ROAD STRUCTURES MANUAL FOR SINGLE LANE BRIDGES
PART-E DESIGN EXAMPLES OF BRIDGES

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Manual on RC Girder & PC Girder Bridges

Part E- Design Examples

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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 $f_y = 413$ MPa.

![Fig 1.1 Cross section of deck slab](image)

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, \( f_{c}^{'} \) = 25MPa
- Yield strength of steel, \( f_{y} \) = 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

![Diagram of c/c Girder Spacing Loading Diagram]

**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, \( P = 72.5 \text{ kN} \), plus lane loading 9.30 kN/m 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.

\[
\text{Distribution width (for +ve moment)} = 660 + 0.55 \times S
\]
\(\text{(Ref. AASHTO 07, Art.4.6.2.1)}\)

\[
\text{Strip width (for -ve moment)} = 1220 + 0.25 \times S
\]
\(\text{(Ref. AASHTO 07, Art.4.6.2.1)}\)

Here, \( S \) = clear spacing of girder = 2050mm

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

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

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

+ve moment due to live load:

\[
\text{MLL = wheel (+) 17.6 kN-m/m (from STAAD)}
\]

\[
\text{Lane (+) 0.85 kN-m/m (from STAAD)}
\]

Total factored moment (Strength-I):

\[
M_{\text{pos}} = (1.25 \times 2.52) + (1.5 \times 0.6) + 1.75 \times 18.45 \times (1+0.33)
\]

\[
= 47 \text{ kN-m/m}
\]

1.1.2.2 Deck overhang

![Diagram of 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 = \((24 \times 17 \times 0.175 \times 0.175 \times 1.005)/24 + (3 \times 0.15 \times 0.125 \times 21.2 \times 24) / 24\)  
\[= 1.75 \text{ kN/m}.\]

**-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 = 1.113 m

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 \text{ kN/m}.

3. Bridge pedestrian load = \(3.6 \times 10^{-3} \text{ N/m}^2\) = 3.6 kN/m

(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):**

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

**Design moment:**  
+ve moment, \(M_{\text{pos}} = 47 \text{ kN-m/m}.\)  
-ve moment, \(M_{\text{neg}} = 22.45 \text{ kN-m/m}.\)
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$^2$.
- $A_S = 200.96 \times 1000/150 = 1339.73 \text{ mm}^2$.
- Lever arm factor, $a = (A_{sfy}) / (0.85f'_c \cdot b)$
  \[ = (1339.73 \times 413) / (0.85 \times 25 \times 1000) \]
  \[ = 26.00 \text{ mm} = 0.26 \text{ m} \]

Effective depth, $d = 200 - 50 - 8 = 142 \text{ mm} = 0.142 \text{ m}$.

Moment capacity, $\varphi M_n = 0.9 A_{sfy} (d-a/2)$
  \[ = 0.9 \times 1339.73 \times 413 \times (142-(26/2)) \times 10^{-6} \]
  \[ = 64.24 \text{ kN-m/m.} \]

Ultimate $M_{pos}$, 47 kN-m/m. < $\varphi M_n$ = 64.24 kN-m/m. OK

1.1.3.2 -ve reinforcement:

Here,
- $M_{neg} = 22.45 \text{ kN-m/m.}$

Ultimate $M_{neg}$, 22.45 kN-m/m. < $\varphi M_n$. 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, $f_r = 0.52 \sqrt{f'_c} = 0.52 \times \sqrt{25} = 2.6 \text{ MPa.}$

(Ref. AASHTO 07, Art.5.4.2.6)

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

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)
  \[ = (2.6 \times 0.0007 \times 10^6) /100 \]
  \[ = 18.2 \text{ kN-m} \]

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

So, $\varphi M_n > M_F$ OK

1.1.3.4 Temperature & shrinkage reinforcement:

Here,
- minimum reinforcement required is
  \[ A_{st} = 0.003bh /2 = 0.003 \times 1000 \times 200/2 = 300 \text{ mm}^2. \] (Ref. AASHTO 07, Art.5.6.3.6)

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

$A_{prov} = 113 \times 1000/200 = 565 \text{ mm}^2 > A_{st} = 300 \text{ mm}^2$
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,
\[
A_{sd} = \frac{3840}{\sqrt{S}} \leq 67\% \quad (Ref. \ AASHTO \ 2007, \ Art.9.7.3.2)
\]
Effective span of deck slab, \( S = 2500-450 = 2050 \text{ mm} = 2.05 \text{ m} \).
Required percentage of distribution reinforcement, \( A_{sd} = \frac{3840}{\sqrt{2050}} = 84.8 \% \)
So, \( A_{sd} = 67\% \) of \( A_s \) will be provided.
Here, \( A_s = R16-150 = 1340 \text{ mm}^2/\text{m} \)
Then, \( 67\% \) of \( A_s = 898 \text{ mm}^2/\text{m} \)
Using R12 for distribution bar required spacing = \( \frac{(113 \times 1000)}{898} \)
\( = 155 \text{ mm} \).
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 \( (Ref. \ AASHTO-\ 07, \ Article \ 3.6.1.2.1) \)
- Design lane load consist of 9.3 N/mm uniformly distributed in longitudinal direction \( (Ref. \ AASHTO-07, \ Article \ 3.6.1.2.1) \)
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, $f_y = 413$ 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

<table>
<thead>
<tr>
<th>Component</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>A (mm²)</th>
<th>y (mm)</th>
<th>Ay (mm³)</th>
<th>y_b (mm)</th>
<th>I (mm⁴)</th>
<th>y_bar (mm)</th>
<th>A(y_bar)² (mm⁶)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2014</td>
<td>200</td>
<td>4.028E+05</td>
<td>1900</td>
<td>7.653E+08</td>
<td></td>
<td>1.343E+09</td>
<td>646.25</td>
<td>1.682E+11</td>
</tr>
<tr>
<td>2</td>
<td>450</td>
<td>1800</td>
<td>8.1E+05</td>
<td>900</td>
<td>7.29E+08</td>
<td></td>
<td>2.187E+11</td>
<td>353.75</td>
<td>1.014E+11</td>
</tr>
<tr>
<td>3</td>
<td>539</td>
<td>150</td>
<td>4.043E+04</td>
<td>1750</td>
<td>7.075E+07</td>
<td>1253.75</td>
<td>5.053E+07</td>
<td>496.25</td>
<td>9.956E+09</td>
</tr>
<tr>
<td>4</td>
<td>150</td>
<td>150</td>
<td>1.125E+04</td>
<td>1750</td>
<td>1.969E+07</td>
<td></td>
<td>1.406E+07</td>
<td>496.25</td>
<td>2.770E+09</td>
</tr>
</tbody>
</table>

∑A = 1.264E+06  
∑Ay = 1.585E+09  
∑I = 2.201E+11  
∑A(y_bar)² = 2.823E+11

y_b = distance from centroid to the extreme bottom fiber of girder.
y_b = \frac{\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(y_bar)²
                                           = 5.024E+11 mm⁴

S_b = section modulus for the extreme bottom fiber = I / y_b = 4.007 x 10⁸ mm³

S_t = section modulus for the extreme top fiber = I / y_t = 6.732 x 10⁸ mm³
1.2.5 Live load distribution factor

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

1.2.6 Dynamic Allowance

\[ \text{IM} = 33\% \]  \hspace{1cm} \text{(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), \( M_{DL1} \)

\( M_{DL1} = 1324.81 + 244.5 + 875.10 + 144.48 = 2589 \text{ kN-m} \)

Dead load moment (Wearing Course) = 145.17 kN-m

1.2.7.3 Dead Load Shear Force Calculation

\[ R1 = [34.4 \times 23.35 \times 11.675 - 34.4 \times 0.325 \times 0.325/2 + 20.664 (23.35 + 17.512 + 11.675 + 5.8375) + 0.325 \times 34.4 \times 23.5125] / 23.35 \]

= 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
P₁ = Rear end wheel load of truck = (72.5 x DFM) = 92.8 kN
P₂ = Middle wheel load of truck = (72.5 x DFM) = 92.8 kN
P₃ = 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
R₁  = (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

![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

![Diagram of Truck Load for Shear Force Calculation]

Fig 1.2.7.6 Loading Diagram of Truck Load for Shear Force Calculation
\[ R_1 = \frac{(92.8 \times 23.35 + 92.8 \times 19.09 + 22.775 \times 14.83)}{23.35} = 183.13 \text{ kN} \]

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](image)

M = 4.7 \times 23.35^2/8 = 320.32 \text{ kN-m}

**1.2.7.9 Bending Moment from Simplified Conventional Method for Truck Load**

\[ \text{Moment at mid section} = 970.21 \text{ kN-m} \]

\[ \text{Moment with impact factor} = 970.21 \times (1 + 0.33) = 1290.39 \text{ kN-m} \]

**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

\[
\text{here, } \frac{c}{c} \text{ girder distance} = 2.5 \text{ m}
\]

Total lane load moment at mid section

\[ M = 4.7 \times 23.35^2/8 = 320.32 \text{ kN-m} \]
1.2.7.10 Summary of design moment and shear factored

**Total factored moment (Simplified Conventional Method)**

\[
= 1.25 \times 2588.97 + 1.5 \times 145.17 + 1.75 \times 1290.39 + 1.75 \times 320.32 = 6272.71 \text{ kN-m}
\]

**Total factored shear**

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

\[
= (1.25 \times 443.796) + (1.5 \times 25.56) + (1.75 \times 120.14) + (1.75 \times 55) = 899.58 \text{ kN}
\]

\[
\approx 900 \text{ kN}
\]

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 staking for maximum crack width of 0.22mm as per AASHTO ’07, Art. 5.7.3.4-1

In which,

\[
S \leq (123000 \gamma_e/ \beta_s f_{ss}) - 2d_c
\]

\[
\gamma_e = 0.5
\]

\[
d_c = \text{clear cover} + \text{bar dia}
\]

(we use T12 for shear reinforcement) = 50 + 6 = 56

\[
\beta_s = \left[ \{ 1 + d_c \{ 0.7( h- d_c)\} \} \right] = [ \{ 1 + 56/(0.7(2000-56)) \}] = 1.076
\]

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

\[
\begin{align*}
\text{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 \text{ N/mm}^2 \\
f_{sc} = 8.82 \text{ N/mm}^2, f_{sc} = 10.49/1288.53 \times 1082.53 \\
f_{ss} = 8082 \times 8.33 \quad f_{ss} = (E_s/E_c) \times f_{sc} \\
= 73.47 \text{ MPa} \quad E_c = 24 \times 10^3 \text{ MPa} \\
E_s = 200 \times 10^3 \text{ MPa} \\
E_s/E_c = 8.33
\end{align*}
\]

\[
S \leq (123 \times 10^3 \times 0.5) \div (73.47 \times 1.076) - 2 \times 56 \leq 666 \text{ mm}
\]
BS Method:

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

Concrete strain = 5.80 /24 x 10^3 = 0.00024

Crack width = 3 \( \varepsilon_{\text{min}} \times a_{\text{cr}} = 3 \times 0.00024 \times 107.35 \) [acr = \( \sqrt{100^2 + 55^2} \) = 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,
\[ \Delta_g = 5wL^4 / (384EI) \]
\[ w = \text{beam self wt.} = 19.44 \text{ kN/m} \]
\[ \Delta_g = (5 \times 19.44 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338) \]
= 0.0059 m = 5.9 mm (↓)

Deflection due to slab wt,
\[ \Delta_s = 5wL^4/ (384EI) \]
\[ w = \text{slab wt.} = 14.96 \text{ kN/m} \]
\[ \Delta_s = (5 \times 14.96 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338) \]
= 0.0045 m = 4.5 mm (↓)

Deflection due to X-girder load,
\[ \Delta_{\text{X-girder}} = PL^3 / (48EI) \]
\[ P = \text{X-girder load} = 20.664 \times 3 = 61.992 \text{ kN} \]
\[ \Delta_{\text{X-girder}} = (61.992 \times (23.35) \times 1000) / (48 \times 24 \times 10^6 \times 0.5338) \]
= 1.28 mm (↓)

Deflection due to wearing course,
\[ \Delta_{\text{wc}} = 5wL^4/ (384EI) \]
\[ w = \text{wearing course wt.} = 2.13 \text{ kN/m} \]
\[ \Delta_{\text{wc}} = (5 \times 2.13 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338) = 0.6 \text{ mm (↓)} \]

Deflection due to lane load,
\[ \Delta_{\text{LL}} = 5wL^4/ (384EI) \]
\[ w = \text{lane load} = 4.7 \text{ kN/m} \]
\[ \Delta_{\text{LL}} = (5 \times 4.7 \times (23.35)^4) / (384 \times 24 \times 10^6 \times 0.5338) \]
= 1.4 mm (↓)

Deflection due to truck & impact load,
\[ \Delta_{\text{WL}} = PL^3 / (48EI) \]
\[ P = \text{wheel load} = (92.8 \times 2 + 22.775) \times 1.33 \times \text{Dynamic allowance} \]
= 277 kN
\[ \Delta_{\text{WL}} = (277 \times (23.35)^3 \times 1000) / (48 \times 24 \times 10^6 \times 0.5338) \]
= 5.73 mm (↓)

 Allowable deflection for live load = L/800 \( (Ref.\ AASHTO\ 07\ Art.\ 2.5.2.6.2) \)
\[ = (23.35) \times 1000/800 = 29.19 \text{ mm} \]

Total live load deflection = 5.73 + 1.4 = 7.13 mm < 29.19 mm \textbf{OK}
1.2.7.13 Reinforcement calculation

![Moment Diagram of Girder](image)

**Fig 1.2.7.13.1 Moment Diagram of Girder**

**a) Flexural Moment**

Total moment, $M_u = 6272.71 \text{kN-m (Simplified Conventional Method)}$.

Using, $16 - T32 = 12862 \text{ mm}^2$

Effective depth of girder, $d = 1626 \text{ mm}$

Lever arm factor, $a = \frac{A_s f_y}{0.85 x f_c x b}$

$$= \frac{(12862 \times 413)}{(0.85 \times 25 \times 2050)}$$

$$= 122 \text{ mm}$$

Nominal moment, $\Phi M_n = 0.9 \times A_s \times f_y \times (d - a/2)$

$$= 0.9 \times 12862 \times 413 \times (16826 - 122/2) \times 10^6$$

$$= 8438 \text{kN-m}$$

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

**b) Cracking Moment**

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

$AASHTO 07 \text{ Art 5.7.3.3.2-1}$

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

$= 0 \text{ (for RC girder)}$

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

$M_{dnc} = \text{total service dead load moment acting on the monolithic or non-composite section}$

(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, $f_r$ (for normal density concrete) = $0.52 \sqrt{f_{c'}}$

Ref: $AASHTO 07 \text{ Art 5.4.2.6}$

$$= 0.52 \sqrt{25}$$

$$= 2.6 \text{ MPa}$$

Distance from N.A to the top fiber of the section, $y_t = 711.47$
Moment of inertia, \( I = 5.338 \times 10^{11} \text{ mm}^4 \)
Cracking moment, \( M_{cr} = \frac{(2.6 \times 5.338 \times 10^{11})}{711.47}/106 = 1638.48 \text{ kN-m} \)
\[ 1.2 \times M_{cr} = 1.2 \times 1638.48 = 2383 \text{ kN-m} \]
As \( \Phi M_n > M_{cr} \) flexure design is OK

c) **Surface Reinforcement**

Minimum reinforcement = 0.003bt
\[ = 0.003 \times 1000 \times 450 / 2 \]
\[ = 675 \text{ mm}^2 \]

Providing T-12,

Spacing = \( \frac{(1000 \times 113)}{675} \)
\[ = 167 \text{ mm} \]
\[ \approx 150 \text{ mm} \]

Effective depth = 1800 mm

Total no. of surface reinforcement = \( \frac{1800}{(150 \times 2)} \)
\[ = 6 \]

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
\[
V_n = V_c + V_s + V_p \quad (Ref. AASHTO 07 Art. 5.8.3.3-1)
\]
\[
V_n = 0.25 f'_c b_{dV} v_{dV} + V_p \quad (Ref. AASHTO 07 Art. 5.8.3.3-2)
\]
Here, \( V_c = 0.083\beta \sqrt{f'_c} \cdot b_{dV} \cdot v_{dV} \quad (Ref. AASHTO 07 Art. 5.8.3.3-3)
\]
\[
V_s = \frac{(A_v f_y d v \cot \theta)}{s}
\]
\( b_{dV} = \) effective web width taken as the minimum web width within the depth \( d_{dV} \).
\( d_{dV} = \) effective shear depth.
\( s = \) spacing of stirrups.
\( \theta = \) angle of inclination of diagonal compressive stresses.
\( \beta = \) 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
\[
d_{dV} = \frac{M_n}{A_s f_y} \quad (Ref. AASHTO Art 5.8.2.9)
\]
\[
= \frac{A_s f_y (d - a/2)}{A_s f_y}
\]
\( = 1738 \text{ mm} \)

\( d_{dV} \) need not to be taken to be less than the greater of 0.9\( d_e \) or 0.72\( h \) (mm)

Here, \( d = 1800 \text{ mm} \)
\( a = \frac{(A_s f_y)}{(0.85 f'_c b)} = 125 \text{ mm} \)

Now, 0.72\( h = 0.72 \times 2000 = 1440 \text{ mm} \)
\( 0.9 d_e = 1794 \times 0.9 = 1614.6 \text{ mm} \)
\( d_{dV} > \) greater of 0.72\( h \) and 0.9 \( d_e \). So, OK.

ii) Calculation of \( \beta \) & \( \theta \)

\( \theta = \) angle of inclination of diagonal compressive stresses.
\( \beta = \) factor indicating ability of diagonally cracked concrete to transmit tension.

\[
\nu_u / f'_c = \frac{0.69}{25} = 0.028
\]

here, \( \nu_u = \) total factored shear/area
\[
= \frac{(900/1333 \times 10^3) \times 1000}{= 0.675 \text{ N/mm}^2 \quad \text{from AASHTO Table-5.8.3.4.2-1}}
\]

Let, \( \varepsilon_x = 0.001 \)
\( \theta = 36.4 \)
\( \beta = 2.23 \)

Hence,
\[
V_c = 0.083\beta \sqrt{f'_c} \cdot b_{dV} \cdot v_{dV}
\]
\[
= 0.83 \times 2.23 \times \sqrt{25} \times 450 \times 1738 / 1000
\]
\( = 724 \text{ kN} \)

Using, T12-150
Vs shall be determined by,

\[ V_s = A \cdot f_v \cdot d_v \cdot \cot \theta / s \]  
\[ (Ref. \ AASHTO \ 07 \ Art \ 5.8.3.3-1) \]

\[ = (2 \times 113 \times 413 \times 1738 \times 1.37)/150 \]
\[ = 1482 \text{ kN} \]

\[ V_n = V_c + V_s + V_p \]
\[ = 724 + 1482 + 0 \]
\[ = 2206 \text{ kN} \]

\[ V_n = 0.25 \, f_v \cdot b_v \cdot d_v + V_p \]
\[ = 0.25 \times 25 \times 450 \times 1738 + 0 \]
\[ = 4888.13 \text{ kN} \]

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

So, \( V_n = 2206 \text{ kN} \)

Therefore,

\[ V_n = 2206 \text{ kN} \]
\[ V_u = 900 \text{ kN} \]

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 whether Transverse Reinforcement needed

For beam, \( V_u > 0.5 \varphi \, (V_c+V_p) \)

\( V_u = 900 \text{ kN} \)

\[ 0.5 \times \varphi \times V_c = 0.5 \times 0.9 \times 724 = 326 \text{ kN} \]

as \( V_u > V_c \) so shear reinforcement  is needed.

### iv) Checking Minimum Transverse Reinforcement

\[ A_v \geq 0.083 \sqrt{f_v \cdot b_v \cdot s / f_y} = 0.083 \sqrt{25 \times 450 \times 150 / 413} = 68 \text{ mm}^2 \]

\( (Ref. \ AASHTO \ 07, \ Art. \ 5.8.2.5) \)

### v) Maximum Spacing of Transverse Reinforcement:

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing \( S_{\text{max}} \) determined as:

If \( v_u < 0.125 \times f_c' \) then,

\[ S_{\text{max}} = 0.8d_v \leq 600 \text{ mm} \]  
\[ (Ref. \ AASHTO \ 07, \ Art. \ 5.8.2.7-1) \]

If \( v_u \geq 0.125 \times f_c' \) then,

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

\[ v_u = V_u/A = 900 \times 10^3 /1333 \times 10^3 = 0.675 \text{ N/mm}^2 \]
\[ v_u/ f_c' = 0.675 / 25 = 0.027 \]

Therefore,

\[ S_{\text{max}} = 0.8d_v \leq 600 \text{ mm} = 0.8 \times 1738 \leq 600 = 1390 \leq 600 \]

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.

![Bridge Cross Section](image)

**Fig 1.3.1.1 Bridge Cross Section**

1.3.2 Data

**Cast in place deck slab:**
- Actual thickness, \( t_s = 200 \) mm
- Concrete strength at 28 days, \( f'_c = 25 \) MPa

**Pre-cast girder:**
- Concrete strength at transfer, \( f'_{ci} = 0.75 \times 35 = 26.25 \) MPa
- Concrete strength at 28 days, \( f'_c = 35 \) MPa
- Concrete unit weight = 24 kN/m³
- 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 mm²
- No of strands in one cable = 12
No of cable = 4
Ultimate strength, $f_{pu} = 1860$ MPa
Yield strength, $f_{py} = 0.9f_{pu} = 1674$ MPa

**Stress limit for pre-stressing strands:**

(Ref. AASHTO 07, Table 5.9.3-1)

Before transfer, $f_{pbt} \leq 0.75f_{pu} = 0.75 \times 1860 = 1395$ 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, $f_y = 413$ 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, $\gamma_c = 2400$ 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, $E_{ci} = 4800\sqrt{26.25} = 24593$ MPa
Pre cast girder at service load, $E_c = 4800 \sqrt{35} = 28397.18$ MPa

![Fig 1.3.3.1.1 Non Composite Mid & End-Section](image-url)
**Table:** Showing $\sum A$, $\sum Ay$, $\sum I$, $\sum A (ybar)^2$ for non-composite mid-section of girder

<table>
<thead>
<tr>
<th>Component</th>
<th>b</th>
<th>h</th>
<th>A</th>
<th>Y</th>
<th>$AY$</th>
<th>$Y_{b}$</th>
<th>I</th>
<th>Ybar</th>
<th>$A^*(Ybar)^2$</th>
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<td>6.55E+10</td>
<td>2.66E+11</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

$= 915.1$ mm

$y_t =$ Distance from centroid to the extreme top fiber of non-composite girder

$= 884.9$ mm

$\sum I = 3.26E+11 \text{ mm}^4$

$S_b =$ Section modulus for the extreme bottom fiber $= I/ y_b = 3.56 \times 10^8 \text{ mm}^3$

$S_t =$ Section modulus for the extreme top fiber $= I/ y_t = 3.68 \times 10^8 \text{ mm}^3$

**Calculation for effective flange width:**

Effective flange width of exterior girder shall be taken as $\frac{1}{2}$ of the girder spacing plus the lesser of

1. $\frac{1}{8} \times \text{span}$

2. $6 \times t_{s} + \text{greater of } \frac{1}{2} \times \text{web thickness or } \frac{1}{4} \times \text{beam top flange width}$

3. Width of the over hang Deck

then effective flange width

**1.3.3.2 Composite Section at Mid Span**

Modular ratio between slab and girder materials, $n = \frac{E_{c} (\text{slab})}{E_{c} (\text{girder})} = 0.85$

Transformed flange width $= n \times \text{(effective flange width)} = 0.85 \times 2700 = 2295$ mm
**Chapter 1**

Fig 1.33.2.1 Composite Mid-Section

**Table:** Showing $\sum A$, $\sum Ay$, $\sum I$, $\sum A(ybar)^2$ for composite mid-section of girder

<table>
<thead>
<tr>
<th>Component</th>
<th>$b$</th>
<th>$h$</th>
<th>$A$</th>
<th>$Y$</th>
<th>$AY$</th>
<th>$Y_{bc}$</th>
<th>$I$</th>
<th>$Y_{bar}$</th>
<th>$A*(Ybar)^2$</th>
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<td>1100</td>
<td>150</td>
<td>1.65E+05</td>
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<td></td>
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</tr>
</tbody>
</table>

$A_c$ = Total area of composite girder  
= 1.377 E+06 mm$^2$

$h_c$ = 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  
= 700.07 mm

$y_{tg}$ = distance from the centroid of the composite section to the extreme top fiber of the pre-cast girder = 500.07 mm

$S_{bc}$ = Composite section modulus for the extreme bottom fiber of the pre-cast girder  
= $I_c / y_{bc} = 4.98 \times 10^8$ mm$^3$
\[ S_{tg} = \text{Composite section modulus for the top fiber of the pre-cast girder} = \frac{I_c}{y_{tg}} \]
\[ = 1.29 \times 10^9 \text{ mm}^3 \]
\[ S_{td} = \text{Composite section modulus for the extreme top fiber of the deck} \]
\[ = \frac{1}{n} x \left( \frac{I_c}{y_{tc}} \right) \]
\[ = 1.088 \times 10^9 \text{ mm}^3 \]

### 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 course 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)](image)

Here \( p \) = wheel load.

\[ \text{Load per interior girder} = P + \frac{(2700-1800)}{2700} \times P \]
\[ = 1.33 \times P \]

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)

\[ M = \frac{wl^2}{8} \]
\[ = \frac{5.7 \times 30^2}{8} \]
\[ = 641.25 \text{ kN-m} \]

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 = 85.50 kN

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

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:

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

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.

Radius of curvature, \( R_v = \frac{L^2}{8H} \)

\[
\alpha = \sqrt{\left(\alpha_v^2 + \alpha_H^2\right)} = \frac{X}{R}
\]

\[
\mu = 0.25 \\
K = 0.007
\]

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

Fig 1.3.5.1.1 Loss of prestress Due to Friction and Wedge Pull-in
**Note:** When distance \( X \) is less than \( X_a \) 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 \( X_a \) prestress force in the cable will be the ordinate of line BO.

Here,
\[
\Delta wp = 7 \text{ mm}
\]
\[
E_{ps} = 197 \times 10^6 \text{ kN/m}^2
\]
\[
A_{ps} = 98.71 \times 10^{-6} \times 12 = 1184.4 \times 10^{-6} \text{ m}^2
\]

**Table: Calculation of sag & radius of curvature**

<table>
<thead>
<tr>
<th>Cable no.</th>
<th>Vertical sag, ( H_v )</th>
<th>Horizontal sag, ( H_h )</th>
<th>Radius of curvature, ( R_v )</th>
<th>Radius of curvature, ( R_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>330</td>
<td>200</td>
<td>340.91</td>
<td>562.50</td>
</tr>
<tr>
<td>2</td>
<td>680</td>
<td>200</td>
<td>165.44</td>
<td>562.50</td>
</tr>
<tr>
<td>3</td>
<td>1030</td>
<td>0</td>
<td>109.22</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>1230</td>
<td>0</td>
<td>91.46</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Table: Calculation of wedge pull in and friction loss**

<table>
<thead>
<tr>
<th>Cable No.</th>
<th>Initial prestress force, ( P_i ) (kN)</th>
<th>( \Delta = X/R ) for ( X=1 ) (rad)</th>
<th>Loss of prestress force per unit length ( \Delta p_i ) (kN/m)</th>
<th>Distance of wedge pull-in ( X_A ) (m)</th>
<th>( \Delta = X/R ) for ( X_A ) distance (rad)</th>
<th>Cable force at ( X_A ) distance (kN)</th>
<th>Cable force at ( X = 15 ) m (kN)</th>
<th>Loss of Prestress %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1652</td>
<td>0.006</td>
<td>12.93</td>
<td>11.24</td>
<td>0.039</td>
<td>1512.59</td>
<td>0.051</td>
<td>1468.539</td>
</tr>
<tr>
<td>2</td>
<td>1652</td>
<td>0.006</td>
<td>14.11</td>
<td>10.76</td>
<td>0.068</td>
<td>1506.61</td>
<td>0.095</td>
<td>1452.816</td>
</tr>
<tr>
<td>3</td>
<td>1652</td>
<td>0.009</td>
<td>15.28</td>
<td>10.34</td>
<td>0.095</td>
<td>1500.93</td>
<td>0.137</td>
<td>1437.345</td>
</tr>
<tr>
<td>4</td>
<td>1652</td>
<td>0.011</td>
<td>16.00</td>
<td>10.10</td>
<td>0.110</td>
<td>1497.51</td>
<td>0.164</td>
<td>1427.794</td>
</tr>
</tbody>
</table>

Average loss of pre-stress = \( 12.44 \% \)

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

Elastic Shortening due to Post-Tensioning,

\[
\Delta f_{prest} = \frac{N - \frac{1}{2} E_p}{E_c} f_{egpt}
\]
\[
f_{egpt} = \frac{I_g}{A_g} + \frac{I_g^2}{E_c I_g} \left( \frac{M_{esw}}{I_g} - \frac{M_{ps}}{I_g} \right)
\]
\[
P_{pt} = A_{pt} \left( f_{ji} - \Delta f_{p,ji} - \Delta f_{p,prest} \right)
<table>
<thead>
<tr>
<th>F_{pt} (kN)</th>
<th>A_g (mm$^2$)</th>
<th>I_g (mm$^4$)</th>
<th>e_{pt} (mm)</th>
<th>M_{assembly} (kN-m)</th>
<th>f_{cpt} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6137.40</td>
<td>842100</td>
<td>3.316E+11</td>
<td>761.20</td>
<td>2315.04</td>
<td>12.70</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>N</th>
<th>E_p (MPa)</th>
<th>E_c (MPa)</th>
<th>f_{cpt} (MPa)</th>
<th>Δf_{mix3} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>197000</td>
<td>28397.18</td>
<td>12.70</td>
<td>33.03</td>
</tr>
</tbody>
</table>

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

\[
Δf_{pEB1} = \frac{E_p}{E_c} Δf_{cd1}
\]

\[
Δf_{cd1} = -\frac{M_{nc} e_{pt}}{I_g}
\]

\[
M_{nc} = M_{slab} + M_{slag} + M_{UserDC} + M_{UserBW}
\]

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

<table>
<thead>
<tr>
<th>E_p (MPa)</th>
<th>E_c (MPa)</th>
<th>M_{nc} (kN-m)</th>
<th>I_g (mm$^4$)</th>
<th>e_{pt} (mm)</th>
<th>Δf_{cd1} (MPa)</th>
<th>Δf_{mix1} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>197000</td>
<td>28397.18</td>
<td>1607.25</td>
<td>3.316E+11</td>
<td>761.20</td>
<td>-3.7</td>
<td>-25.6</td>
</tr>
</tbody>
</table>

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

\[
Δf_{pEB2} = \frac{E_p}{E_c} Δf_{cd2}
\]

\[
Δf_{cd2} = -\frac{M_{c} e_{opt}}{I_c}
\]

\[
M_{c} = M_{slag} + M_{overlay} + M_{UserDC} + M_{UserBW}
\]

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

<table>
<thead>
<tr>
<th>E_p (MPa)</th>
<th>E_c (MPa)</th>
<th>M_{c} (kN-m)</th>
<th>I_c (mm$^4$)</th>
<th>e_{opt} (mm)</th>
<th>Δf_{cd2} (MPa)</th>
<th>Δf_{mix2} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>197000</td>
<td>28397.18</td>
<td>1148.51</td>
<td>6.48E+11</td>
<td>1142.4</td>
<td>-2.02</td>
<td>-14.04</td>
</tr>
</tbody>
</table>
\[ \Delta f_{pcd1} = -(M_{nc}e_{pt})/I_g \]
\[ M_{nc} = M_{slab} + M_{diaphragm} \]
\[ e_{pt} = \text{eccentricity of prestressing force with respect to centroid of noncomposite section} \]
\[ \Delta f_{pcd2} = -(M_{c}e_{cpt})/I_c \]
\[ M_{c} = M_{barrier} + M_{w/c} \]
\[ e_{cpt} = \text{eccentricity of prestressing force with respect to centroid of composite section.} \]
\[ f_{cgpt} = \text{Sum of concrete stresses at the centre of gravity of pre-stressing tendon due the}
\[ \text{pre-stressing force after jacking and self weight of the member at the section of}
\[ \text{maximum moment}
\[ = P/A + Pe^2/I - M_Ge/I \]
\[ F_{pt} = \text{effective jacking force.} \]

### 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:

\[ k_s = 1.45 - 0.005 \left( \frac{V}{S} \right) \geq 1.0 \]
\[ k_{hc} = 1.56 - 0.008H \]
\[ k_{hs} = 2.00 - 0.014H \]
\[ k_f = \frac{37}{7 + f_a'} \]
\[ k_{td} = \left( \frac{t}{61 - 0.58f_a' + t} \right)^{-0.118} \]
\[ \psi(t, t_i) = 1.9k_s k_{hc} k_f k_{td} t_i^{-0.118} \]
\[ \varepsilon_{sh} = k_s k_{hs} k_f k_{td} \times 0.48 \times 10^{-3} \]

\( k_s \) = 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.

ti = 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)

<table>
<thead>
<tr>
<th>V/S Girder (mm)</th>
<th>kvs Girder</th>
<th>V/S Deck (mm)</th>
<th>kvs Deck</th>
<th>H</th>
<th>khc</th>
<th>khs</th>
</tr>
</thead>
<tbody>
<tr>
<td>135</td>
<td>0.775</td>
<td>93</td>
<td>0.985</td>
<td>70</td>
<td>1</td>
<td>1.02</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>f'ci (MPa)</th>
<th>f'c (MPa)</th>
<th>k_l Girder</th>
<th>k_l Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.25</td>
<td>35</td>
<td>1.11278</td>
<td>0.88095</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ti (day)</th>
<th>tpt (day)</th>
<th>td (day)</th>
<th>tf (day)</th>
<th>ktd t=tpt</th>
<th>ktd t=td</th>
<th>ktd t=tf</th>
<th>ktd t=tf - td f'=f'_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>7</td>
<td>60</td>
<td>25000</td>
<td>0.1326</td>
<td>0.567242</td>
<td>0.998172</td>
<td>0.998371</td>
</tr>
</tbody>
</table>

ti = Starting time of construction
tpt = time counting from start time to post tensioning
td = time counting from start time to deck placement
tf = time counting from start time to final time (service stage)

<table>
<thead>
<tr>
<th>ψb(tpt,ti)</th>
<th>ψb(td,ti)</th>
<th>ψb(tf,ti)</th>
<th>ψb(tpt,td)</th>
<th>ψb(ti,tpt)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.318</td>
<td>1.359</td>
<td>2.391</td>
<td>1.041</td>
<td>2.074</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>εbibX 1000</th>
<th>εbidX 1000</th>
<th>εbilX 1000</th>
<th>εedilX 1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0560</td>
<td>0.240</td>
<td>0.421</td>
<td>0.424</td>
</tr>
</tbody>
</table>
a) Loss due to Shrinkage of Girder Concrete:

\[
\Delta f_{pSRpt} = 0
\]

\[
\Delta f_{pSRpt} = \varepsilon_{bpg} E_p K_{id}
\]

\[
\Delta f_{pSRpt} = \varepsilon_{bpg} E_p K_{df}
\]

\[
\varepsilon_{bpt} = \varepsilon_{bh} \left( t = t_p \right)
\]

\[
\varepsilon_{bid} = \varepsilon_{bh} \left( t = t_d \right)
\]

\[
\varepsilon_{bdf} = \varepsilon_{bh} \left( t = t_f \right)
\]

\[
\varepsilon_{bpt} - \varepsilon_{bid} - \varepsilon_{bpt}
\]

\[
\varepsilon_{bdf} = \varepsilon_{bdf} - \varepsilon_{bid}
\]

\[
K_{id} = \frac{1}{1 + \frac{E_p}{E_c} \frac{A_{pt}}{A_c} \left( 1 + \frac{A_{pg}^2}{I_g} \right) \left[ 1 + 0.7W \left( t_f, t_i \right) \right]}
\]

\[
K_{df} = \frac{1}{1 + \frac{E_p}{E_c} \frac{A_{pt}}{A_c} \left( 1 + \frac{A_{pg}^2}{I_g} \right) \left[ 1 + 0.7W \left( t_f, t_i \right) \right]}
\]

<table>
<thead>
<tr>
<th>$E_p$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$A_{pt}$ (mm$^2$)</th>
<th>$A_g$ (mm$^2$)</th>
<th>$I_g$ (mm$^4$)</th>
<th>$e_{pt}$ (mm)</th>
<th>$\psi_b(t_i, t_l)$</th>
<th>$K_{id}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>197000</td>
<td>24593</td>
<td>4737.6</td>
<td>842100</td>
<td>3.32E+11</td>
<td>761.2031429</td>
<td>2.391</td>
<td>0.771</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$E_p$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$A_{pt}$ (mm$^2$)</th>
<th>$A_g$ (mm$^2$)</th>
<th>$I_g$ (mm$^4$)</th>
<th>$e_{pt}$ (mm)</th>
<th>$\psi_b(t_i, t_l)$</th>
<th>$K_{df}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>197000</td>
<td>28397.18</td>
<td>4737.6</td>
<td>1.38E+06</td>
<td>6.48E+11</td>
<td>1142.4</td>
<td>2.391</td>
<td>0.806</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\varepsilon_{bpt} \times 1000$</th>
<th>$\varepsilon_{bid} \times 1000$</th>
<th>$\varepsilon_{bif} \times 1000$</th>
<th>$K_{id}$</th>
<th>$K_{df}$</th>
<th>$\Delta f_{pSRptd}$ (MPa)</th>
<th>$\Delta f_{pSD}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.056004508</td>
<td>0.23950876</td>
<td>0.421462288</td>
<td>0.771</td>
<td>0.806</td>
<td>27.85</td>
<td>28.89</td>
</tr>
</tbody>
</table>

b) Loss due to Creep of Girder Concrete:

\[
\Delta f_{pCpt} = 0
\]

\[
\Delta f_{pCpt} = \frac{E_p}{E_d} \int_{c_{gpt}} \left[ \psi_b \left( t_d, t_i \right) - \psi_b \left( t_{pt}, t_i \right) \right] K_{id}
\]

\[
\Delta f_{pCpt} = \frac{E_p}{E_d} \int_{c_{gpt}} \left[ \psi_b \left( t_d, t_i \right) - \psi_b \left( t_{pt}, t_i \right) \right] K_{df} + \frac{E_p}{E_c} \left( \Delta f_{c1} + \Delta f_{c2} \right) \psi_b \left( t_f, t_d \right) K_{df} \geq 0.0
c) Loss due to Relaxation of Tendon:

\[
\Delta f_{\text{pr1}} = \frac{f_{\text{pt}}}{K_L} \left( \frac{f_{\text{pt}}}{f_{\text{py}}} - 0.55 \right)
\]

\[f_{\text{pt}} = f_{\text{pj}} - \Delta f_{\text{pF}} - \Delta f_{\text{pES3}} - \Delta f_{\text{pA}}\]

\[\Delta f_{\text{pr1,ptd}} = \left( \frac{t_d - t_{\text{pt}}}{t_d - t_i} \right) \Delta f_{\text{pr1}}\]

\[\Delta f_{\text{pr2}} = \Delta f_{\text{pr1}}\]

<table>
<thead>
<tr>
<th>(f_{\text{pj}}) (MPa)</th>
<th>(\Delta f_{\text{pF}}) mid-span (MPa)</th>
<th>(\Delta f_{\text{pA}}) mid-span (MPa)</th>
<th>(\Delta f_{\text{pES3}}) (MPa)</th>
<th>(f_{\text{pt}}) (MPa)</th>
<th>(f_{\text{py}}) (MPa)</th>
<th>(K_L)</th>
<th>(\Delta f_{\text{pr1}}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1395</td>
<td>57.49</td>
<td>0</td>
<td>33.03</td>
<td>1304.48</td>
<td>1674</td>
<td>30</td>
<td>9.969</td>
</tr>
</tbody>
</table>

\(K_c = 30\) for low relaxation strand
\(f_{\text{pt}} = \) stress in prestressing strands immediately after transfer, taken not less than \(0.55f_{\text{py}}\).

<table>
<thead>
<tr>
<th>(t_i) (day)</th>
<th>(t_{\text{pt}}) (day)</th>
<th>(t_d) (day)</th>
<th>(\Delta f_{\text{pr1,ptd}}) (MPa)</th>
<th>(\Delta f_{\text{pr2}}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>7</td>
<td>60</td>
<td>8.81</td>
<td>9.969</td>
</tr>
</tbody>
</table>

d) Loss due to Shrinkage of Deck:

\[
\Delta f_{\text{pSS}} = \frac{E_p}{E_c} \Delta f_{\text{cdy}} \frac{K_{d1}}{1 + \nu_{\text{dy}}(t_f, t_d)}
\]

\[
\Delta f_{\text{cdy}} = \frac{a_{\text{cd}} A_d E_{\text{cd}}}{1 + \nu_{\text{dy}}(t_f, t_d)} \left( \frac{1}{A_c} + \frac{a_{\text{pt}} E_{\text{cd}}}{I_i} \right)
\]

\[a_{\text{cd}} = a_{\text{dy}} = t_f - t_d\]

<table>
<thead>
<tr>
<th>(\varepsilon_{\text{df}} \times 1000)</th>
<th>(A_d) (Deck area per Girder) (mm²)</th>
<th>(E_{\text{cd}}) (MPa)</th>
<th>(\psi_d(t_i, t_d))</th>
<th>(A_c) (mm²)</th>
<th>(l_c) (mm)</th>
<th>(\varepsilon_{\text{cpt}}) (mm)</th>
<th>(\varepsilon_d) (mm)</th>
<th>(\Delta f_{\text{df}}) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.424</td>
<td>535000</td>
<td>24000</td>
<td>1.032</td>
<td>1.38E+06</td>
<td>6.48E+11</td>
<td>1142.434585</td>
<td>-600.07</td>
<td>-0.887</td>
</tr>
</tbody>
</table>
\( \Delta f_{cd} \) = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete.

\( \varepsilon_{def} \) = 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:**

<table>
<thead>
<tr>
<th>Elastic Shortening/Gain</th>
<th>MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial ( \Delta f_{pES3} )</td>
<td>33.03</td>
</tr>
<tr>
<td>Due to Deck Placement ( \Delta f_{pED1} )</td>
<td>-25.59</td>
</tr>
<tr>
<td>Due to Superimposed Dead Loads ( \Delta f_{pED2} )</td>
<td>-14.04</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shrinkage of Girder Concrete</th>
<th>MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT to Deck Placement ( \Delta f_{pSRptd} )</td>
<td>27.85</td>
</tr>
<tr>
<td>Deck to Final ( \Delta f_{pSD} )</td>
<td>28.89</td>
</tr>
<tr>
<td>Due to Shrinkage of Deck ( \Delta f_{pSS} )</td>
<td>-10.082</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Creep of Girder Concrete</th>
<th>MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT to Deck Placement ( \Delta f_{pCRptd} )</td>
<td>81.602</td>
</tr>
<tr>
<td>Deck to Final ( \Delta f_{pCD} )</td>
<td>51.660</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Relaxation of Tendons</th>
<th>MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT to Deck Placement ( \Delta f_{pR1ptd} )</td>
<td>8.81</td>
</tr>
<tr>
<td>Deck to Final ( \Delta f_{pR2} )</td>
<td>9.969</td>
</tr>
</tbody>
</table>

**Schedule of Stress for Girder (at mid-section)**

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Description</th>
<th>Axial Force, kN</th>
<th>Moment, kN-m</th>
<th>Girder Bottom</th>
<th>Girder Top</th>
<th>Deck top</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Moment due to Selfweight</td>
<td>2315.04</td>
<td>-6.41</td>
<td>6.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Axial force due to Prestressing</td>
<td>6608.95</td>
<td>7.85</td>
<td>7.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Moment due to Prestressing</td>
<td>5031</td>
<td>13.94</td>
<td>-13.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Friction Loss</td>
<td>12.44 %</td>
<td>-2.71</td>
<td>0.69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Elastic Shortening loss</td>
<td>2.37 %</td>
<td>-0.52</td>
<td>0.13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Wedge pull-in loss</td>
<td>0.00 %</td>
<td>0.00</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Check 1**

| Allow. Compression, 0.6f'ci | 15.75 |
| Allowable tension, 0.25(f'ci)^{1/2} | -1.28 |

<table>
<thead>
<tr>
<th>Stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.146</td>
</tr>
<tr>
<td>1.449</td>
</tr>
<tr>
<td>Stage 2</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>11</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stage 3</th>
<th>Service condition: i. Composite section; ii. Full concrete strength &amp; deck</th>
<th>Stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>Moment due to other dead load (barrier, overlay etc.)</td>
<td>1148.51</td>
</tr>
<tr>
<td>13</td>
<td>Elastic gain due to other dead load</td>
<td>-1.01%</td>
</tr>
<tr>
<td>14</td>
<td>Live load moment</td>
<td>2375.84</td>
</tr>
<tr>
<td>15</td>
<td>Final Shrinkage loss of girder concrete</td>
<td>2.07%</td>
</tr>
<tr>
<td>16</td>
<td>Final creep loss of girder concrete</td>
<td>3.70%</td>
</tr>
<tr>
<td>17</td>
<td>Final Relaxation loss of prestressing steel</td>
<td>0.71%</td>
</tr>
<tr>
<td>18</td>
<td>Shrinkage of deck</td>
<td>-0.72%</td>
</tr>
<tr>
<td></td>
<td>Check 3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Allow. Compression, 0.60f'c with LL</td>
<td>21.00</td>
</tr>
<tr>
<td></td>
<td>Allow. Compression, 0.45f'c without LL</td>
<td>15.75</td>
</tr>
<tr>
<td></td>
<td>Allowable tension, 0.5(f'c)^0.5</td>
<td>-2.96</td>
</tr>
</tbody>
</table>

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

### 1.3.6 Calculation of Moment Capacity:

Average stress in pre-stressing steel when $f_{pe} \geq 0.5 f_{pu}$

$$f_{ps} = f_{pu}(1 - K c/d_p)$$  
(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)
  - $f_{py}$ = 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
  - $d_p = h - y_{bs} = 2000 - 157 = 1843$ mm

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

- $c$ = distance between the neutral axis and the compression face
Assuming rectangular behavior:

c = \frac{(A_{ps} f_{pu} + A_{s} f_y - A_{s}' f_{y}')}{(0.85 f_c' b + k A_{ps} f_{pu}/d_p)}

= \frac{(12 \times 98.7 \times 4 \times 1860 + 0 - 0)/(0.85 \times 25 \times 0.85 \times 2675) + (0.28 \times 12 \times 98.7 \times 4 \times 1860/1843)}{1881936/(48317.18 + 1338.76)} = 177.45 < h_{fmm}

= 177.45 < 200 \text{ mm} \quad \text{ok}

a = \beta_1 c \times 0.85 \times 177.45 = 150.84 \text{ mm}

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

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

M_r = \Phi M_n \quad \text{where } M_n = \text{Nominal resisting moment}

\Phi = 1 \quad (Ref. AASHTO 07, Art.5.5.4.2)

M_n = A_{ps} f_{ps}(d_p-a/2) + A_{s} f_y(d_s-a/2) + A_{s}' f_{y}'(d_s'-a/2) + 0.85 f_{c}'(b-b_w)h_{f}(a/2 - h_{f}/2)

= (12 \times 4 \times 98.7) \times 1806 * (1843 - 161.09/2) + 0 + 0 + 0.85 \times 35(2675 - 2675) \times 200 *

(161.09/2 - 200/2)

= 15079 \text{ kN-m}

M_r > M_0 \quad \text{OK}

1.3.7 Calculation of Non-prestressing Reinforcement:

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

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

Providing T-10

Spacing = 1000 \times 78.5/420 = 186 \text{ mm}

Total no. of surface bar needed = 1015/186x2 = 11

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

The nominal shear resistance V_n shall be determined as lesser of

\[ V_n = V_c + V_s + V_p \quad ...........(1) \]

\[ V_n = 0.25 f_{c}' b_{v} \quad d_{v} + V_p \quad ........(2) \]

Where, \( V_c = 0.083 \beta \sqrt{f_{c}' b_{v} d_{v}} \quad b_v \quad = \text{ effective web width as the minimum web width within The depth } dv \]

= 280 \text{ mm}

\[ d_{v} \quad = \text{ effective shear depth taken as the distance measured perpendicular to neutral axis between the resultant of the tensile and compressive forces due to flexure it need not be taken to be lesser than the greater of 0.9 \text{ de or} 0.72h(\text{mm})] \]
(i) \( d_v = \frac{M_v}{A_f f_y} = A_f f_y (d-a/2)/A_f f_y = (d-a/2) = (1843-150.84/2) = 1767.58 \text{ mm} \)

(ii) \( 0.72h = 0.72 \times 2000 = 1440 \text{ mm} \)

(iii) \( 0.9de = 0.9 \times 1843 = 1658.7 \text{ mm} \)

Hence \( d_v = 1767.58 \text{ mm} \)

1.3.8.1 Calculation of \( \beta \) & \( \Theta \):

Here, \( \beta \) = factor indicating ability of diagonally cracked concrete to transmit tension as specified in AASHTO ‘07 Article 5.8.3.4

\( \theta \) = Angle of inclination of diagonal compressive stresses as determined in AASHTO ‘07 , Article 5.8.3.4

\( V_u = \) Total factored shear / Area

\( = \frac{[1260 \times 10^3 / 1.377 \times 10^6]}{1} \times 1/1000 \)

\( = 9.1 \times 10^{-4} \)

\( V_u/f_c = 9.1 \times 10^{-4}/35=2.61 \times 10^{-5} \)

Referred from AASHTO 07 Table: 5.8.3.4.2

\( \theta = 36.4, \beta=2.23 \)

\( \therefore V_c = 0.083\beta\sqrt{f_c} b_v d_v \) \( \text{(Ref. AASHTO 07, Art. 5.8.3.3)} \)

\( = 0.083 \times 2.23 \times \sqrt{35} \times 280 \times 1767.58 \)

\( = 541943.59 \text{ N} \)

\( \therefore V_s = A_v f_d d_v \cot \theta / S \)

Here, \( S = 125 \text{ mm C/C} \)

\( A_v = 113 \times 2=226 \text{ mm}^2 \)

\( V_s = 226 \times 420 \times 1767.58 \times 1.36/125 \)

\( = 1825432.186 \text{ N} \)

1.3.8.2 Calculation of Prestress Force Component:

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

Effective prestress force of Cable No.2 \( P_2 =1652-2*(1652-1506.61) = 1361.22 \text{kN} \)

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

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

\( V_{P1} = 1373.18 \times .052 = 71.5 \text{kN} \) \( \theta_1 = 2.98 \)

\( V_{P2} = 1361.22 \times .095 = 129.32 \text{ kN} \) \( \theta_2 = 5.47 \)

\( V_{P3} = 1349.86 \times .138 = 186.28 \text{ kN} \) \( \theta_3 = 7.95 \)

\( V_{P4} = 1343.02 \times .165 = 221.59 \text{ kN} \) \( \theta_4 = 9.49 \)

Total \( V_P = 608.69 \text{ kN} \)

\( V_n = 2367.98 \text{ kN} \)

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

\( = 4939.26 \text{ kN} \)

Hence \( V_n = 2367.98 \text{ kN}>1260 \text{ kN} \), Hence OK.
1.3.9 Calculation of Deflection:

Uniformly distributed load due to prestressing force at transfer,

c.g of cables at end \( = 975 \text{ mm} \)
c.g of cables at mid \( = 157.5 \text{ mm} \)

\[ w = \frac{8Fh}{L^2} \]

\[ = 8 \times 1652 \times 4 \times 0.817/30^2 \quad \text{Here, } h = 817.5 \text{ mm} \]

\[ = 48.03 \text{ kN/m} \]

Deflection due to prestressing force at transfer,

\[ \Delta_P = \frac{5wL^4}{384EI} \]

\( E_{ci} = \) modulus of elasticity of concrete at transfer \( = 24.59 \times 10^6 \text{ kN/m}^2 \)

\( I = \) moment of inertia of non-composite girder \( = 0.332 \text{ m}^4 \)

\( \Delta_P = 62 \text{ mm} \)

Net deflection due to pretress \( = 62 - \left(0.04 \times 1652 \times 4 \times 30^2 / 8 \times 24.59 \times 10^6 \times 0.403 \right) \times \frac{1000}{8} \)

\( = 59 \text{ (↑)} \)

Deflection due to beam self wt,

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

\( w = \) beam self wt. \( = 27 \text{ kN/m} \)

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

\( = 34.88 \text{ mm (↓)} \)

Deflection after transfer \( = 59 - 34.88 = 24.12 \text{ mm (↑)} \)

Deflection due to X-girder,

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

Deflection due to slab wt,

\( \Delta_s = \frac{5wL^4}{(384E_{ci}I)} \)

\( w = \) slab wt. \( = 12.84 \text{ kN/m} \)

\( \Delta_s = \left(5 \times 12.84 \times (30)^4 \right) / (384 \times 28.39 \times 10^6 \times 0.648) = 7.36 \text{ mm (↓)} \)

Deflection due to wearing course,

\( \Delta_{wc} = \frac{5wL^4}{(384E_{ci}I)} \)

\( w = \) wearing course wt. \( = 3.4 \text{ kN/m} \)

\( \Delta_{wc} = (5 \times 3.4 \times (30)^4 ) / (384 \times 28.39 \times 10^6 \times 0.648) = 1.94 \text{ mm (↓)} \)

Deflection due to lane load,

\( \Delta_{ll} = \frac{5wL^4}{(384E_{ci}I)} \)

\( w = \) lane load \( = 5.7 \text{ kN/m} \)

\( \Delta_{ll} = (5 \times 5.7 \times (30)^4 ) / (384 \times 28.39 \times 10^6 \times 0.648) = 3.26 \text{ mm (↓)} \)

Deflection due to truck & impact load,

\( \Delta_{WL} = \frac{PL^3}{(48E_{ci}I)} \)
P = wheel load = (145x2+35) x DFM x Dynamic allowance
= 325x0.64x1.33 = 276.64 kN

\[ \Delta_{WL} = \frac{(276.64 \times (30)^3 \times 1000)}{(48 \times 28.39 \times 10^6 \times .648)} = 8.45 \text{ mm (↓)} \]

Allowable deflection for live load = \( \frac{L}{800} \) (Ref. AASHTO 07 Art. 2.5.2.6.2)
= (30) \times 1000/800 = 37.5 mm

Total live load deflection = 11.71 mm < 37.5 mm, Hence OK.

1.3.10 Design of End block

Taking bursting force = 0.25\( \sum \)Pu(1-a/h)+0.5\( \sum \)(Pu sin\( \alpha \))
\( \text{ (Ref. AASHTO 07, Art.5.10.9.6) } \)

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 = 925 mm
\( \alpha \) = 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 = 478.52 kN

Yield stress of steel, \( f_y \) = 420 Mpa
Allowable stress, \( f_s \) = 207.5 Mpa

\( A_t = \frac{478.52 \times 10^3}{207.5} = 2306.12 \text{ mm}^2 \) (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](Web Copy)
### Table: Cable Profile Ordinates.

<table>
<thead>
<tr>
<th>Cable No.</th>
<th>Cable Position</th>
<th>Cable</th>
<th>DISTANCE 'X' FROM END TOWARDS THE C/L IN mm</th>
<th>Emergence Angle, Φ</th>
<th>Elongation at each end excluding grip, Δ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>14850</td>
<td>13850</td>
<td>13000</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>End</td>
<td>Mid</td>
<td>Sag. a</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td>450</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>800</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>1150</td>
<td>0</td>
<td>120</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>1500</td>
<td>0</td>
<td>270</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Node</th>
<th>FY, kN</th>
<th>Node</th>
<th>FY, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>449.234</td>
<td>19</td>
<td>482.353</td>
</tr>
<tr>
<td>14</td>
<td>565.222</td>
<td>20</td>
<td>505.617</td>
</tr>
<tr>
<td>15</td>
<td>345.136</td>
<td>21</td>
<td>160.038</td>
</tr>
<tr>
<td>16</td>
<td>482.538</td>
<td>22</td>
<td>450.261</td>
</tr>
<tr>
<td>17</td>
<td>505.970</td>
<td>23</td>
<td>564.811</td>
</tr>
<tr>
<td>18</td>
<td>160.246</td>
<td>24</td>
<td>344.830</td>
</tr>
</tbody>
</table>

2.2 Abutment Wall Design

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.
From the plate stress results, maximum flexural moment, $M_y$ 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 $M_u = 89 \text{kN-m/m}$

Compressive strength of Concrete, $f_c' = 25 \text{ MPa}$

Thickness of abutment wall = 0.5 m

Modulus of Rupture $= 0.52 \times \sqrt{f_c'} = 2.6 \text{ MPa}$

Moment of Inertia $= \frac{b \times h}{12} = 0.01 \text{ m}^4$ (B=1m , H=0.5m)

Cracking Moment $= 2.6 \times 1000 \times 0.01/ (0.5/2) = 104 \text{ kN-m/m}$

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, $1.2 \times $ Cracking moment $= 125 \text{kN-m/m}$

Criteria 2, $1.33 \times $ Flexural Moment $M_u = 119 \text{kN-m/m}$

Minimum reinforcement required is for 119 kN-m/m which is greater than $M_u = 89 \text{kN-m/m}$. 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 $M_u = 14 \text{kN-m/m}$

Compressive strength of Concrete, $f_c' = 25 \text{ MPa}$

Thickness of Abutment Wall = 0.5 m

Modulus of Rupture $= 0.52 \times \sqrt{f_c'} = 2.6 \text{ MPa}$

Moment of Inertia $= \frac{b \times h}{12} = 0.01 \text{ m}^4$ (B=1m , H=0.5m)

Cracking Moment $= 2.6 \times 1000 \times 0.01/ (0.5/2) = 104 \text{ kN-m/m}$

Checking against AASHTO 07 criteria for minimum reinforcement,

Criteria 1, $1.2 \times $ Cracking moment $= 125 \text{kN-m/m}$

Criteria 2, $1.33 \times $ Flexural Moment $M_u = 19 \text{kN-m/m}$

Minimum reinforcement required is for 19kN-m/m which is greater than $M_u = 14 \text{kN-m/m}$. 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

It is observed 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 $M_u = 77 \text{kN-m/m}$
- Compressive strength of Concrete, $f_c' = 25 \text{ MPa}$
- Thickness of Wall = 0.35 m
- Modulus of Rupture = $0.52 \times \sqrt{f_c'} = 2.6 \text{ MPa}$
- Moment of Inertia $= b \times h^{3}/12 = 0.0036 m^4$ (B=1m, H=0.35m)
- Cracking Moment $= 2.6 \times 1000 \times 0.036/ (0.35/2) = 54 \text{kN-m/m}$
- Checking against AASHTO 07 criteria for minimum reinforcement,
  - Criteria 1, $1.2 \times$ Cracking moment $= 65\text{kN-m/m}$
  - Criteria 2, $1.33 \times$ Flexural Moment $= 103\text{kN-m/m}$

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.

*Fig 2.2.5 Bending Moment contour of abutment wall (with plate numbers,) $M_x, \text{kN-m}$*
**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.

![Reinforcement Details of abutment wall](image)

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

<table>
<thead>
<tr>
<th>SL NO</th>
<th>PLATE NO (from downward to upward)</th>
<th>$M_y$ (kN-m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>474</td>
<td>-23</td>
</tr>
<tr>
<td>2</td>
<td>475</td>
<td>-27</td>
</tr>
<tr>
<td>3</td>
<td>476</td>
<td>-34</td>
</tr>
<tr>
<td>4</td>
<td>477</td>
<td>-45</td>
</tr>
<tr>
<td>5</td>
<td>478</td>
<td>-58</td>
</tr>
<tr>
<td>6</td>
<td>479</td>
<td>-73</td>
</tr>
<tr>
<td>7</td>
<td>480</td>
<td>-90</td>
</tr>
<tr>
<td>8</td>
<td>481</td>
<td>-108</td>
</tr>
<tr>
<td>9</td>
<td>482</td>
<td>-128</td>
</tr>
</tbody>
</table>
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, $f_c' = 25$ MPa
- Thickness of Wing Wall = 0.5 m
- Modulus of Rupture = $0.52 \times \sqrt{f_c'} = 2.6$ MPa
- Moment of Inertia $= b \times h^3/12 = 1 \times 0.5^3 /12 = 0.01 m^4$
- Cracking Moment $= 2.6 \times 1000 \times 0.01 / (0.5/2) = 104$ 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.

<table>
<thead>
<tr>
<th>SL NO</th>
<th>PLATE NO (from downward to upward)</th>
<th>$M_y$ (kN-m/m)</th>
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<tbody>
<tr>
<td>10</td>
<td>483</td>
<td>-150</td>
</tr>
<tr>
<td>11</td>
<td>484</td>
<td>-175</td>
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<td>-228</td>
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<td>14</td>
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<td>-241</td>
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<td>15</td>
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<td>16</td>
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<td>-318</td>
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<tr>
<td>17</td>
<td>41</td>
<td>-360</td>
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</table>
**Vertical Reinforcement For Water face (W/F),**

- Design Moment $M_u = 17$ kN-m/m
- Compressive strength of Concrete, $f_c' = 25$ MPa
- Thickness of Wing Wall = 0.5 m
- Modulus of Rupture $= 0.52 \times \sqrt{f_c'} = 2.6$ MPa
- Moment of Inertia $= b \times h^3/12 = 1 \times 0.5^3/12 = 0.01 m^4$
- Cracking Moment $= 2.6 \times 1000 \times 0.01/ (0.5/2) = 104$ kN-m/m
- Checking against AASHTO 07 criteria for minimum reinforcement,
  - Criteria 1, $1.2 \times$ Cracking moment $= 125$ kN-m/m
  - Criteria 2, $1.33 \times$ 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

$$A_{st} = 0.003bh/2$$

$$= 0.003 \times 1000 \times 500/2$$

$$= 750 mm^2/m$$

(Ref. AASHTO 07, Article 5.6.3.6)

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

<table>
<thead>
<tr>
<th>SL NO</th>
<th>PLATE NO</th>
<th>$M_x$ (kN-m/m)</th>
<th>SL NO</th>
<th>PLATE NO</th>
<th>$M_x$ (kN-m/m)</th>
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</thead>
<tbody>
<tr>
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<td>9</td>
<td>1385</td>
<td>24</td>
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<td>1434</td>
<td>8</td>
<td>10</td>
<td>1378</td>
<td>-30</td>
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<tr>
<td>3</td>
<td>1427</td>
<td>6</td>
<td>11</td>
<td>1371</td>
<td>37</td>
</tr>
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<tr>
<td>8</td>
<td>1392</td>
<td>-14</td>
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<td></td>
</tr>
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</table>
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  $M_u = 179\, \text{kN-m/m}$
Compressive strength of Concrete, $f_c' = 25\, \text{MPa}$
Thickness of Pile Cap= 0.65m  \quad (t-150 = 650, \text{ here } t=800\, \text{mm})
Modulus of Rupture = 0.52 x $\sqrt{f_c'} = 2.6\, \text{MPa}$
Moment of Inertia = $b \times h^3/12 = 1 \times 0.65^3/12 = 0.023\, \text{m}^4$
Cracking Moment= $2.6 \times 1000 \times 0.023/ (0.65/2) = 184\, \text{kN-m/m}$
Checking against AASHTO 07 criteria for minimum reinforcement,
Criteria 1, 1.2 x Cracking moment= 221\, \text{kN-m/m}
Criteria 2, 1.33 x Flexural Moment= 238\, \text{kN-m/m}

So, minimum reinforcement required is for 221\, \text{kN-m/m} which is greater than design moment 179\, \text{kN-m/m}. So, design moment finally considered is 179\, \text{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 129 kN-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 281 kN-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

![Reinforcement Details of Pile Cap](image)
2.5 Structural Design of Pile

Material Properties

Concrete Cylinder Strength at 28 Days, $f'_c = 25$ MPa
Yield Strength of reinforcing steel $f_y = 413$ MPa

General Data

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

Maximum Horizontal Load in Pile (factored) $P_x = 73$ kN
Maximum Axial Load in Pile (factored) $P_y = 769$ kN
Moment Applied at the Pile Head (factored) $M_T = 189$ 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

<table>
<thead>
<tr>
<th>Soil</th>
<th>$n_h$ (kN/m$^3$)</th>
</tr>
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<tbody>
<tr>
<td>Medium Dense Sand</td>
<td>2500</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>1500</td>
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<tr>
<td>Soft Clay</td>
<td>700</td>
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<tr>
<td>Very Soft Clay</td>
<td>350</td>
</tr>
<tr>
<td>Organic Silt</td>
<td>150</td>
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</table>

NB: For loose sub merged cohesion less soil $n_h = 1400$ kN/m$^3$
Allowable Compressive Stress in pile = 6.25 N/mm$^2$ (W. C. Teng, 1962)
(Considering Structural Strength of Pile only)

Proposed Reinforcement:

<table>
<thead>
<tr>
<th>No. of Bar</th>
<th>12</th>
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<tbody>
<tr>
<td>Bar Diameter</td>
<td>25mm</td>
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<tr>
<td>Clear Cover</td>
<td>75mm</td>
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</table>
Analysis:
Axial Stress on Pile, $\sigma_x = 2.72 \text{ N/mm}^2$ Allowable stress of Pile, $\sigma_{allowable} = 6.25 \text{ N/mm}^2$

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, $A_s = n \times \text{Area of Rebar} = 9 = (12 \times 490 = 5880 \text{ mm}^2)$

Gross Area of Concrete, $A_g = 3.14 \times (D^2)/4 = 282600 \text{ mm}^2$
% Steel provided = $A_s/A_g = 2$

For the Circular pile:
Moment of Inertia, $I = 3.14 \times (D^4)/64 = 3.14 \times (0.6^4)/64 = 0.006 \text{ m}^4$
Stiffness Factor, $T = (EI/\eta h)^{0.2} = 2.936 \text{ m}$
Minimum Length of pile, $L = 4 \times T = 12 \text{ m}$
Level of Fixity, $L' = 1.8 \times T = 5.29 \text{ m}$

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

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<tr>
<td>3</td>
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<tr>
<td>4</td>
<td>EP</td>
</tr>
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<td>5</td>
<td>EL</td>
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</table>

### Combination Load Cases

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<th>Primary</th>
<th>Primary L/C Name</th>
<th>Factor</th>
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### Node Displacement Summary

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<tr>
<th>Node</th>
<th>L.C</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Z (in)</th>
<th>Resultant (in)</th>
<th>rX (rad)</th>
<th>rY (rad)</th>
<th>rZ (rad)</th>
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<td>-0.001</td>
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<td>-0.454</td>
<td>-0.033</td>
<td>-0.030</td>
<td>0.457</td>
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<tr>
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### Plate Centre Stress Summary

<table>
<thead>
<tr>
<th>Plate</th>
<th>L.C</th>
<th>Gx (ksi)</th>
<th>Gy (ksi)</th>
<th>Sx (ksi)</th>
<th>Sy (ksi)</th>
<th>Sxy (ksi)</th>
<th>My (ksi)</th>
<th>My (ksi)</th>
<th>My (ksi)</th>
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