minimum reinforcement required is for 119 kN-m/m which is greater than M_u =89 kN-m/m. So, design moment considered is 119 kN-m/m.
R16-150 is provided for design moment of 119 kN-m/m. Considering convenience of working condition in the field this spacing of reinforcement is provided for the entire abutment wall.

Vertical Reinforcement For Water Face (W/F)

Design Moment M_u = 14 kN-m/m
Compressive strength of Concrete, fc′= 25 MPa
Thickness of Abutment Wall = 0.5 m
Modulus of Rupture = 0.5 2 x √ fc′= 2.6 MPa
Moment of Inertia = b x h^3/12 = 0.01 m4 (B=1m , H=0.5m)
Cracking Moment = 2.6 x 1000 x 0.01/ (0.5/2) = 104 kN-m/m
Checking against AASHTO 07 criteria for minimum reinforcement,
Criteria 1, 1.2 x Cracking moment = 125kN-m/m
Criteria 2, 1.33 x Flexural Moment = 19 kN-m/m.

Minimum reinforcement required is for 19kN-m/m which is greater than M_u (14 kN-m/m). So, design moment finally considered is 19kN-m/m. This is significantly less than the temperature-shrinkage reinforcement requirement. So we provide R16-200.

Horizontal Reinforcement Design

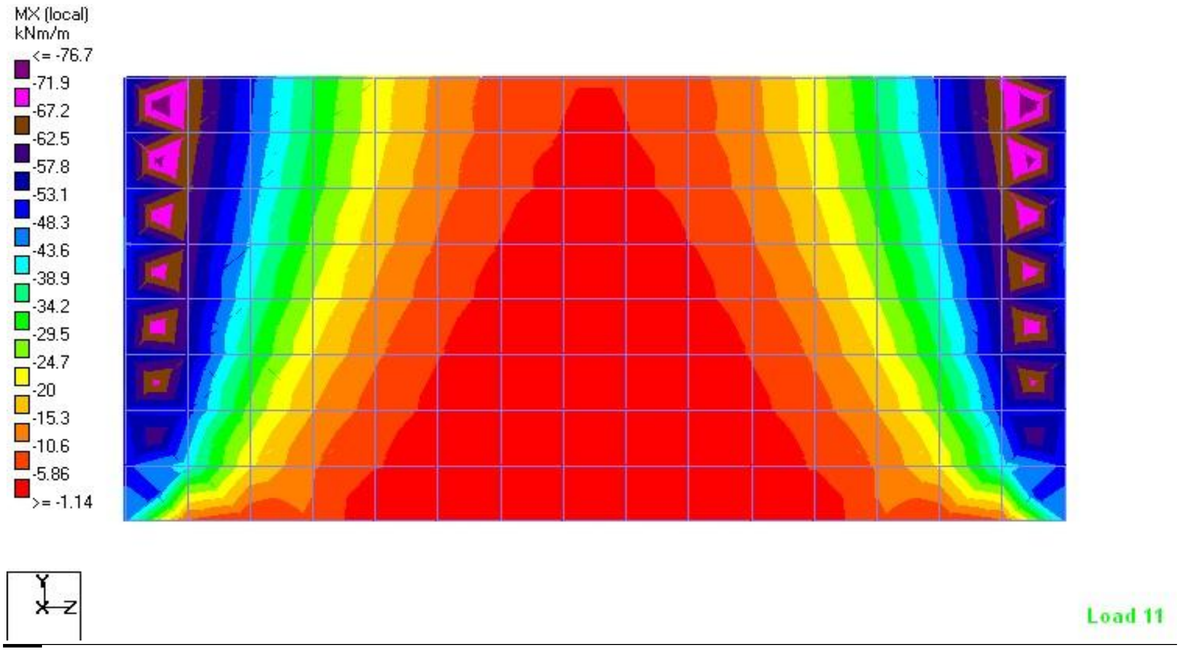


***Fig 2.2.5** Bending Moment contour of abutment wall (with plate numbers,) M_x,kN-m*

It is observ that maximum bending moment are at plate no 1195 and 1209 That is Maximum tensile stress has developed at earth face near at joining section between abutment and wing wall.

The detail calculation are provided below.

Reinforcement Design Calculation

Horizontal Reinforcement For Earth face (E/F)

Design Moment M_u = 77 kN-m/m
Compressive strength of Concrete, fc´= 25 MPa
Thickness of Wall = 0.35 m
Modulus of Rupture = 0.52 x √ fc´= 2.6 MPa
Moment of Inertia = b x h^3/12 = 0.0036m^4 (B=1m , H=0.35m)
Cracking Moment= 2.6 x 1000 x 0.036/ (0.35/2) = 54 kN-m/m
Checking against AASHTO 07 criteria for minimum reinforcement,
Criteria 1, 1.2 x Cracking moment = 65kN-m/m
Criteria 2, 1.33 x Flexural Moment = 103kN-m/m

Minimum reinforcement required is for 65kN-m/m which is Less than M_u (77 kN-m/m). So, design moment finally considered is 77 kN-m/m. R16-200 is provided for design moment of 77 kN-m/m.

Horizontal Reinforcement For Water face (W/F)

Here Design Moment for Water Face Tension is very low. So minimum reinforcement required for temperature and shrinkage is provided here. R16-200 is provided for horizontal reinforcement at water face.

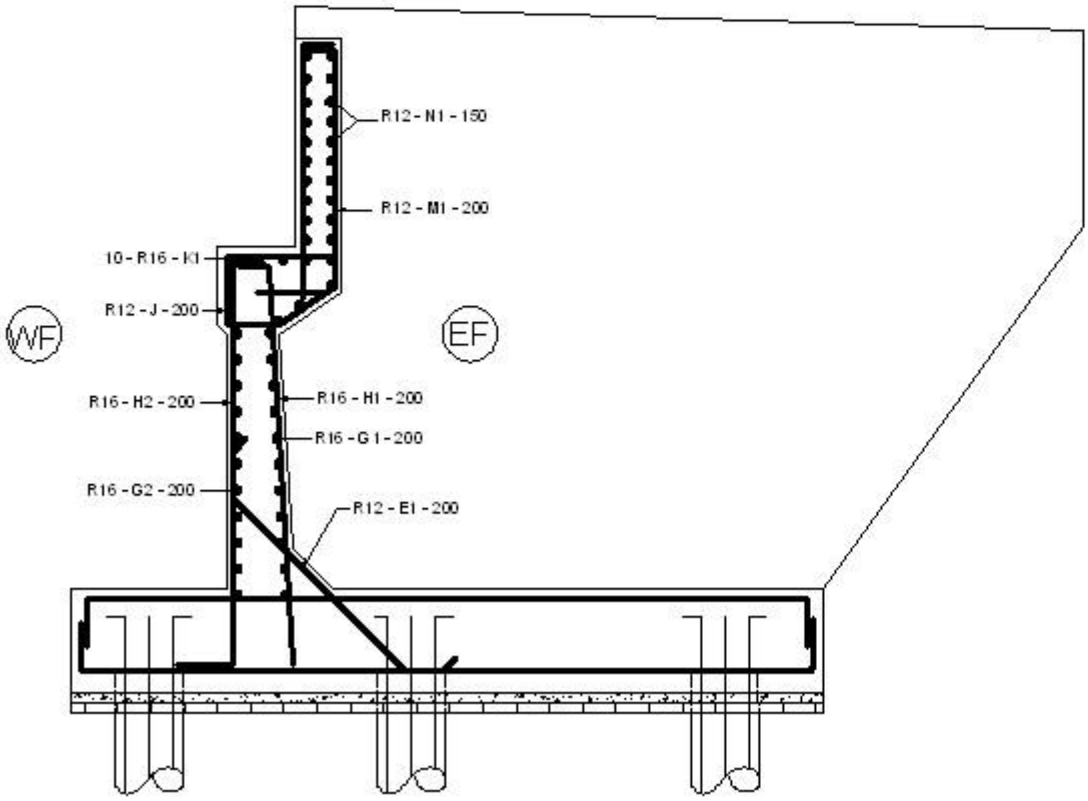


Fig 2.2.6 Reinforcement Details of abutment wall

2.3 Wing Wall Design

Wing wall is designed considering 500mm thickness at bottom to 350mm at top. Plate 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. Detailed Moment values a particular plate no at any section for both axis as M_x and M_y in table no:1, and :2 are provided in the axis are attached here with reinforcement calculations and curtailment.

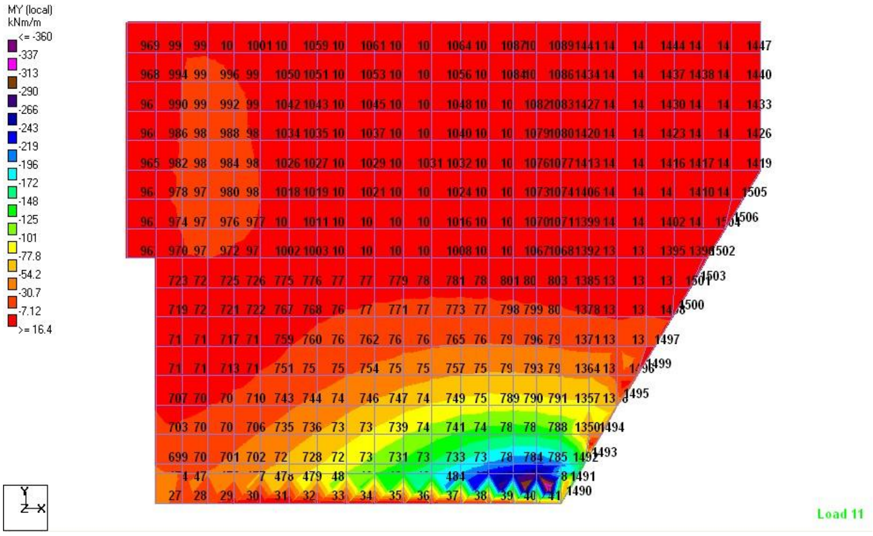


Fig 2.3.1 *Plate No of Wing Wall, M_y,kN-m*

Fig 2.3.2 *Plate orientation*

Table: Bending Moment, M_y along critical section at bottom for Vertical Reinforcement Calculation.

SL NO	PLATE NO (from downward to upward)	My (kN-m/m)
1	474	-23
2	475	-27
3	476	-34
4	477	-45
5	478	-58
6	479	-73
7	480	-90
8	481	-108
9	482	-128

SL NO	PLATE NO (from downward to upward)	My (kN-m/m)
10	483	-150
11	484	-175
12	485	-203
13	486	-228
14	487	-241
15	488	-255
16	40	-318
17	41	-360

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

It can be observed that maximum negative moment (-360 kN-m/m) is observed at plate no 41.Near the pile cap end and gradually reduces to (-23 kN-m/m) at plate no 474 near adjoining section between abutment wall and wing wall. Again from bending moment contour it appears that bending moment gradually reduces from bottom section to top. The detail calculation is provided below.

Reinforcement Design Calculation

Vertical Reinforcement For Earth face (E/F)

Design Moment M_u = 360 kN-m/m

Compressive strength of Concrete, fc′= 25 MPa

Thickness of Wing Wall = 0.5 m

Modulus of Rupture = 0.52 x √ fc′= 2.6 MPa

Moment of Inertia = b x h^3/12 = 1 x 0.5^3 /12 = $0.01m^4$

Cracking Moment= 2.6 x 1000 x 0.01/ (0.5/2) = 104 kN-m/m

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, 1.2 x Cracking moment= 125 kN-m/m

Criteria 2, 1.33 x Flexural Moment= 480 kN-m/m

Minimum reinforcement required against 125 kN-m/m which is less than the design moment M_u So, design moment finally considered is 360 kN-m/m.

By calculation R20-100 is provided for this moment 360 kN-m/m. From bending moment contour it is observed that higher concentration of flexural requirement is confined to a very limited area nearer to the end of pile cap so the reinforcement R20-100 are provided at 2 m laterally and 1.5 m vertically and half of this reinforcement is provided for entire other sections. Since the bending moment of that areas did not exceed 150k-Nm/m (from Bending Moment Contour of M_y).

Again for Bending Moment contour the design Moment for horizontal reinforcement is found 189 kN-m/m developing tensile stress at earth face following the same design calculation procedure the reinforcement requirement for horizontal tensile stress is found R16-100 mm c/c up to 1 m height from pile cap top and R16-200 for the remaining part of the wing wall. This reinforcement is provided horizontally at earth face.

Vertical Reinforcement For Water face (W/F),

Design Moment M_u = 17 kN-m/m
Compressive strength of Concrete, fc′= 25 MPa
Thickness of Wing Wall = 0.5 m
Modulus of Rupture = 0.52 x √ fc′= 2.6 MPa
Moment of Inertia = b x h^3/12 = 1 x 0.5^3 /12 = .01m^4
Cracking Moment = 2.6 x 1000 x 0.01/ (0.5/2) = 104 kN-m/m
Checking against AASHTO 07 criteria for minimum reinforcement,
Criteria 1, 1.2 x Cracking moment = 125 kN-m/m
Criteria 2, 1.33 x Flexural Moment = 23 kN-m/m

Minimum reinforcement required is for 23 kN-m/m which is greater than design Moment M_u 17 kN-m/m. But the reinforcement for this Bending Moment does not satisfy the temperature and shrinkage reinforcement. So we provide R16-200 vertically at water face for the entire wing wall.
Again for Bending Moment contour the design Moment for horizontal reinforcement is found 10 kN-m/m developing tensile stress at water face following the same design calculation procedure the reinforcement requirement for horizontal tensile stress is found much less than the temperature and shrinkage requirement.

So minimum reinforcement required is
A_{st} = 0.003bh/2
=0.003 x 1000 x 500/2 *(Ref. AASHTO 07, Article 5.6.3.6)*
=750mm^2/m

R16-200 is provided to satisfy this reinforcement at water face horizontally.

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

SL NO	PLATE NO	Mx (kN-m/m)	SL NO	PLATE NO	Mx (kN-m/m)
1	1441	10	9	1385	24
2	1434	8	10	1378	-30
3	1427	6	11	1371	37
4	1420	4	12	1364	44
5	1413	0	13	1357	72
6	1406	-4	14	1350	76
7	1399	9	15	1493	189
8	1392	-14			

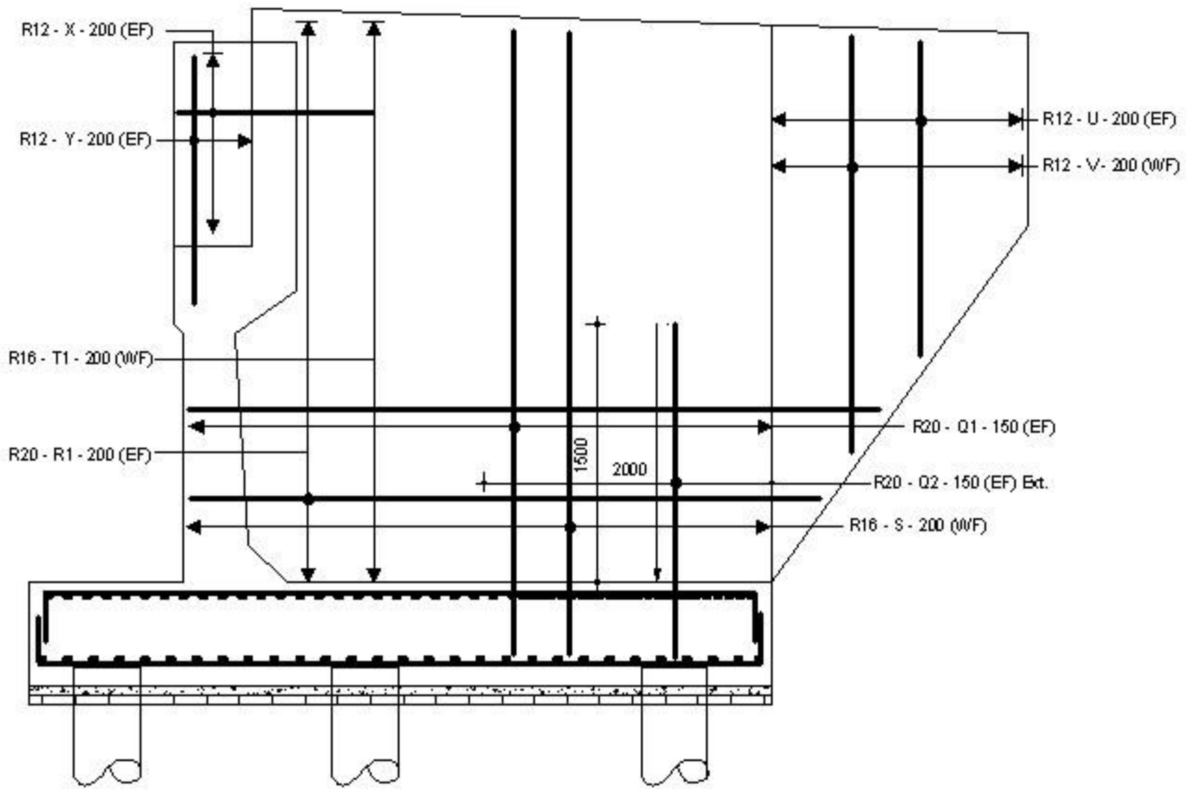
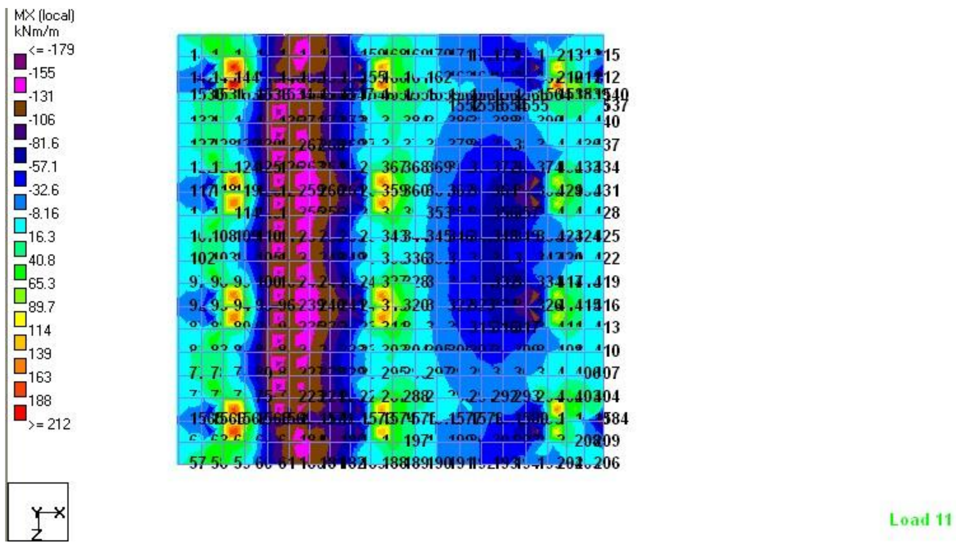
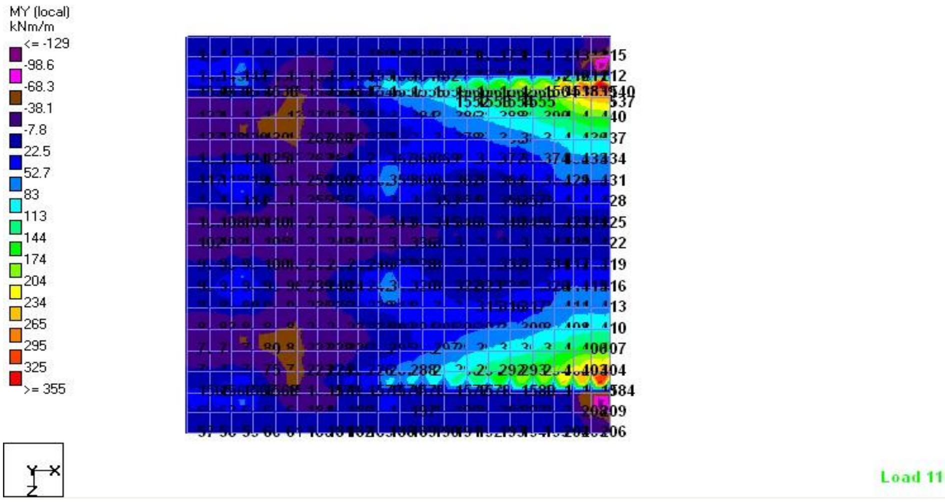


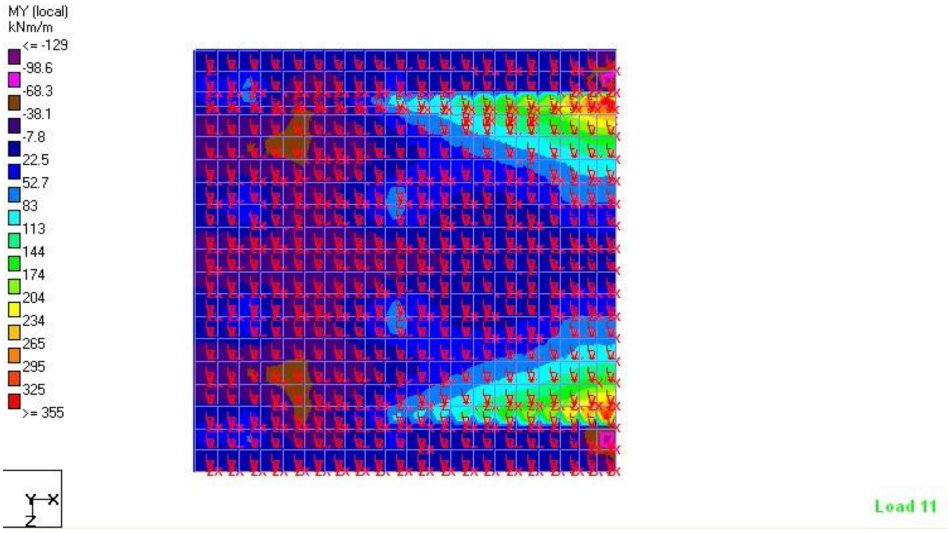
Fig 2.3.3 Reinforcement Details of Wing Wall

2.4 Pile Cap Design

The thickness of pile cap has been selected as 800 mm which has been found satisfactory after detail design calculation. Plate numbers of pile cap with orientation are provided in the following figures Moment contour maps of plates are also given.







Reinforcement Design of Pile Cap along Traffic Direction
Bottom Face Reinforcement Design

Design Moment M_u = 179 kN-m/m
Compressive strength of Concrete, fc′= 25 MPa
Thickness of Pile Cap= 0.65m (t-150 = 650 , here t=800 mm)
Modulus of Rupture = 0.52 x √ fc′= 2.6 MPa
Moment of Inertia = b x h^3/12 = 1 x 0.65^3 /12 =0.023m^4
Cracking Moment= 2.6 x 1000 x 0.023/ (0.65/2)= 184 kN-m/m
Checking against AASHTO 07 criteria for minimum reinforcement,
Criteria 1, 1.2 x Cracking moment= 221 kN-m/m
Criteria 2, 1.33 x Flexural Moment= 238 kN-m/m

So, minimum reinforcement required is for 221 kN-m/m which is greater than design moment 179 kN-m/m. So, design moment finally considered is 179 kN-m/m. And R200-200 is provided to satisfy this flexural requirement at bottom of the pile cap.

Top Face

The same reinforcement is provided as R20-200 for top face along the traffic direction. This is found adequate to satisfy the Bending Moment 212 kN-m/m.

Reinforcement Design perpendicular to Traffic Direction
Bottom face

Flexural moment of 129kN-m/m is found at the bottom face observed from STAAD.Pro moment contour. Thus, according to previous calculation R20-200 is provided as bottom face y direction reinforcement conservatively.

Top face

Flexural moment of 281kN-m/m is found at the top face observed from STAAD.Pro moment contour. Through detail calculation following the same procedure R20-100 is provided at the critical areas as observed in the moment contour and R20-200 is provided at the remaining areas of top face. The critical areas are 2m x 2 m at two corner place at the approach.

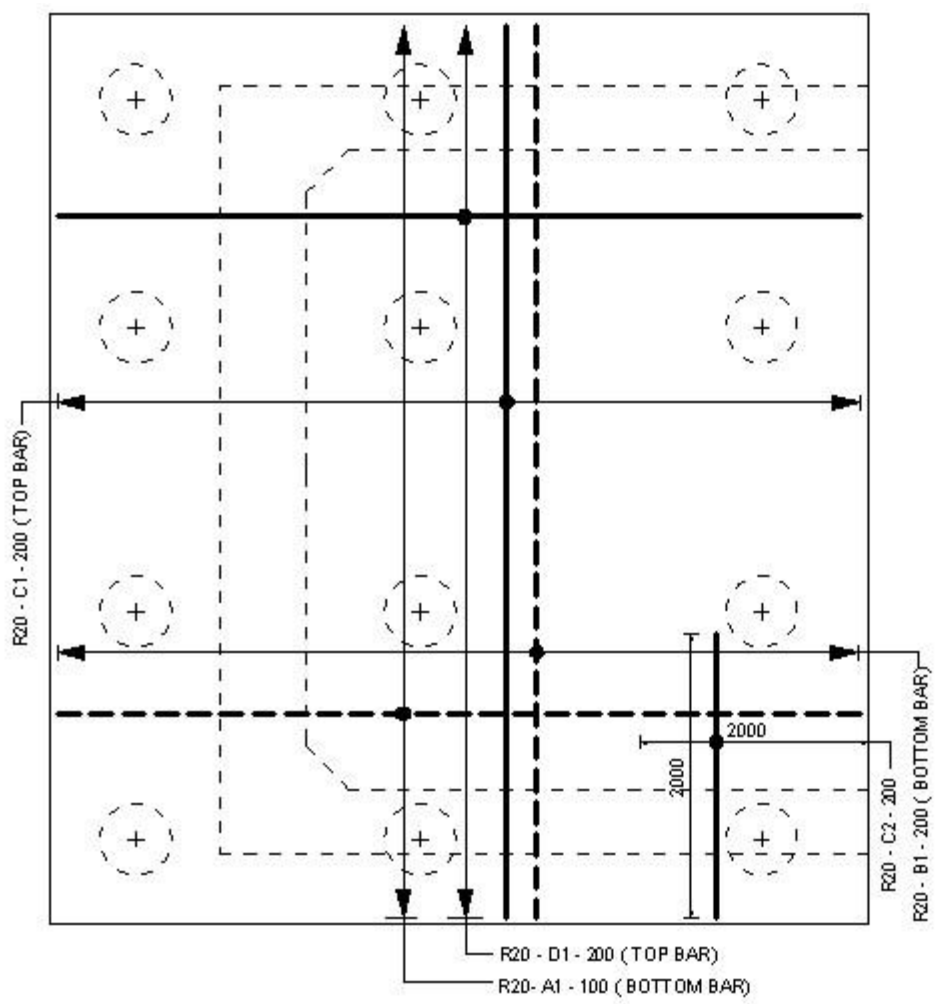


Fig 2.4.4 Reinforcement Details of Pile Cap

2.5 Structural Design of Pile

Material Properties

Concrete Cylinder Strength at 28 Days, f'c= 25 MPa
Yield Strength of reinforcing steel fy= 413 MPa

General Data

Unit Weight of Soil, **γ = 18 kN/m^3**

Maximum Horizontal Load in Pile (factored)	Px = 73 kN
Maximum Axial Load in Pile (factored)	Py = 769 kN
Moment Applied at the Pile Head (factored)	MT= 189 kN-m
Coefficient. of Subgrade Modulus	nh = 700 kN/m^3
Pile Diameter	D = 0.6 m
Modulus of Elasticity	E = 24000000 kN-m^2

Soil Data

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

NB: For loose sub merged cohesion less soil nh =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	12
Bar Diameter	25mm
Clear Cover	75mm

Analysis:

Axial Stress on Pile, σx = 2.72 N/mm^2 Allowable stress of Pile, σallowable= 6.25 N/mm^2

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 = 12
Total Steel Provided, As= n x Area of Rebar= 9 = (12 x 490=5880 mm^2)

Gross Area of Concrete, Ag = 3.14 x (D^2)/4= 282600 mm^2
% Steel provided= As/Ag= 2
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= $(EI/ήh)^{0.2}$ = 2.936m
Minimum Length of pile, L = 4xT= 12m
Level of Fixity, L'= 1.8xT = 5.29m

On the basis of these inputs from STAAD.Pro 2006 analysis and design results the assumed dia of pile and the reinforcement provided is found satisfactory for structural design requirement of pile.

Basic Load Cases

Number	Name
1	SW
2	GDL
3	GLL
4	EP
5	EL

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
10	COMBINATION SERVICE	1	SW	1.00
		2	GDL	1.00
		3	GLL	1.00
		4	EP	1.00
		5	EL	1.00
11	COMBINATION FACTORED	1	SW	1.25
		2	GDL	1.25
		3	GLL	1.75
		4	EP	1.50
		5	EL	1.35

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	1127	5:EL	**0.010**	-0.006	-0.001	0.012	-0.000	0.000	-0.000
Min X	988	11:COMBINATI	**-0.454**	-0.033	-0.030	0.457	-0.000	0.002	0.000
Max Y	466	4:EP	-0.285	**0.016**	-0.000	0.286	0.000	-0.000	0.000
Min Y	1023	11:COMBINATI	-0.432	**-0.041**	0.001	0.434	-0.000	0.000	0.000
Max Z	1275	11:COMBINATI	-0.453	0.002	**0.446**	0.635	0.002	-0.002	0.000
Min Z	1276	11:COMBINATI	-0.454	0.002	**-0.448**	0.637	-0.002	0.002	0.000
Max rX	1455	11:COMBINATI	-0.436	-0.004	0.191	0.476	**0.002**	-0.001	0.000
Min rX	1333	11:COMBINATI	-0.437	-0.004	-0.193	0.477	**-0.002**	0.001	0.000
Max rY	955	11:COMBINATI	-0.454	-0.021	-0.166	0.484	-0.001	**0.002**	0.000
Min rY	1131	11:COMBINATI	-0.453	-0.021	0.165	0.483	0.001	**-0.002**	0.000
Max rZ	6	11:COMBINATI	-0.421	-0.006	-0.001	0.421	-0.000	0.000	**0.001**
Min rZ	217	11:COMBINATI	-0.423	-0.028	0.000	0.424	0.000	0.000	**-0.000**
Max Rst	1276	11:COMBINATI	-0.454	0.002	-0.448	**0.637**	-0.002	0.002	0.000

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (kNm/m)	My (kNm/m)	Mxy (kNm/m)
Max Qx	1490	11:COMBINATI	**159.936**	73.563	-97.394	-14.388	-24.243	-404.729	-58.950	-72.046
Min Qx	1532	11:COMBINATI	**-220.441**	156.211	-28.741	1.185	8.250	250.472	76.330	-107.719
Max Qy	1589	11:COMBINATI	75.627	**167.240**	-17.364	-68.911	-8.783	-100.791	-358.111	67.840
Min Qy	1567	11:COMBINATI	-181.062	**-132.426**	-24.326	-0.880	-5.642	206.808	54.646	76.461
Max Sx	1104	11:COMBINATI	-70.480	-3.886	**124.075**	-11.037	17.978	-49.611	-1.163	-10.275
Min Sx	1507	11:COMBINATI	-158.065	73.306	**-98.580**	-14.714	24.179	-402.328	-59.699	71.908
Max Sy	1540	11:COMBINATI	-13.688	22.990	28.175	**191.487**	38.910	47.166	348.513	74.259
Min Sy	695	11:COMBINATI	-2.699	-12.739	-22.536	**-309.548**	-120.814	-16.221	-4.127	-4.858
Max Sxy	687	11:COMBINATI	2.691	-12.737	-22.539	-309.533	**120.833**	-16.217	-4.126	4.861
Min Sxy	695	11:COMBINATI	-2.699	-12.739	-22.536	-309.548	**-120.814**	-16.221	-4.127	-4.858
Max Mx	1532	11:COMBINATI	-220.441	156.211	-28.741	1.185	8.250	**250.472**	76.330	-107.719
Min Mx	1490	11:COMBINATI	159.936	73.563	-97.394	-14.388	-24.243	**-404.729**	-58.950	-72.046
Max My	1540	11:COMBINATI	-13.688	22.990	28.175	191.487	38.910	47.166	**348.513**	74.259
Min My	41	11:COMBINATI	-74.858	166.604	-19.375	-68.883	9.077	-98.907	**-358.135**	-68.023
Max Mxy	210	11:COMBINATI	-126.727	-50.030	-30.612	8.209	-3.953	142.054	19.856	**149.077**
Min Mxy	207	11:COMBINATI	-120.941	47.982	-29.837	8.654	3.779	128.487	18.792	**-145.178**