NCR Planning Board
Asian Development Bank

Capacity Development of the National Capital Region Planning Board (NCRPB) – Component B (TA No. 7055-IND)

FINAL REPORT
Volume V-A2: DPR for Flyover at Mohan Nagar Junction in Ghaziabad
Detailed Designs

July 2010

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Appendix D-2 : Super Structure Design: Cross Girder Design
Appendix D-3 : Design of Substructure & Foundation
Appendix D-1: Super Structure Design: Deck Slab
Deck slab design

General

Slab is designed as one way slab spanning between main beams. The slab is discretised into 8 beam elements for finding our sectional forces at various sections in the transverse direction.

Live load calculation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit in meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total width</td>
<td>8.5</td>
</tr>
<tr>
<td>Cantilever length</td>
<td>0.8</td>
</tr>
<tr>
<td>C/C of main beams(lo)</td>
<td>2.2</td>
</tr>
<tr>
<td>C/C of Cross girders(b)</td>
<td>10</td>
</tr>
</tbody>
</table>

Effective width of dispersion

Effective width \( k^* a^*(1-a/lo)+b1 \)

\( k \) depends on \( b/lo \) ratio
\( a = \) distance of the load from the nearest support
\( b/lo = 4.55 \)
\( k = 2.6 \) Refer cl. 305.16.2 IRC 1-2000

LOADING - CLASS A WHEELED - minimum distance from the ker

Impact factor

Refer clause 211.2 IRC 6-2000

\[ \text{Impact factor} = 1+4.5(6+L) \]

\[ = 1.55 \]

Tyre contact dimensions

0.5 x 0.25

\( b1 = \) Dispersion up to the top of the slab

\[ = 0.25+2^*0.065 = 0.38 \text{ m} \]

Effective dispersion along the span

Dispersion up to the bottom of the deck slab

\[ = \text{wheel dim. along span} + 2^*(0.065+0.24) = 1.11 \text{ m} \]

Maximum wheel load

57 kN including impact = 85.5 kN

Effective width for L1

\( a = 0.1 \text{ m} \)

\( b1 = 0.63 \text{ m} \)

L1/contact area = 122.62 \( (\text{kN/m}^2) \)

Effective width for L2

\( a = 0.3 \text{ m} \)

\( b = 1.05 \text{ m} \)

L2/contact area = 73.11 \( (\text{kN/m}^2) \)
Effective width for L3
\[ a = 0.8 \text{ m} \]
\[ b_{eff} = 1.70 \text{ m} \]

L2/contact area = 45.21 (kN/m²)
(incl. Impact)

Effective width for L4
\[ a = 1 \text{ m} \]
\[ b_{eff1} = 1.798 \text{ m} \]

L2/contact area = \( \frac{85.5}{(1.092 \times 0.362)} \)
(incl. Impact)
42.84 (kN/m²)

LOADING - CLASS 70R WHEELED - minimum distance from the ker

Refer clause 211.3 IRC 6-2000
Impact factor = 1.25

tyre contact dimensions .36 x .263

Dispersion perpendicular to span = 0.263 + 2 \times 0.075
0.413 m

Dispersion along span = 0.36 + 2 \times (0.075 + 0.24)
0.99 m

Maximum wheel load = 85 kN
Load with impact = 106.25 kN

Effective width of dispersion
For L1
\[ a = 0.72 \text{ m} \]
\[ b_{eff1} = 1.67 \text{ m} \]

L1/contact area = 64.18 kN/m²
(Including impact)

For L2
\[ a = 0.99 \text{ m} \]
\[ b_{eff2} = 1.83 \text{ m} \]

L2/contact area = 58.69 kN/m²
(Including impact)

LOADING - CLASS A WHEELED (For max: support moment)

For L1 And L2
\[ a = 0.9 \text{ m} \]
\[ L3 \]
\[ b_{eff} = 1.76 \text{ m} \]
\[ 0.4 \]
\[ 1.25 \]

L/contact area = 43.70 kN/m²
(including impact)
60.49 kN/m²

LOADING - CLASS 70R WHEELED (For max: support moment)

For both loads
\[ a = 0.965 \text{ m} \]
\[ b_{eff} = 1.82 \text{ m} \]

Load/contact area = 58.92 kN/m²
(including impact)
**LOADING - CLASS A WHEEL**

Impact factor
Refer clause 211.2 IRC 6-2000

Impact factor = \( \frac{1+4.5}{6+L} \) = 1.55

Impact factor = 1.5

Tyre contact dimensions 0.5 x 0.25

\( b_1 \) = Dispersion up to the top of the slab (0.25+2*0.075)

= 0.4 m

Effective dispersion along the span
Dispersion up to the bottom of the deck slab

= wheel dim. along span + 2*(0.75+0.2)

= 1.13 m

Maximum load at mid span

Maximum wheel load = 57 kN
Including impact = 85.5 kN

Effective width for L1

a = 1.1 m

beff1 = 1.83 m

L1/contact area = 41.35 (kN/m²)

Effecitve width for L2

a = 0.7 m

beff = 1.64 m

L2/contact area = 46.11 (kN/m²)

**LOADING - CLASS 70R WHEEL**

Maximum load at mid span
Refer clause 211.3 IRC 6-2000

Impact factor = 1.25

tyre contact dimensions 0.36 x 0.263

Dispersion perpendicular to span = 0.263+2*0.075

= 0.413 m

Dispersion along span = 0.36+2*(0.075+0.24)

= 0.99 m

Maximum wheel load = 85 kN
Load with impact = 106.25 kN

Effective width of dispersion

For L1

a = 1.1 m

beff1 = 1.84 m

L1/contact area = 58.23 kN/m²

(Incuding impact)

For L2

a = 0.83 m

beff2 = 1.76 m

L2/contact area = 61.09 kN/m²

(Incuding impact)
FIG. 2 LOAD ARRANGEMENT FOR TRANSVERSE ANALYSIS

GIRDER SPACING - 2.2m
STAAD INPUT

STAAD PLANE TRANSVERSE ANALYSIS OF DECK
UNIT METER KNS
PAGE LENGTH 100
JOINT COORDINATES
1 0 0 0; 2 0.8 0 0; 3 3.0 0 0; 4 5.2 0 0; 5 7.4 0 0
6 8.5 0 0;
MEMBER INCIDENCES
1 1 2 5
MEMBER PROPERTIES
1 TO 5 PRI YD 0.25 ZD 1
CONSTANTS
E CONCRETE ALL
POISSON CONCRETE ALL
SUPPORT
2 3 4 5 PINNED
LOAD 1 DL
MEM LOAD
1 TO 5 UNI GY -6.25
LOAD 2 SIDL
MEM LOAD
CRASH BARRIER
member load
1 UNI GY -6.25 0 0.5
5 UNI GY -6.25 0.6
WEARING COAT
member load
1 UNI GY -1.43 0.5
5 UNI GY -1.43 0 0.6
2 3 4 UNI GY -1.43
LOAD 3 LIVELoad (CLASS A SINGLE LANE)
MEM LOAD
1 UNI GY -122.62 0.585
2 UNI GY -122.62 0.0 0.415
2 UNI GY -73.11 1.395
3 UNI GY -73.11 0 0.2025
LOAD 4 LIVELoad (CLASS A DOUBLE LANE)
MEM LOAD
1 UNI GY -122.62 0.585
2 UNI GY -122.62 0.0 0.415
2 UNI GY -73.11 1.395
3 UNI GY -73.11 0 0.2025
3 UNI GY -45.21 0.55 2.055
4 UNI GY -45.21 0.0 0.05
4 UNI GY -42.84 0.5 1.5
LOAD 5 LIVELoad (CLASS 70 R MAX. SPAN MOMENT)
MEM LOAD
2 UNI GY -64.18 0.805 2.155
3 UNI GY -58.69 0.575 1.845
LOAD 6 LIVELoad (CLASS 70 R MAX. SUPPORT MOMENT)
MEM LOAD
3 UNI GY -58.92 0.61 1.86
4 UNI GY -58.92 0.34 1.59
LOAD 7 LIVELoad (CLASS A MAX. SUPPORT MOMENT)
MEM LOAD
2 UNI GY -43.7 0.655 1.945
3 UNI GY -43.7 0.255 1.545
LOAD 8 LIVELoad (CLASS 70 R MAX LOAD AT MID SPAN)
MEM LOAD
3 UNI GY -58.23 0.55 1.65
4 UNI GY -61.09 0.165 1.505
LOAD 9 LIVELoad (CLASS A MAX LOAD AT MID SPAN)
MEM LOAD
3 UNI GY -41.35 0.555 1.645
4 UNI GY -46.11 0.035 1.375
LOAD COMBINATION 10
1 1 2 1 3 1
LOAD COMBINATION 11
1 1 2 1 4 1
LOAD COMBINATION 12
1 1 2 1 5 1
LOAD COMBINATION 13
1 1 2 1 6 1
LOAD COMBINATION 14
1 1 2 1 7 1
LOAD COMBINATION 15
1 1 2 1 8 1
LOAD COMBINATION 16
1 1 2 1 9 1
PERFORM ANALYSIS
PRINT MEMBER FORCES MEMB
PRINT MAX FORCE ENVELOPE
LOAD LIST 10 TO 16
SEC 0.001 0.999 MEM 1 TO 5
PRINT SECTION FORCES
SEC .25 .5 .75 MEM 2 TO 4
PRINT SECTION FORCES
Print section forces
FINISH
Member numbers

BMD Envelope
Design of section

Material properties and design constants

<table>
<thead>
<tr>
<th>Material</th>
<th>M50</th>
<th>f'c (MPa)</th>
<th>E (GPa)</th>
<th>k</th>
<th>j</th>
<th>Q (MPa)</th>
<th>Depth</th>
<th>Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td></td>
<td>20.0</td>
<td>30</td>
<td>0.455</td>
<td>0.848</td>
<td>3.21</td>
<td>250</td>
<td>50</td>
</tr>
<tr>
<td>Steel</td>
<td>Fe415</td>
<td>200</td>
<td>16.67</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The design is carried out for maximum bending moments at the following locations:

(a) Cantilever support (hogging)
(b) Intermediate support (hogging)
(c) Mid span moment (sagging)

The design moments have been taken from staad Output

Refer to the staad details attached

Design

(a) Cantilever support (hogging)

Maximum moment = 7 KNm
Depth required = 47 mm
Provided depth = 192

Provided depth enough

Steel requirement

Ast = 215 mm²
provide 10mm bars at 150mm c/c
Bar area = 78.5 mm²
Steel provided = 524 mm²

(b) Intermediate support (hogging)

Maximum moment = 29.00 KNm (Load combination 13)
Depth required = 95 mm
Provided depth = 192

Provided depth enough

Steel requirement

Ast = 890 mm²
provide 16 mm bars at 150mm c/c
Bar area = 201 mm²
Steel provided = 1340 mm²

(c) Mid span moment (sagging) (Load combination 13)

Maximum moment = 23 KNm
Depth required = 85 mm
Provided depth = 192

Provided depth enough

Steel requirement

Ast = 706 mm²
provide 16 mm bars at 150mm c/c
Bar area = 201 mm²
Steel provided = 1340 mm²

Distribution steel

Design moment = 0.3"Limoment + 0.2" DL moment
Maz live load moment = 26 KNm
Maz dead load moment = 6 KNm
Design moment = 9
Ast required = 296.29 mm²
0.12% of Cross sectional area = 300 mm²
Provide 10mm dia bars
Bar Area = 78.5 mm²
Spacing = 261.6667
Provided 10mm bars at 175mm c/c
**STRUCTURAL DATA (Mid Girder)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span of Bridge</td>
<td>39520 mm (length of girder)</td>
</tr>
<tr>
<td>Centre to centre of bearing</td>
<td>38440 mm</td>
</tr>
<tr>
<td>Carriageway Width</td>
<td>15 mm</td>
</tr>
<tr>
<td>Width of footpath</td>
<td>0 mm</td>
</tr>
<tr>
<td>Width of crash barrier</td>
<td>500 mm</td>
</tr>
<tr>
<td>O/O of Parapet</td>
<td>17000 mm</td>
</tr>
<tr>
<td>Depth of Deck Slab</td>
<td>250 mm</td>
</tr>
<tr>
<td>Overall Depth of the Girder</td>
<td>2200 mm</td>
</tr>
<tr>
<td>C/C distance between the girders</td>
<td>2200 mm</td>
</tr>
<tr>
<td>Cantilever distance beyond end girder</td>
<td>800 mm</td>
</tr>
<tr>
<td>Web thickness (Running X)</td>
<td>350 mm</td>
</tr>
<tr>
<td>Web thickness (End Block X)</td>
<td>700 mm</td>
</tr>
<tr>
<td>Thickness of End Diaphragm</td>
<td>800 mm</td>
</tr>
<tr>
<td>Thickness of Intermediate Diaphragm</td>
<td>300 mm</td>
</tr>
<tr>
<td>Density of Concrete</td>
<td>2.5 T/m³</td>
</tr>
<tr>
<td>Grade of Concrete Used</td>
<td>M 500</td>
</tr>
<tr>
<td>Characteristic Compressive Strength</td>
<td>500 Kg/cm²</td>
</tr>
<tr>
<td>Permissible stress in Concrete.</td>
<td>165 Kg/cm²</td>
</tr>
<tr>
<td>Modulus of Elasticity of Concrete &quot;Ec&quot;</td>
<td>353553.3906 5000 √fck Mpa</td>
</tr>
<tr>
<td>Grade of Steel Used</td>
<td>Fe-415</td>
</tr>
<tr>
<td>Yield strength of Steel</td>
<td>2000 Kg/cm²</td>
</tr>
<tr>
<td>Modulus of Elasticity of Steel &quot;Es&quot;</td>
<td>2.0E+06 Kg/cm²</td>
</tr>
</tbody>
</table>

**References:**

I.R.C :18 - 2000 - Design Criteria for Prestressed Concrete Road Bridges (Post Tensioned)
I.R.C :21 - 2000 - For Plain and Reinforced Cement Concrete.
I.R.C :6 - 2000 - For Loads and Stresses

**SECTION PROPERTIES**

Section properties for the grillage members with proper sketches are calculated below.

a) **Simple Section at Mid span**
**1) Moment of Inertia (I_{zz})**

<table>
<thead>
<tr>
<th>Sl</th>
<th>Description</th>
<th>Nos</th>
<th>Area</th>
<th>x</th>
<th>Ax</th>
<th>Ax^2</th>
<th>I_{self}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1)</td>
<td>35 x 175.0 x 1</td>
<td>= 6125.00</td>
<td>117.50</td>
<td>7.2E+05</td>
<td>8.5E+07</td>
<td>1.6E+07</td>
<td></td>
</tr>
<tr>
<td>2)</td>
<td>70 x 15.0 x 1</td>
<td>= 1050.00</td>
<td>212.50</td>
<td>2.2E+05</td>
<td>4.7E+07</td>
<td>2.0E+04</td>
<td></td>
</tr>
<tr>
<td>3)</td>
<td>17.5 x 15 x 2</td>
<td>= 262.50</td>
<td>200.00</td>
<td>5.3E+04</td>
<td>1.1E+07</td>
<td>3.3E+03</td>
<td></td>
</tr>
<tr>
<td>4)</td>
<td>17.5 x 15 x 2</td>
<td>= 262.50</td>
<td>35.00</td>
<td>9.2E+03</td>
<td>3.2E+05</td>
<td>3.3E+03</td>
<td></td>
</tr>
<tr>
<td>5)</td>
<td>70 x 30 x 1</td>
<td>= 2100.00</td>
<td>15.00</td>
<td>3.2E+04</td>
<td>4.7E+05</td>
<td>1.6E+05</td>
<td></td>
</tr>
<tr>
<td>6)</td>
<td>17.5 x 15 x 2</td>
<td>= 262.50</td>
<td>35.00</td>
<td>9.2E+03</td>
<td>3.2E+05</td>
<td>3.3E+03</td>
<td></td>
</tr>
<tr>
<td>7)</td>
<td>0 x 15 x 2</td>
<td>= 0.00</td>
<td>93.33</td>
<td>0.0E+00</td>
<td>0.0E+00</td>
<td>0.0E+00</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>9800.00</td>
<td>= 1036000</td>
<td>= 105.7142857</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Distance of N.A from bottom "Vb" = \( \frac{Ax}{A} \) = 105.7142857 cm

Distance of N.A from top "Vt" = 220 - 105.714 = 114.2857143 cm

Moment of Inertia about N.A "I_{zz}" = \( I_{self} + Ax^2 - (Ax \times Vb) \) = 62113333.33 cm^4

Section Modulus about top "Zt" = \( \frac{I_{self}}{Vt} \) = 433708.3333 cm^3

Section Modulus about bottom "Zb" = \( \frac{I_{self}}{Vb} \) = 468873.8739 cm^3

---

**b) Simple section at end Block**

![Diagram of simple section at end Block](attachment:image.png)

<table>
<thead>
<tr>
<th>Sl</th>
<th>Description</th>
<th>Nos</th>
<th>Area</th>
<th>x</th>
<th>Ax</th>
<th>Ax^2</th>
<th>I_{self}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1)</td>
<td>70 x 220 x 1</td>
<td>= 15400.00</td>
<td>110.00</td>
<td>1.7E+06</td>
<td>1.9E+08</td>
<td>6.2E+07</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>15400.00</td>
<td>= 1694000</td>
<td>= 110</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Distance of N.A from bottom "Vb" = \( \frac{Ax}{A} \) = 110 cm

Distance of N.A from top "Vt" = 220 - 110 = 110 cm

Moment of Inertia about N.A "I_{zz}" = \( I_{self} + Ax^2 - (Ax \times Vb) \) = 62113333.33 cm^4
Section Modulus about top "Zt" = \( \frac{I_{self}}{Vt} = 6.2 \times 10^7 \) cm
\(^3\)

Section Modulus about bottom "Zb" = \( \frac{I_{self}}{Vb} = 6.2 \times 10^7 \) cm
\(^3\)

### Composite Section at Mid span for end Girders

| Sl | Description | Nos | Area | x | Ax | Ax
\(^2\) | I_{self} |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>220 x 25.0 x 1</td>
<td>5500.00</td>
<td>232.5</td>
<td>1.3 \times 10^6</td>
<td>3.0 \times 10^8</td>
<td>2.9 \times 10^5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>35 x 175.0 x 1</td>
<td>6125.00</td>
<td>117.5</td>
<td>7.2 \times 10^5</td>
<td>8.5 \times 10^7</td>
<td>1.6 \times 10^7</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>(17.5 x 15.0) / 2 x 2</td>
<td>262.50</td>
<td>200.0</td>
<td>5.3 \times 10^4</td>
<td>1.1 \times 10^6</td>
<td>3.3 \times 10^3</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>(17.5 x 15) / 2 x 2</td>
<td>262.50</td>
<td>35.0</td>
<td>9.2 \times 10^3</td>
<td>3.2 \times 10^5</td>
<td>3.3 \times 10^3</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>70 x 30 x 1</td>
<td>2100.00</td>
<td>15.0</td>
<td>3.2 \times 10^4</td>
<td>4.7 \times 10^6</td>
<td>1.6 \times 10^5</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>(70 x 15 x 2)</td>
<td>1050.00</td>
<td>212.5</td>
<td>2.2 \times 10^5</td>
<td>4.7 \times 10^7</td>
<td>2.0 \times 10^4</td>
<td></td>
</tr>
</tbody>
</table>

Total Area = 15300.00

Moment of Inertia about N.A "I_{zz}" = \( I_{self} + Ax^2 - (Ax \times Vb) \)

= 16101718.75 + 4.4 \times 10^8 - (2314750 \times 151.2908497 )

= 10648205.7 cm
\(^4\)

Distance of N.A from bottom "Vb" = \( \frac{Ax}{A} \)
A = 15300

= 2314750 = 151.2908497 cm
\(^3\)

Distance of N.A from top "Vt" = 245 = 151.291 = 93.70915033 cm
\(^3\)

Momemt of Inertia about N.A "I_{zz}" = 16101718.75 + 4.4 \times 10^8 - (2314750 \times 151.2908497 )

= 10648205.7 cm
\(^4\)

Section Modulus about top "Zt" = \( \frac{I_{self}}{Vt} = 1.1 \times 10^8 \) cm
\(^3\)

Section Modulus about bottom "Zb" = \( \frac{I_{self}}{Vb} = 703823.172 \) cm
\(^3\)
2) **Torsional Stiffness** ($I_{xx}$)

Refer EC Hambly's book on Bridge Deck Behaviour.

\[
\begin{align*}
    \text{Area 1} &= 400133.0618 \\
    \text{Area 2} &= 1762379.284 \\
    \text{Area 3} &= 1175849.905 \\
    \text{Area 4} &= 1018103.009 \\
    I_{xx} &= 4356465.259 \text{ cm}^4 \\
    &= 0.0436 \text{ m}^4
\end{align*}
\]

b) **Composite section at end Block for Inner Girders**
1) Moment of Inertia, \( I_{zz} \)

<table>
<thead>
<tr>
<th>Sl</th>
<th>Description</th>
<th>Nos</th>
<th>Area</th>
<th>( x )</th>
<th>( Ax )</th>
<th>( Ax^2 )</th>
<th>( I_{self} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1)</td>
<td>220 x 25 x 1</td>
<td>= 5500.00</td>
<td>232.5</td>
<td>1.3.E+06</td>
<td>3.0.E+08</td>
<td>2.9.E+05</td>
<td></td>
</tr>
<tr>
<td>2)</td>
<td>70 x 220 x 1</td>
<td>= 15400.00</td>
<td>110</td>
<td>1.7.E+06</td>
<td>1.9.E+08</td>
<td>6.2.E+07</td>
<td></td>
</tr>
<tr>
<td>3)</td>
<td>15 x 7.5 x 0</td>
<td>= 0.00</td>
<td>217.5</td>
<td>0.0.E+00</td>
<td>0.0.E+00</td>
<td>0.0.E+00</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>= 20900.00</td>
<td>-</td>
<td>-</td>
<td>2972750</td>
<td>4.8.E+08</td>
<td>6.2.E+07</td>
</tr>
</tbody>
</table>

Distance of N.A from bottom "Vb" = \( \frac{Ax}{A} \) = \( \frac{2972750}{20900} \) = 142.2368421 cms

Distance of N.A from top "Vt" = 245 - 142.237 = 102.7631579 cms

Moment of Inertia about N.A "I_{zz}" = \( I_{self} + Ax^2 - (Ax \times Vb) \)

\( = 62399791.67 + 4.8E+08 - (2972750 \times 142.2368421) \)

\( = 123214594.3 \text{ cm}^4 \)

Section Modulus about top "Zt" = \( \frac{I_{self}}{Vt} \) = \( \frac{1.2E+08}{102.763} \) = 1199015.258 cm³

Section Modulus about bottom "Zb" = \( \frac{I_{self}}{Vb} \) = \( \frac{1.2E+08}{142.237} \) = 866263.5677 cm³

2) Torsional Stiffness \( I_{xx} \)

Refer EC Hambly's book on Bridge Deck Behaviour.

\( \Sigma I_{xx} = 2157436.03 \text{ cm}^4 \)

\( = 0.2157 \text{ m}^4 \)
c) **End Diaphragm**

Effective flange width = \( bw + (lo /10) \), as per IRC: 21-2000, for L beams.

\[
x = 1482.81 \text{ mm}
\]

1) **Moment of Inertia (I)\(_{ZZ}\)**

<table>
<thead>
<tr>
<th>Area</th>
<th>( cm^4 )</th>
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</thead>
<tbody>
<tr>
<td>Area 1</td>
<td>5.7E+07</td>
</tr>
<tr>
<td>Area 2</td>
<td>9.7E+07</td>
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</tbody>
</table>

\[ \Sigma I_{XX} = 1.5E+08 \text{ cm}^4 \]

**Torsional Stiffness (I)\(_{XX}\)**

<table>
<thead>
<tr>
<th>Area</th>
<th>( cm^8 )</th>
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<tbody>
<tr>
<td>Area 1</td>
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</tr>
<tr>
<td>Area 2</td>
<td>1749469</td>
</tr>
</tbody>
</table>

\[ \Sigma I_{XX} = 2.288570 \text{ cm}^3 \]

\[ = 0.02289 \text{ m}^3 \]

---

d) **Intermediate Diaphragm**

Effective flange width = \( bw + (lo /5) \), as per IRC: 21-2000, for T beams.

\[
x = 1482.81 \text{ mm}
\]

**Moment of Inertia (I)\(_{ZZ}\)**

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<tr>
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<th>( cm^4 )</th>
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</thead>
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<td>Area 2</td>
<td>3.6E+07</td>
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</table>

\[ \Sigma I_{XX} = 5.8E+07 \text{ cm}^4 \]

**Torsional Stiffness (I)\(_{XX}\)**

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<td>Area 2</td>
<td>1749469</td>
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</tbody>
</table>

\[ \Sigma I_{XX} = 2.288570 \text{ cm}^3 \]

\[ = 0.02289 \text{ m}^3 \]
### Summary of Member Properties for Grillage Analysis

<table>
<thead>
<tr>
<th>Description</th>
<th>Area (m²)</th>
<th>C.G from Bottom (m)</th>
<th>Moment of Inertia Constant (m⁴)</th>
<th>Torsional Vb (in cm)</th>
<th>Zt (in cm³)</th>
<th>Zb (in cm³)</th>
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<tr>
<td>Running</td>
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<td>105.7143</td>
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<td>1.1000</td>
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<td>499187.5000</td>
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<td><strong>Composite</strong></td>
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<tr>
<td>Running 2&amp;3</td>
<td>1.5300</td>
<td>1.5129</td>
<td>1.0648</td>
<td>0.0436</td>
<td>151.2908</td>
<td>1136303.1822</td>
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<td>0.1250</td>
<td>0.0032</td>
<td>0.0064</td>
<td>-</td>
<td>0.0000</td>
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STAAD PLANE stage - 1
START JOB INFORMATION
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
joint coordinates
1 0 0 0; 2 0.54 0 0; 3 1.74 0 0; 4 4.94 0 0 10 34.58 0 0
11 37.78 0 0; 12 38.98 0 0; 13 39.52 0 0
member incidences
1 1 2 12
member property indian
1 2 11 12 pri ax 1.54 ix 0.001 iz 0.62
3 10 pri ax 1.26 ix 0.001 iz 0.558
4 to 9 pri ax 0.98 ix 0.001 iz 0.496
constants
e concrete all
poi concrete all
supports
2 12 pinned
load 1
member load
1 2 11 12 uni gy -38.5
3 10 uni gy -31.5
4 to 9 uni gy -24.5
Load 2 end girder
member load
slab
0.25*1.9*25 = 11.875
1 to 12 uni gy -11.875
pre-cast slab
0.05*1.5*25/2=0.9375
1 to 12 uni gy -0.9375
diaphragm
joint load
1 13 fy -4.9
7 FY -16.5
load 3 mid girder
member load
slab
0.25*2.2*25 = 13.75
1 to 12 uni gy -13.75
pre-cast slab
0.05*1.5*25=1.875
1 to 12 uni gy -1.875
diaphragm
joint load
1 13 fy -9.75
7 FY -33
perform analysis
PRINT MAXFORCES ENVELOPE
print member forces
print support reactions
FINISH
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 15-Dec-07
END JOB INFORMATION
INPUT WIDTH 79

unit kn meter

joint coordinates
1  0  0  0; 2  0.54  0  0; 3  1.74  0  0; 4  4.94  0  0  10  34.58  0  0
11  37.78  0  0; 12  38.98  0  0; 13  39.52  0  0
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repeat 1  0  0  -2.2
repeat 1  0  0  -1.1
repeat 1  0  0  -2.2
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -2.2
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -2.2
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -2.2
repeat 1  0  0  -2.2
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -1.1
repeat 1  0  0  -0.8

member incidences
1  2  12
13 14 15 24
25 27 28 36
37 40 41 48
49 53 54 60
61 66 67 72
73 79 80 84
85 92 93 96
97 105 106 108
109 118 119 120
121 131 132 132
133 144 145 144
145 157 158 156
157 1 14 168 1 13
169 2 15 180 1 13
181 3 16 192 1 13
193 4 17 204 1 13
205 5 18 216 1 13
217 6 19 228 1 13
229 7 20 240 1 13
241 8 21 252 1 13
253 9 22 264 1 13
265 10 23 276 1 13
277 11 24 288 1 13
289 12 25 300 1 13
301 13 26 312 1 13

member property indian

end members
1 to 12 145 to 156 pri ax 0.001 ix 0.0001 iy 0.0001 iz 0.0001

end members outer girders
13 14 23 24 133 134 143 144 pri ax 2.09 ix 0.216 iy 0.00001 iz 1.23

end members inner girders
25 26 35 36 49 50 59 to 62 71 72 85 86 pri ax 1.95 ix 0.213 iy 0.00001 iz 1.11
107 108 121 122 131 to 132 95 to 98 pri ax 1.95 ix 0.213 iy 0.00001 iz 1.11
taper members outer girders
15 22 135 142 pri ax 1.81 ix 0.13 iy 0.00001 iz 1.145
taper members inner girders
27 34 51 58 63 70 87 99 123 94 106 130 pri ax 1.672 ix 0.127 iy 0.00001 iz 1.037

middle membes - end girders
16 to 21 136 to 141 pri ax 1.53 ix 0.0436 iy 0.00001 iz 1.06

middle membes - inner girders
28 to 33 52 to 57 64 to 69 pri ax 1.393 ix 0.041 iy 0.00001 iz 0.964
88 to 93 100 to 105 124 to 129 pri ax 1.393 ix 0.041 iy 0.00001 iz 0.964

membes at bearing location
end cross girders
170 to 179 290 to 299 pri ax 1.76 ix 0.3136 iy 0.00001 iz 1.5
mid cross girders
230 to 239 pri ax 0.66 ix 0.023 iy 0.00001 iz 0.58
deck members near end cross girder
182 to 191 278 to 287 pri ax 0.435 ix 0.00045 iy 0.00001 iz 0.0023
deck members near mid cross girder
218 to 227 242 to 251 pri ax 0.935 ix 0.0097 iy 0.00001 iz 0.0049
deck members
e193 to 216 253 to 276 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
deck end members - along width
157 to 168 301 to 312 pri ax 0.0001 ix 0.00001 iy 0.00001 iz 0.00001
deck end members - along span
181 192 217 228 229 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
241 252 277 288 240 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
deck corner members
169 180 289 300 pri ax 0.62 ix 0.0064 iy 0.00001 iz 0.0032

Constants
E concrete all
Poi concrete all
den 25 all
supports
41 80 119 pinned
51 90 129 fixed but fx mz
load 1
member load
end members outer girders
13 14 23 24 133 134 143 144 uni gy -52.25
end members inner girders
25 26 35 36 49 50 to 59 to 62 71 72 85 86 uni gy -48.75
107 108 121 122 131 to 132 95 to 98 uni gy -48.75
taper members outer girders
15 22 135 142 95 to 98 uni gy -45.25
taper members inner girders
27 34 51 58 63 70 87 99 123 94 106 130 uni gy -41.8
middle members - end girders
16 to 21 136 to 141 uni gy -38.25
middle members - inner girders
28 to 33 52 to 57 64 to 69 uni gy -34.825
88 to 93 100 to 105 124 to 129 uni gy -34.825
tapers at bearing location
37 to 48 73 to 84 109 to 120 uni gy -6.875
cross girders
170 to 179 290 to 299 uni gy -44
mid cross girders
230 to 239 uni gy -16.5
load 2
member load
crash barrier
1 to 12 145 to 156 uni gy -9.4
Wearing course
13 to 24 uni gy -1.65 0.5
133 to 144 uni gy -1.65 0.3
25 to 132 uni gy -1.65
median .3*25 = 7.5
7.5+1.65 = 5.85)
73 to 84 uni gy -5.85
DEFINE MOVING LOAD
CLASS A four LANES
TYPE 1 LOAD 13.5 13.5 57 57 34 34 34 34 DISTANCE 1.1 3.2 1.2 4.3 3.0 3.0 3.0
TYPE 2 LOAD 13.5 13.5 57 57 34 34 34 34 DISTANCE 1.1 3.2 1.2 4.3 3.0 3.0 3.0
LOAD GENERATION 60
a ECCENTRIC (near FP)
TYPE 1 -18.8 0. -0.9 XINC 1.
TYPE 2 -18.8 0. -2.7 XINC 1.
TYPE 1 -18.8 0. -4.4 XINC 1.
TYPE 2 -18.8 0. -6.2 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
d ECCENTRIC (near median FP)
TYPE 1 -18.8 0. -7.6 XINC 1.
TYPE 2 -18.8 0. -5.8 XINC 1.
TYPE 1 -18.8 0. -4.1 XINC 1.
TYPE 2 -18.8 0. -2.3 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
c ECCENTRIC TWO LANE (near fp)
TYPE 1 -18.8 0. -0.9 XINC 1.
TYPE 2 -18.8 0. -2.7 XINC 1.
TYPE 1 -18.8 0. -4.4 XINC 1.
TYPE 2 -18.8 0. -5.2 XINC 1.
d ECCENTRIC TWO LANE (near median)
TYPE 1 -18.8 0. -7.6 XINC 1.
TYPE 2 -18.8 0. -5.8 XINC 1.
TYPE 1 -18.8 0. -4.1 XINC 1.
TYPE 2 -18.8 0. -2.3 XINC 1.
CLASS 70R WHEELED
DEFINE MOVING LOAD
TYPE 3 LOAD 40 60 60 85 85 85 85 DISTANCE 3.96 1.52 2.13 1.37 3.05 1.37
TYPE 4 LOAD 40 60 60 85 85 85 85 DISTANCE 3.96 1.52 2.13 1.37 3.05 1.37
LOAD GENERATION 55
E 70R WHEELED + CLASS A (70R E1) (70R near FP)
TYPE 3 -13.4 0. -2.13 XINC 1.
TYPE 4 -13.4 0. -4.06 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
f CLASS A + 70 R (70RE2) (70R near median)
TYPE 3 -13.4 0. -5.87 XINC 1.
TYPE 4 -13.4 0. -4.94 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
g 70R WHEELED (on both 2 lanes near FP)
TYPE 3 -13.4 0. -2.13 XINC 1.
TYPE 4 -13.4 0. -4.06 XINC 1.
TYPE 3 -13.4 0. -10.63 XINC 1.
TYPE 4 -13.4 0. -12.56 XINC 1.
h 70R WHEELED (on both 2 lanes near FP)
TYPE 3 -13.4 0. -6.87 XINC 1.
TYPE 4 -13.4 0. -4.94 XINC 1.
TYPE 3 -13.4 0. -10.63 XINC 1.
TYPE 4 -13.4 0. -12.56 XINC 1.
perform analysis
print maxforce envelope
print support reactions
FINISH
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DESIGN OF PSC GIRDER

Calculation of Bending Moments due to the following at Various X

Summary of Shear & Moments at Various X obtained from respective STAAD analysis

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Vehicular Loads

a) Class _70r -- Total
| M | 20.94 | 204.02 | 244.07 | 245.78 | 243.68 |
| S | 49.44 | 36.93  | 27.11  | 18.28  | 29.62  |

b) Class _A -- Total
| M | -0.33 | 107.92 | 176.20 | 209.24 | 220.25 |
| S | 30.39 | 24.34  | 19.05  | 13.99  | 9.03   |

Ultimate
| M | 42.25 | 1054.91| 1589.71| 1874.44| 2000.44|
| S | 355.75| 259.30 | 184.08 | 108.38 | 83.86  |

The effect of vehicular impact is taken as follows

Impact Factor for Class 70r = \( \frac{4.5}{6 + 38.44} = 15.00\% \) (Vide Fig-5 on page 23 of IRC-6:2000)

Impact Factor for Class A = \( \frac{10.126\%}{211.2} \) (Vide CI:211.2 on page 22 of IRC-6:2000)

Position of Cables at Running X

Prestressing Details

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Cable make = BBRV Cona make.
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<td>Value of friction co-efficient</td>
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<tr>
<td>(15mm dia)</td>
<td></td>
</tr>
<tr>
<td>Stressing @ 0.74 times UTS</td>
<td>20.646 T / STRAND</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td></td>
</tr>
<tr>
<td>Force per cable (A, B)</td>
<td>454.212 T</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td>Force per cable as per BBR's brochure is 2827Kn at 0.8UTS.</td>
</tr>
<tr>
<td>Force per cable C</td>
<td>454.212 T</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td></td>
</tr>
<tr>
<td>C/S Area of 1 Cable</td>
<td>33 cm²</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td>As per BBRV's brochure.</td>
</tr>
<tr>
<td>C/S Area of 1 Cable</td>
<td>33 cm²</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td></td>
</tr>
<tr>
<td>Total Force</td>
<td>1362.636 T</td>
</tr>
<tr>
<td>(15mm dia)</td>
<td></td>
</tr>
<tr>
<td>Total C/S Area</td>
<td>33 x 2 + 33.0 x 1</td>
</tr>
<tr>
<td></td>
<td>99 cm²</td>
</tr>
<tr>
<td>Clear cover to Prestressing Cables</td>
<td>100 mm, 75 mm whichever is greater.</td>
</tr>
</tbody>
</table>
### Girder No : 1

#### Summary of Stresses at Various X

<table>
<thead>
<tr>
<th>Stress due</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) DL of girder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.099</td>
<td>81.128</td>
<td>72.443</td>
<td>91.570</td>
<td>97.946</td>
</tr>
<tr>
<td>T</td>
<td>0.099</td>
<td>38.101</td>
<td>78.317</td>
<td>98.995</td>
<td>105.888</td>
</tr>
<tr>
<td>2) DL of deck slab</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.040</td>
<td>24.004</td>
<td>48.525</td>
<td>62.462</td>
<td>68.267</td>
</tr>
<tr>
<td>T</td>
<td>0.040</td>
<td>43.936</td>
<td>52.459</td>
<td>67.527</td>
<td>73.802</td>
</tr>
<tr>
<td>2) S.I.D.Load</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.515</td>
<td>2.610</td>
<td>9.147</td>
<td>12.550</td>
<td>15.788</td>
</tr>
<tr>
<td>T</td>
<td>0.797</td>
<td>1.186</td>
<td>4.154</td>
<td>5.700</td>
<td>7.170</td>
</tr>
<tr>
<td>4) Vehicular Live Load</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Class 70-R</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>2.418</td>
<td>14.494</td>
<td>34.678</td>
<td>34.921</td>
<td>34.622</td>
</tr>
<tr>
<td>T</td>
<td>3.740</td>
<td>6.582</td>
<td>15.749</td>
<td>15.859</td>
<td>15.724</td>
</tr>
<tr>
<td>Class - A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>-0.038</td>
<td>7.667</td>
<td>25.035</td>
<td>29.729</td>
<td>31.294</td>
</tr>
<tr>
<td>T</td>
<td>-0.059</td>
<td>3.482</td>
<td>11.370</td>
<td>13.501</td>
<td>14.212</td>
</tr>
</tbody>
</table>

#### Lifting point for cables

<table>
<thead>
<tr>
<th>B Bottom</th>
<th>T Top</th>
</tr>
</thead>
<tbody>
<tr>
<td>y</td>
<td>x</td>
</tr>
<tr>
<td>----------</td>
<td>-------</td>
</tr>
<tr>
<td>1000.000</td>
<td>18760.0</td>
</tr>
<tr>
<td>6.10829206</td>
<td></td>
</tr>
<tr>
<td>550.000</td>
<td>17760.0</td>
</tr>
<tr>
<td>3.54872508</td>
<td></td>
</tr>
<tr>
<td>100.000</td>
<td>13760.0</td>
</tr>
<tr>
<td>0.83278749</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Half length of cable</th>
<th>y</th>
<th>x</th>
<th>k</th>
<th>a</th>
</tr>
</thead>
<tbody>
<tr>
<td>19760.000</td>
<td>14820.000</td>
<td>9880.000</td>
<td>4940.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cables</th>
<th>Dist</th>
<th>1-1</th>
<th>3-3</th>
<th>5-5</th>
<th>7-7</th>
<th>9-9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cables</td>
<td></td>
<td>1500.00</td>
<td>1192.69</td>
<td>874.06</td>
<td>694.11</td>
<td>650.00</td>
</tr>
<tr>
<td>C</td>
<td>0.11</td>
<td>0.08</td>
<td>0.05</td>
<td>0.02</td>
<td>0.00</td>
<td>6.108292</td>
</tr>
<tr>
<td>Cables</td>
<td>Dist</td>
<td>950.000</td>
<td>686.585</td>
<td>508.275</td>
<td>400.000</td>
<td>400.000</td>
</tr>
<tr>
<td>Cables</td>
<td></td>
<td>250.000</td>
<td>191.087</td>
<td>157.951</td>
<td>150.000</td>
<td>150.000</td>
</tr>
<tr>
<td>C</td>
<td>0.015</td>
<td>0.009</td>
<td>0.004</td>
<td>0.000</td>
<td>0.000</td>
<td>0.832787</td>
</tr>
</tbody>
</table>

**Resultant C.G:**

| 950.000 | 690.12 | 513.43 | 414.70 | 400.00 |

**Calculation of Losses in Prestress:**

Ref: IRC 18: 2000, Cl. 11.6, Table 5.

1) Instantaneous Losses consisting of the following:

a) **Frictional Loss:** (for galvanised wire cable)

\[
Wobble \ Coefficient = k = 0.002 \quad P_x = P_x e^{(k + x)}
\]
<table>
<thead>
<tr>
<th>Cable Nos</th>
<th>Section</th>
<th>Tension Factor</th>
<th>Prestress Force(T)</th>
<th>Ave Force (T)</th>
<th>Elongation (cms)</th>
<th>Total Elong - gator (cms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>c A</td>
<td>1</td>
<td>454.21</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.99838</td>
<td>453.48</td>
<td>453.84</td>
<td>0.56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.94429</td>
<td>428.91</td>
<td>441.19</td>
<td>12.56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0.94241</td>
<td>428.05</td>
<td>428.48</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b A</td>
<td>1</td>
<td>454.21</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.99822</td>
<td>453.40</td>
<td>453.81</td>
<td>0.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.95328</td>
<td>432.99</td>
<td>443.20</td>
<td>11.93</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0.94947</td>
<td>431.26</td>
<td>432.13</td>
<td>1.31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a A</td>
<td>1</td>
<td>454.21</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.99822</td>
<td>453.40</td>
<td>453.81</td>
<td>0.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.96873</td>
<td>440.01</td>
<td>446.71</td>
<td>9.31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0.95717</td>
<td>434.76</td>
<td>437.38</td>
<td>3.98</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

b) Slip Loss:
Assuming 6mm slip in the Prestressing Cables & that the Cables are stressed from both ends.
Force due to 6mm slip. (for 13mm dia) for Cable A and B
\[ F = \frac{AE\delta}{\mu} \]
\[ = \frac{33.000 \times 2.0E+06 \times 0.6}{0.17} = 4.0E+05 \text{ Kg} \]
For Both end stressing case \( \frac{1}{2} \) force on either side of midspan is considered.

\[
\text{Force} = \frac{396000}{2 \times 1000} = 198.00 \text{ T}
\]

\[F = AE \delta = 33.00 \times 2.0E+06 \times 0.6 = 4.0E+05 \text{ Kg}\]

For Both end stressing case \( \frac{1}{2} \) force on either side of midspan is considered.

\[
\text{Force} = \frac{396000}{2 \times 1000} = 198.00 \text{ T}
\]

Computation of Force after slip for the cables:

<table>
<thead>
<tr>
<th>Cable Nos</th>
<th>Force in Tonne</th>
<th>Force Diagram after 6mm Slip</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>453.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>428.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>428.1</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>453.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>433.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>431.3</td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>446.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>437.4</td>
<td></td>
</tr>
</tbody>
</table>

Computation of Force after slip for the cables:
### Summary of Cable Forces after slip loss at various X°:

<table>
<thead>
<tr>
<th>Cable Nos</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>c</td>
<td>419.4</td>
<td>426.1</td>
<td>432.8</td>
<td>440.2</td>
<td>428.1</td>
</tr>
<tr>
<td>b</td>
<td>410.7</td>
<td>416.43</td>
<td>422.10</td>
<td>427.78</td>
<td>431.26</td>
</tr>
<tr>
<td>a</td>
<td>421.68</td>
<td>424.23</td>
<td>426.78</td>
<td>429.33</td>
<td>437.38</td>
</tr>
<tr>
<td>Σ Force</td>
<td>1251.81</td>
<td>1266.73</td>
<td>1281.66</td>
<td>1297.27</td>
<td>1296.70</td>
</tr>
</tbody>
</table>

- Average force in the cables = 429.3 T
- Force / strands = 429.3 / 22 = 19.5 T
- Ultimate Force / strand = 27.9 T
- Ultimate stress / strand = 0.699 < 0.74 times ultimate. Hence safe

**Average force in the cables (II and III cable):**

- Force / strands = 849.5 / 44 = 19.3 T
- Ultimate Force / strand = 27.9 T
- Ultimate stress / strand = 0.692 < 0.74 times ultimate. Hence safe

**Average Stress in the cables:**

Horizontal Component of Cable forces at various X°:

<table>
<thead>
<tr>
<th>Cable Nos</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>c</td>
<td>417.0</td>
<td>424.8</td>
<td>432.2</td>
<td>440.0</td>
<td>428.1</td>
</tr>
<tr>
<td>b</td>
<td>410.0</td>
<td>416.0</td>
<td>421.9</td>
<td>427.8</td>
<td>431.3</td>
</tr>
<tr>
<td>a</td>
<td>421.6</td>
<td>424.2</td>
<td>426.8</td>
<td>429.3</td>
<td>437.4</td>
</tr>
<tr>
<td>Σ Force</td>
<td>1249.60</td>
<td>1264.99</td>
<td>1280.94</td>
<td>1297.16</td>
<td>1296.70</td>
</tr>
</tbody>
</table>

Vertical Component of Cable forces at various X°:

<table>
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<th>Cable Nos</th>
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<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>c</td>
<td>44.6</td>
<td>33.4</td>
<td>21.8</td>
<td>9.9</td>
<td>0.0</td>
</tr>
<tr>
<td>b</td>
<td>25.4</td>
<td>18.6</td>
<td>11.6</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>a</td>
<td>6.1</td>
<td>4.0</td>
<td>1.7</td>
<td>0.0</td>
<td>0.0</td>
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<tr>
<td>Σ Force</td>
<td>76.18</td>
<td>55.99</td>
<td>35.18</td>
<td>9.85</td>
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Effect of Prestressing :-

<table>
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<tr>
<th>Sections</th>
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<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force, F (T)</td>
<td>1248.6</td>
<td>1265.0</td>
<td>1280.9</td>
<td>1297.2</td>
<td>1296.7</td>
</tr>
<tr>
<td>Area, A (cm²)</td>
<td>15400.0</td>
<td>9800.0</td>
<td>9800.0</td>
<td>9800.0</td>
<td>9800.0</td>
</tr>
<tr>
<td>F/A (Kg/cm²)</td>
<td>81.1</td>
<td>129.1</td>
<td>130.7</td>
<td>132.4</td>
<td>132.3</td>
</tr>
<tr>
<td>Vb (cms)</td>
<td>110.0</td>
<td>105.7</td>
<td>105.7</td>
<td>105.7</td>
<td>105.7</td>
</tr>
<tr>
<td>e = Vb-x (cms)</td>
<td>15.0</td>
<td>36.7</td>
<td>54.4</td>
<td>64.2</td>
<td>65.7</td>
</tr>
<tr>
<td>Zt (cm³)</td>
<td>564667</td>
<td>433708</td>
<td>433708</td>
<td>433708</td>
<td>433708</td>
</tr>
<tr>
<td>Zb (cm³)</td>
<td>564666.7</td>
<td>468873.9</td>
<td>468874</td>
<td>468874</td>
<td>468874</td>
</tr>
<tr>
<td>F x e / Zt</td>
<td>33.2</td>
<td>107.0</td>
<td>160.6</td>
<td>192.1</td>
<td>196.5</td>
</tr>
<tr>
<td>F/A -(Fxe / Zt)</td>
<td>47.9</td>
<td>22.0</td>
<td>-29.9</td>
<td>-59.8</td>
<td>-64.2</td>
</tr>
<tr>
<td>F/A +(Fxe / Zt)</td>
<td>114.2</td>
<td>228.1</td>
<td>279.2</td>
<td>310.1</td>
<td>314.1</td>
</tr>
</tbody>
</table>

2) Time Dependent Losses consisting of the following :-


It is proposed to stress the cables after 28-Days when the concrete attains 100% of its full strength. The effect of Prestress & Dead Load acts together.

Descriptions | 1 - 1 | 3 - 3 | 5 - 5 | 7 - 7 | 9 - 9 |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>47.910</td>
<td>114.246</td>
<td>22.032</td>
<td>228.100</td>
<td>-29.876</td>
</tr>
<tr>
<td>Dead Load</td>
<td>-0.099</td>
<td>0.099</td>
<td>38.101</td>
<td>-81.128</td>
<td>78.317</td>
</tr>
<tr>
<td>Resultant</td>
<td>47.810</td>
<td>114.345</td>
<td>60.132</td>
<td>146.972</td>
<td>48.441</td>
</tr>
</tbody>
</table>

Sections

<table>
<thead>
<tr>
<th>Sections</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress at the C.G of the Cables</td>
<td>128</td>
<td>60.132</td>
<td>68.85</td>
<td>150.73</td>
<td>140.97</td>
</tr>
<tr>
<td>95</td>
<td>114.35</td>
<td>69.87</td>
<td>51.54</td>
<td>208.81</td>
<td>218.53</td>
</tr>
</tbody>
</table>

Average stress at C.G of the Xn = \( \frac{148.86 \times 5.657 \times 148.9}{200000} = \frac{421.05}{5.66} \times 353553 \% \) Loss = 2.446 %

\( m = \frac{200000}{5.66} \)

Initial relaxation loss (Cl-11.4 IRC 18-2000) = 2.446 %
b) Creep of Concrete:  
(Vide Cl:11.1 of I.R.C:-18-2000)

<table>
<thead>
<tr>
<th></th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_t )</td>
<td>47.910</td>
<td>114.246</td>
<td>22.032</td>
<td>228.100</td>
<td>-29.876</td>
</tr>
<tr>
<td>( \sigma_b )</td>
<td>114.246</td>
<td>22.032</td>
<td>228.100</td>
<td>-29.876</td>
<td>279.249</td>
</tr>
<tr>
<td>( \sigma_t )</td>
<td>22.032</td>
<td>228.100</td>
<td>-29.876</td>
<td>279.249</td>
<td>-59.781</td>
</tr>
<tr>
<td>( \sigma_b )</td>
<td>228.100</td>
<td>-29.876</td>
<td>279.249</td>
<td>-59.781</td>
<td>310.066</td>
</tr>
<tr>
<td>( \sigma_t )</td>
<td>-29.876</td>
<td>279.249</td>
<td>-59.781</td>
<td>310.066</td>
<td>-64.156</td>
</tr>
<tr>
<td>( \sigma_b )</td>
<td>279.249</td>
<td>-59.781</td>
<td>310.066</td>
<td>-64.156</td>
<td>314.053</td>
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<tr>
<td>Prestress</td>
<td>-0.099</td>
<td>0.099</td>
<td>38.101</td>
<td>-81.128</td>
<td>78.317</td>
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<td>Dead Load</td>
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<td>-1.3015</td>
<td>1.053</td>
<td>-15.934</td>
<td>3.411</td>
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<td>Inst. Loss %</td>
<td>5.706</td>
<td>0.099</td>
<td>38.101</td>
<td>-81.128</td>
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<tr>
<td>Resultant</td>
<td>45.077</td>
<td>107.826</td>
<td>58.875</td>
<td>133.957</td>
<td>50.146</td>
</tr>
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</table>

<table>
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<th>7 - 7</th>
<th>9 - 9</th>
</tr>
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<tbody>
<tr>
<td>Stress at the C.G of the Cables</td>
<td>45.08</td>
<td>58.675</td>
<td>50.146</td>
<td>48.63</td>
<td>49.39</td>
</tr>
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<td>95</td>
<td>107.83</td>
<td>133.957</td>
<td>50.146</td>
<td>48.63</td>
<td>49.39</td>
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<tr>
<td>Average stress at C.G of the cables</td>
<td>138.12 Kg/cm²</td>
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</tr>
<tr>
<td>Creep Strain at 100% Maturity at 28-days</td>
<td>0.00040 / 10 Mpa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E_s )</td>
<td>2000000</td>
<td></td>
<td></td>
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<tr>
<td>Loss</td>
<td>0.00040 x 138.12 x 2000000 = ( 1104.93 ) Kg/cm²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shrinkage Strain at 100% Maturity at 28-days</td>
<td>0.00019</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shrinkage Strain at 100% Maturity at 28-days</td>
<td>0.00019 x 2000000 = ( 380.0 ) Kg/cm²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| c) Shrinkage of Concrete:  
(Vide Cl:11.1 of I.R.C:-18-2000) |
| d) Relaxation of H.T.Steel  
(Vide Cl:11.4 of I.R.C:-18-2000) |

From table 4A, percentage relaxation loss corresponding to 0.696 times Ultimate Strength of Wires is calculated as for low relaxation steel

\[ \text{Relaxation Loss corresponding to } 0.696 \text{ times Ultimate Strength of Wires is calculated as for low relaxation steel} \]

| Inst. Loss % | 2.446 | 0.696 |
| 2.4 % | 0.696 |
| 2.5 % | 0.70 |

Relaxation Loss corresponding \( 2.446 \% \) of the average prestress \( \text{ie... } 12918 \text{ Kg/cm²} = 632.04 \text{ Kg/cm²} \text{ie... } 4.89\% \)

\[ \text{Total Time Dependent Losses } = 2116.96 \text{ Kg/cm²} \]
% Loss = \frac{2116.96 \times 100}{12917.5} = 16.388\%

3) Time Dependent Losses consisting of the following:-

a) Creep of Concrete: = 1104.93 Kg/cm²

b) Shrinkage of Concrete: = 380.00 Kg/cm²

c) Relaxation of H.T.Steel = 1.47%

\text{Total Loss} = \frac{1484.93 \times 20 + 1.47}{12917.5} = 3.767\%

Recapitulation of Stresses at Various Stages of Prestressing:

\begin{array}{|c|c|c|c|c|c|c|c|c|c|}
\hline
\text{Descriptions} & 1 - 1 & 3 - 3 & 5 - 5 & 7 - 7 & 9 - 9 \\
\hline
\text{Prestress} & 47.910 & 114.246 & 22.032 & 229.100 & -29.876 & 279.249 & -59.781 & 310.096 & -64.156 \\
\text{Dead Load} & -0.099 & 0.099 & 38.101 & -81.128 & 78.317 & -72.443 & 98.995 & -91.570 & 105.888 \\
\text{Bal: Loss %} & 5.706 & & & & & & & & \\
\text{Resultant (28-Days)} & 45.077 & 107.826 & 58.875 & 133.957 & 50.146 & 190.872 & 42.625 & 200.832 & 45.392 \\
\text{Vehicular Load} & -0.040 & 0.040 & 43.936 & -24.004 & 52.459 & -48.525 & 67.527 & -62.462 & 73.802 \\
\text{Bal: Loss %} & 16.388 & & & & & & & & \\
\text{20%Extra Loss} & -1.805 & -4.304 & -0.830 & -8.592 & 1.125 & -10.519 & 2.252 & -11.681 & 2.417 \\
\text{Resultant} & 38.324 & 82.937 & 106.138 & 46.674 & 128.529 & 42.239 & 141.402 & 28.398 & 155.019 \\
\hline
\end{array}

Remarks about Stresses at various Stages of Prestressing:

1) Permissible Temporary Stress in Concrete

a) Maximum Compressive Stress immediately after Prestressing shall not exceed minimum of the following:

\text{300 Kg/cm² or } \text{0.5 Fcj}

\text{Fcj} = \text{Expected Concrete Strength at the time of Prestressing.}

\text{Max Compressive Stress developed} = \frac{\text{0.5} \times 500}{10} = 250 \text{ Kg/cm²}

\text{Hence O.K}

b) Temporary Tensile Stress in the extreme fibre immediately after Prestressing shall not exceed,

\text{1} \text{ of Maximum Compressive Stress immediately after Prestressing}

Minimum Stress developed = \frac{1}{70} \times 250.00 = 25.00 \text{ Kg/cm}^2

Tension developed is within limits at this stage. Hence O.K

2) Permissible Stress in Concrete at Service Condition

a) Maximum Compressive Stress allowed during Service Condition
   \[ F_{ck} = 0.33 \times 500 = 165 \text{ Kg/cm}^2 \]
   Maximum Compressive Stress attained at Service
   \[ = 155.0 \text{ Kg/cm}^2 \] Hence O.K

b) Minimum Stress attained at Service
   \[ = 16.211 \text{ Kg/cm}^2 \] No Tension is developed. The Stresses are Compressive only. Hence O.K

Check for Deflection at Midspan.

Downward deflection is given by

\[ \delta = \frac{5}{48} \times \frac{M \times L^2}{E \times I} \]

M = Moment = 1134.132 Tm
L = Span = 39.52 m
E = Modulus of Elasticity of Concrete = 353553.39 \text{ Kg/cm}^2
I = Moment of Inertia = 6.2 \times 10^7 \text{ cm}^4

\[ \therefore \delta_1 = \frac{5}{48} \times \frac{113413240 \times 3952^2}{353553.39 \times 62113333} \]

\[ \delta_1 = 8.402075 \text{ cms} = 84.02075 \text{ mm} \]

Upward deflection due to prestress

\[ \delta_2 = \frac{P \times e \times L}{8 \times E \times I} \]

P = Prestressing Force at Mid Span = 1296.699 T
E = Eccentricity = 65.7 \text{ cms}
I = Moment of Inertia = 353553.39 \text{ Kg/cm}^2

\[ \therefore \delta_2 = \frac{1.3 \times 10^6}{8} \times \frac{65714286 \times 3952^2}{353553.39 \times 62113333} \]

\[ \delta_2 = 7.5753576 \text{ mm} \]

Net \( \delta = 84.02 - 75.75 = 8.27 \text{ Downward} \)

Permissible Deflection

\[ \frac{L}{800} = 39520 \]

\[ 800 \]

\[ 49.4 \text{ mm} \text{ Hence O.K} \]
Check for Ultimate Strength at Various X

(i) Failure by yield of steel (Under Reinforced section)

\[ M_{ult} (Steel) = 0.9 d_s A_s f_p \]

- \( M_{ult} \) = Area of High Tensile Steel
- \( A_s \) = High Tensile Steel
- \( f_p \) = The Ultimate Tensile Strength of Steel
- \( d_s \) = The Depth of the beam from the maximum compression edge to C.G of Tendons.

(ii) Failure by crushing of concrete (Over Reinforced section)

\[ M_{ult} (Conc) = 0.176 b d_s^2 f_{ck} + (2/3) x 0.8(B_f - b)(d_s - t/2)x t x f_{ck} \]

- \( b \) = Total width of the Webs.
- \( B_f \) = Overall width of the top flange
- \( t \) = Average thickness of flange.

UTS = 0.74

<table>
<thead>
<tr>
<th>Sections</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_s (m^2) )</td>
<td>0.00990</td>
<td>0.010</td>
<td>0.010</td>
<td>0.010</td>
<td>0.010</td>
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<tr>
<td>( f_{p, (T/mm^2)} )</td>
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<td>186000</td>
<td>186000</td>
<td>186000</td>
<td>186000</td>
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<tr>
<td>( d_s (m) )</td>
<td>1.500</td>
<td>1.760</td>
<td>1.937</td>
<td>2.035</td>
<td>2.050</td>
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<tr>
<td>( M_{ult, Steel} (Tm) )</td>
<td>2485.89</td>
<td>2916.58</td>
<td>3209.40</td>
<td>3372.02</td>
<td>3397.38</td>
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<tr>
<td>( b (m) )</td>
<td>0.700</td>
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<td>( B_f (m) )</td>
<td>2.200</td>
<td>2.200</td>
<td>2.200</td>
<td>2.200</td>
<td>2.200</td>
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<tr>
<td>( t (m) )</td>
<td>0.550</td>
<td>0.550</td>
<td>0.550</td>
<td>0.550</td>
<td>0.550</td>
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<tr>
<td>( M_{ult, Conc} (Tm) )</td>
<td>4081.00</td>
<td>4982.91</td>
<td>5663.49</td>
<td>6052.14</td>
<td>6110.54</td>
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<td>Min MUlt</td>
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<td>2916.58</td>
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<tr>
<td>MUltimate</td>
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<td>1054.91</td>
<td>1589.71</td>
<td>1874.44</td>
<td>2000.44</td>
</tr>
<tr>
<td>Remarks</td>
<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
</tr>
</tbody>
</table>

Check for Ultimate Shear Strength at Various X

(i) Sections Uncracked in flexure

\[ V_{so} = 0.67bd_s \sqrt{f_{p\, (T/mm^2)} f_t} \]

\[ V_{so} = \text{Ultimate Shear Resistance of the X} \]

- \( b \) = Width of Webs - (2/3 x Duct Diameter) if the Cables are grouted.
- \( d \) = Overall depth
- \( f_t \) = Max principal stress
- \( f_{p\, (T/mm^2)} \) = Stress at c.g at the section due to prestress after inst: loss is accounted.

(ii) Sections Cracked in flexure

\[ V_{so} = 0.037bd_s \sqrt{M_t x V} \]

- \( V_t = (0.37 \sqrt{f_{p\, (T/mm^2)} + 0.8 f_t}) V \]

\[ V_t = \text{Ultimate Shear} & \text{corresponding moment at the section} \]

\[ V_{cr} (min) = 0.1bd_s \sqrt{f_{ck}} \]

- \( V_C \) = Stress at c.g of Tendons, which ever is more.

- \( V_{capacity} = \frac{530 x b \text{ web x db}}{P \sin(q)} \) if the X is Uncracked.

- \( db = 0.8 x \text{Overall Depth or Dist: from comp: face to C.G of Tendons, which ever is more.} \)

Acc. to IRC 18 - 2000 Cl No. 14.1.5 & Table 6.
### Check for Shear Reinforcement Requirement

Check for Shear Reinforcement Requirement :-

(Vide CI:14.1.4 of IRC :18 - 2000.)

<table>
<thead>
<tr>
<th>Sections</th>
<th>1 - 1</th>
<th>3 - 3</th>
<th>5 - 5</th>
<th>7 - 7</th>
<th>9 - 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>b (m)</td>
<td>0.5000</td>
<td>0.1500</td>
<td>0.1500</td>
<td>0.1500</td>
<td>0.1500</td>
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<tr>
<td>$f_{cu}$ (T/m²)</td>
<td>807.299</td>
<td>1104.04</td>
<td>1580.296</td>
<td>1710.094</td>
<td>1704.063</td>
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<td>$f_{ct}$ (T/m²)</td>
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<td>169.706</td>
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<tr>
<td>$V_{ua}$ (T)</td>
<td>222.7238</td>
<td>169.3343</td>
<td>199.2360</td>
<td>203.8136</td>
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<tr>
<td>$d_s$ (m)</td>
<td>1.500</td>
<td>1.76</td>
<td>1.937</td>
<td>2.035</td>
<td>2.050</td>
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<td>$l$ (m)</td>
<td>0.621</td>
<td>0.496</td>
<td>0.496</td>
<td>0.496</td>
<td>0.496</td>
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<tr>
<td>y = $V_b$ (m)</td>
<td>1.100</td>
<td>1.06</td>
<td>1.057</td>
<td>1.057</td>
<td>1.057</td>
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<td>$r$ (T/m²)</td>
<td>479.096</td>
<td>220.32</td>
<td>-298.758</td>
<td>-597.812</td>
<td>-641.560</td>
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<td>$r_b$ (T/m²)</td>
<td>1142.460</td>
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<td>$V_{cr}$ (T)</td>
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<td>978.273</td>
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<td>355.753</td>
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<td>247.4</td>
<td>143.1</td>
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<td>$V_{cr}$ (T)</td>
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Check for Shear Reinforcement Requirement :-

(Vide CI:14.1.4 of IRC :18 - 2000.)

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<th>9 - 9</th>
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<tr>
<td>$V_{ua}$ (T)</td>
<td>355.7525</td>
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<td>143.063</td>
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<td>62.568</td>
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<td>Spacing (mm)</td>
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<td>150</td>
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<td>$V_{cr}$ (T)</td>
<td>4.78</td>
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<td>3.23</td>
<td>1.47</td>
<td>1.25</td>
<td>1.02</td>
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<td>Remarks</td>
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<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
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</table>
**EFFECTS OF DIFFERENTIAL SHRINKAGE**

It is generally assumed that the time lag between transfer of prestress and casting of deck slab will be about 30 days.

Differential shrinkage strain, $\varepsilon_{sh} = 0.0001$

Force induced by differential shrinkage, $F = \eta \cdot A \cdot E \cdot \varepsilon_{sh}$ where $A = 0.55 \text{ m}^2$,

$E = 5000 \sqrt{50}$

$= 35355 \text{ MPa}$

$= 3535533.91 \text{ Kg/cm}^2$

$\eta = 0.43$ (As per BS:5400)

$E = 83.6 \text{ T}$

Eccentricity of the above force from top of the composite section $= 0.125 \text{ m}$

$Y_t$ (Distance of the NA of composite section from top) $= 0.937 \text{ m}$

Eccentricity of the above force CG of the composite section $= 0.812 \text{ m}$

Moment due to above force $= 67.90 \text{ Tm}$

$Z_t = 564666.67 \text{ cm}^2$

$Z_b = 564666.67 \text{ cm}^2$

$Z_j = 1549750.00 \text{ cm}^2$

$A = 15400.00 \text{ cm}^2$

Stresses induced due to differential shrinkage

$\sigma_t = \frac{83615.4}{15400.0} + \frac{6790333.7}{564666.7} = 17.45 \text{ Kg/m}^2$

$\sigma_b = \frac{83615.4}{15400.0} - \frac{6790333.7}{564666.7} = -6.60 \text{ Kg/m}^2$

$\sigma_j = \frac{83615.4}{15400.0} + \frac{6790333.7}{1549750.0} = 9.81 \text{ Kg/m}^2$

Tensile stress in deck slab $F = \frac{-83615.4}{5500.0} = -15.2 \text{ Kg/m}^2$

**Stresses due to differential shrinkage**
Analysis for temperature stresses

The analysis for temperature stresses is carried out as outlined in IRC 6-2000
Concrete bridge practice by V K Raina (Analysis design and economics) is referred

Stress calculation due to variation in temperature
Calculation of eigen stresses (Positive temperature difference)

Geometric section

Concrete mix = M45
fck = 45 Mpa

E (concrete) = 3.35E+04

Coefficient of thermal expansion of concrete = 1.17E-05 /deg C

Calculation of thermal strain and gradient due to variation in temperature along the cross section
<table>
<thead>
<tr>
<th>Zone</th>
<th>Area (m²)</th>
<th>Y (m)</th>
<th>AY (m³)</th>
<th>AY^2 (m^4)</th>
<th>t (deg)</th>
<th>At (m)</th>
<th>AY (Deg)</th>
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</thead>
<tbody>
<tr>
<td>a</td>
<td>0.33</td>
<td>0.079</td>
<td>0.02475</td>
<td>0.00185625</td>
<td>10.9</td>
<td>3.597</td>
<td>0.269775</td>
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<td>a1</td>
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<td>0.2</td>
<td>0.044</td>
<td>0.0088</td>
<td>3.2</td>
<td>0.704</td>
<td>0.1408</td>
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<td>b</td>
<td>0.105</td>
<td>0.325</td>
<td>0.034125</td>
<td>0.011990625</td>
<td>1.20</td>
<td>0.126</td>
<td>0.04095</td>
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<tr>
<td>b1</td>
<td>0.324</td>
<td>0.85</td>
<td>0.2754</td>
<td>0.23409</td>
<td>0.0</td>
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<td>0</td>
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<td>c</td>
<td>0.324</td>
<td>1.75</td>
<td>0.567</td>
<td>0.99225</td>
<td>0.0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>d</td>
<td>0.07</td>
<td>2.1</td>
<td>0.147</td>
<td>0.3087</td>
<td>0.0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>d1</td>
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<td>2.375</td>
<td>0.249375</td>
<td>0.59225625</td>
<td>1.05</td>
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<tr>
<td>Sum</td>
<td>1.48E+00</td>
<td>1.3417E+00</td>
<td>2.1490525</td>
<td>4.53725</td>
<td>0.71336875</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For equilibrium

\[ \alpha \cdot A + \beta \cdot AY = \alpha A t \quad (1) \]
\[ \alpha \cdot AY - \beta \cdot AY^2 = \alpha A Yt \quad (2) \]

\[ \begin{align*}
\alpha & \cdot 1.48E+00 - \theta \cdot 1.34E+00 = 1.17E-05 \times 4.54 \\
\alpha & \cdot 2.15E+00 - \theta \cdot 1.34E+00 = 1.17E-05 \times 0.71 \\
\theta & \cdot 0.9077E+06 = 3.59E-05 \quad (3) \\
\theta & \cdot 1.60179816 = 6.22E-05 \quad (4) \\
\end{align*} \]

Calculation of thermal stresses

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Y (m)</th>
<th>t (deg)</th>
<th>At (m)</th>
<th>( \phi = \frac{\text{Ec} \cdot (\alpha \cdot A + \beta \cdot AY - \alpha A t)}{1} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00E+00</td>
<td>17.80</td>
<td>2.08E-04</td>
<td>-4.48E+00</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>1.07E-05</td>
<td>2.80E-05</td>
<td>1.21E+00</td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td>4.92E-05</td>
<td>0.00E+00</td>
<td>8.57E-01</td>
</tr>
<tr>
<td>4</td>
<td>2.05</td>
<td>8.77E-05</td>
<td>0.00E+00</td>
<td>-4.35E-01</td>
</tr>
<tr>
<td>5</td>
<td>2.3</td>
<td>9.84E-05</td>
<td>2.48E-05</td>
<td>-1.62E+00</td>
</tr>
</tbody>
</table>
Calculation of eigen stresses (Reverse temperarature difference)

**Equivalent I - section for running section**

Concrete mix = M50

\[ f_{ck} = 50 \text{ Mpa} \]

\[ E_{\text{concrete}} = 5000f_{ck} = 3.54E+04 \]

Coefficient of thermal expansion of concrete = 1.17E-05 /deg C

Calculation of thermal strain and gradient due to variation in temperature along the cross section

<table>
<thead>
<tr>
<th>Zone</th>
<th>Area (m^2)</th>
<th>Y (m)</th>
<th>AY (m^4)</th>
<th>AY^2 (m^6)</th>
<th>( t ) (Deg)</th>
<th>At (m)</th>
<th>AY (m^4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0.55</td>
<td>0.125</td>
<td>0.06875</td>
<td>0.006599375</td>
<td>5.65</td>
<td>0.1075</td>
<td>0.3864375</td>
</tr>
<tr>
<td>b</td>
<td>0.105</td>
<td>0.325</td>
<td>0.034125</td>
<td>0.01099625</td>
<td>0.439</td>
<td>0.01429885</td>
<td>0.3458375</td>
</tr>
<tr>
<td>b1</td>
<td>0.018</td>
<td>0.425</td>
<td>0.00765</td>
<td>0.0003125</td>
<td>0.0001575</td>
<td>0.00068635</td>
<td>0.00000025</td>
</tr>
<tr>
<td>b2</td>
<td>0.252</td>
<td>0.8</td>
<td>0.2016</td>
<td>0.16128</td>
<td>0.000000</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>c1</td>
<td>0.252</td>
<td>1.5</td>
<td>0.378</td>
<td>0.567</td>
<td>0.000000</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>c2</td>
<td>0.072</td>
<td>1.95</td>
<td>0.1404</td>
<td>0.27398</td>
<td>0.0288</td>
<td>0.05616</td>
<td>0</td>
</tr>
<tr>
<td>c3</td>
<td>0</td>
<td>2.05</td>
<td>0</td>
<td>0.80</td>
<td>0.000000</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>d</td>
<td>0.175</td>
<td>2.325</td>
<td>0.406875</td>
<td>0.94586375</td>
<td>3.7</td>
<td>0.6475</td>
<td>1.5054375</td>
</tr>
<tr>
<td>Sum</td>
<td>1.428+00</td>
<td>1.2374+00</td>
<td>1.97988</td>
<td>3.8313125</td>
<td>1.965534683</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Calculation of thermal strain and gradient due to variation in temperature along the cross section

\[ \text{At}_{Y} = \frac{1}{2} \sum_{i=1}^{n} \Delta T_{i} \Delta Y_{i} \]
For equilibrium

\[ \begin{align*}
\sum \Delta A - \theta \sum \Delta Y &= \Delta A t \\
\sum \Delta Y - \theta \Delta Y^2 &= \Delta A Yt
\end{align*} \]  

(1) \hspace{1cm} (2)

\[
\begin{align*}
\theta &= 1.42 \times 10^{-2} \Rightarrow 1.24 \times 10^{-2} = 1.17 \times 10^{-5} \times 3.83 \\
\theta &= 1.24 \times 10^{-2} \Rightarrow 1.97 \times 10^{-2} = 1.17 \times 10^{-5} \times 1.97 \\
\theta \times 1.00 \Rightarrow 0.86896067 = 3.15 \times 10^{-5} \\
\theta \times 1.00 \Rightarrow 1.59283983 = 1.86 \times 10^{-5}
\end{align*}
\]

(3) \hspace{1cm} (4)

\[
\begin{align*}
\theta &= 0.72387915 = 1.29 \times 10^{-5} \\
\theta &= 1.78 \times 10^{-3} \\
\theta &= 4.70 \times 10^{-5}
\end{align*}
\]

Calculation of thermal stresses

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Y (m)</th>
<th>Y θ</th>
<th>t</th>
<th>α t</th>
<th>Δε = Ec (co. - Y θ - α t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00E+00</td>
<td>0.00E+00</td>
<td>10.00 deg</td>
<td>1.24E-04</td>
<td>2.58E+00</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>4.55E-05</td>
<td>0.44 deg</td>
<td>5.119E-06</td>
<td>1.25E+00</td>
</tr>
<tr>
<td>3</td>
<td>1.15</td>
<td>2.05E-05</td>
<td>0.00 deg</td>
<td>0.00E+00</td>
<td>8.88E-01</td>
</tr>
<tr>
<td>4</td>
<td>2.05</td>
<td>3.65E-05</td>
<td>0.00 deg</td>
<td>0.00E+00</td>
<td>3.50E-01</td>
</tr>
<tr>
<td>5</td>
<td>2.31</td>
<td>4.10E-05</td>
<td>6.0 deg</td>
<td>7.72E-05</td>
<td>2.39E+00</td>
</tr>
</tbody>
</table>
Design for Bursting Tensile Force:

(Vide Cl:17.2 on page no -35 of I.R.C:18-2000)

\[
\begin{align*}
\text{2 Yo} &= \text{Width of End Block} = 800 \text{ mm} \\
\therefore \text{Yo} &= 400 \text{ mm} \\
\text{2 YPo} &= \text{Width of Bearing Plate} = 375 \text{ mm assumed} \\
\therefore \text{YPo} &= 187.5 \text{ mm} \\
\text{Pk} &= \text{Force at anchorage} = 454.212 \text{ T} \\
\text{YPo} &= 187.5 \\
\text{YPo} &= 0.469 \\
\text{Yo} &= 400 \\
\frac{F_{bst}}{\text{Pk}} &= 0.179375 \text{ (From Table- 8) for corresponding } \frac{\text{YPo}}{\text{Yo}} \\
\therefore F_{bst} &= 0.1794 \times 454.212 = 81.474 \text{ T} \\
\text{Calculation of area of steel required.} \\
\text{Permissible tensile stress in H.Y.S.D steel} \\
&= 0.87 \times 4150 = 3610.5 \text{ Kg/cm}^2 \\
\text{Ast reqd} &= \frac{81.4742775 \times 1000}{3610.5} = 22.566 \text{ cm}^2 \\
\text{Provide 16 mm dia bars, 4 nos in 3 layers at the front face of anchoring cone.} \\
\text{Ast prov} &= 24.127 \text{ cm}^2 \text{ Safe}
\end{align*}
\]
Appendix D-2: Super Structure Design - Cross Girder
STAAD PLANE cross-girder
START JOB INFORMATION
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
joint coordinates
1 0 0 0; 2 2.2 0 0; 3 3.3 0 0; 4 4.4 0 0; 5 6.6 0 0
6 7.7 0 0; 7 8.8 0 0; 8 11 0 0; 9 12.1 0 0; 10 13.2 0 0;
11 15.4 0 0;
member incidences
1 1 2 10
member property indian
1 to 10 pri ax 2.56 ix 0.001 iz 0.57
constants
e concrete all
poi concrete all
supports
3 6 9 pinned
load 1
member load
******cross girder self weight
1 to 10 uni gy -55
******girder load
joint load
1 2 4 5 7 8 10 11 fy -531
******slab load
1 11 fy -262
2 4 5 7 8 10 fy -326
***SIDL
1 fy -216
2 fy -118
4 fy -88
5 fy -92
7 fy -92
8 fy -90
10 fy -118
11 fy -217
****live load
1 fy -145
2 fy -267
4 fy -576
5 fy -395
7 fy -320
8 fy -689
10 fy -294
11 fy -165
perform analysis
PRINT MAXFORCES ENVELOPE
print member forces
print support reactions
print displacements
FINISH
Cross girder – Analysis model

BMD-Cross girder

Max: 2718.1 kNm
Max: 5572.88 kNm
Max: 5572.88 kNm
Max: 2163.27 kNm
Max: -874.169 kNm
Max: -912.089 kNm
Max: 2163.27 kNm
Max: 5473.88 kNm
Max: 5473.88 kNm
Max: 2671.9 kNm
Max: 2718.1 kNm
DESIGN OF END CROSS GIRDER

Design forces are taken from STAAD

<table>
<thead>
<tr>
<th>SECTION</th>
<th>1</th>
<th>2</th>
<th>BM</th>
<th>NOTE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUM</td>
<td>554.00</td>
<td>-134.00</td>
<td>+ve</td>
<td>554 Tm</td>
</tr>
<tr>
<td>SUM</td>
<td>269.00</td>
<td>-ve</td>
<td>134 Tm</td>
<td></td>
</tr>
</tbody>
</table>

- ve = Hogging
- ve = Sagging

Torsional moment = 0
Shear = 269 T

<table>
<thead>
<tr>
<th>Grade of Concrete</th>
<th>= 500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eff. depth reqd. (mm)</td>
<td>= 1479</td>
</tr>
<tr>
<td>Eff. depth provided (mm)</td>
<td>= 2384</td>
</tr>
<tr>
<td>Total depth provided (mm)</td>
<td>= 2450</td>
</tr>
<tr>
<td>Dia of bar provided (mm)</td>
<td>= 32</td>
</tr>
<tr>
<td>No of bars reqd.</td>
<td>= 25</td>
</tr>
<tr>
<td>Shear Stress Kg/cm²</td>
<td>= 14.10</td>
</tr>
<tr>
<td>Allow Shear Stress Kg/cm²</td>
<td>= 4.04</td>
</tr>
</tbody>
</table>

Safe

Shear Reinft Provided
Dia of Bar provided = 12
No of legs = 6
Spacing = 125

Check for effective depth at section 1-1

<table>
<thead>
<tr>
<th>Total Moment</th>
<th>= 554.00 Tm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear due to</td>
<td>Total Shear</td>
</tr>
</tbody>
</table>

Check for eff. Depth

\[
\text{deff} = \sqrt{\frac{554.000 \times 100000}{31.67 \times 80}}
\]

Bar Size = 32 mm
Cover = 50 mm

Depth provided at 1-1 = 2450 mm
Deff prov. = 2384 cms

Ast Req'd = 554.000 x 100000 = 198.57 cm²

No. of bars reqd. = 25
Check for effective depth at section 2-2

Moments due to

<table>
<thead>
<tr>
<th>Moment</th>
<th>134.00 Tm</th>
<th>Total Shear</th>
<th>269.00 T</th>
</tr>
</thead>
</table>

Check for eff. Depth

\[ \sqrt{\frac{134,000}{31.67 \times 80}} = \frac{269.00}{72.72 \text{ cms}} \]

Depth provided at 2-2

\[ \text{Total Moment} = 134.00 \times 100000 \]

\[ \text{Bar Size} = 32 \text{ mm} \]

\[ \text{Cover} = 50 \text{ mm} \]

\[ \text{Safe} \]

Ast Req'd = 134.000 \times 100000 = 48.03 \text{ cm}^2

No. of bars reqd. = 6

2) Maximum Permissible Shear Stress :-

\[ \tau = \frac{V}{B \times d} \]  
\[ V = \text{The design shear across the section} \]

\[ B = \text{Breadth of slab} \]

\[ d = \text{Effective depth of the section} \]

\[ \tau = \frac{269.000 \times 1000}{80 \times 238.4} = 14.1044 \text{ Kg/cm}^2 \]

Maximum Permissible Shear Stress :-

\[ \tau_{\text{max}} = 2.50 \text{ Mpa} \]

\[ \tau_{\text{max perm}} = 25.0 \text{ Kg/cm}^2 \]

Calculation of permissible Shear Stress :-

\[ \text{V}_{\text{c}} = \text{4.04 Kg/cm}^2 \]

3) Shear Reinforcement Reqd :-

\[ \text{Shear reinforcement is provided to carry a shear equivalent to} \]

\[ \text{Design shear} \]

\[ V_s = (V - \tau c bd) \]

\[ A_{sw} = \frac{V}{s x d} \]

\[ V = 269.00 \text{ T} \]

\[ s, \text{ spacing} = 125 \text{ mm} \]

\[ A_{sw} = 191.95 \times 1000 \times 12.5 = 5.0 \text{ cm}^2 \]

Using 12 mm dia stirrups

\[ A_{sw} = 6.8 \text{ cm}^2 > 5.0 \text{ cm}^2 \]

Hence O.K.
DESIGN OF INTERMEDIATE DIAPHARM:

The output of BM and SF is taken from STAAD Analysis for the worst effects on the intermediate diaphragm.

| SECTION | 1 | 2 |
| SUM | 153.00 | 125.00 |
| SUM | 47.00 |

Hogging = 153 Tm
Sagging = 125 Tm
Shear = 47 T

| Torssional moment | 0 |

Grade of Concrete = 0

eff. depth reqd. (mm) = 1269

eff. depth provided (mm) = 2090

Total depth provided (mm) = 2150

Dia of bar provided (mm) = 25

No of bars reqd. = 19

Shear Stress Kg/cm² = 7.50

Allow Shear Stress Kg/cm² = 4.20

Safe

Dia of Bar provided = 12

No of legs = 2

Spacing = 125

Check for effective depth at section 1-1

Total Moment = 153.00 Tm
Total Shear = 47.00 T

Check for eff: Depth

deff = \sqrt{\frac{153.00 \times 100000}{31.67 \times 30}}

= 126.89 cms

Bar Size = 20 mm

Cover = 50 mm

Depth provided at 1-1 = 2150 mm

def prov: = 209 cms

Safe

Ast Reqd = 153.000 x 100000

= 58.62 cm²

2000 x 1.00 x 130.5

No. of bars reqd. = 19
Check for effective depth at section 2 - 2

Total Moment = 125.00 Tn

Check for eff. Depth

\[ \text{Depth provided at 2-2} = \frac{125.000 \times 100000}{31.67 \times 30} \text{ cms} \]

Bar Size = 20 mm
Cover = 50 mm

Depth provided at 2-2 = 2150 mm

def depth prov. = 209 cms

Safe Ast Reqd = \( \frac{125.000 \times 100000}{2000 \times 1.00 \times 130.5} \) = 47.89 cm²

No. of bars reqd. = 16

2) Maximum Permissible Shear Stress :-

Shear stress,

\[ \tau = \frac{V}{B \times d} \]

\( V \) = The design shear across the section
\( d \) = Effective depth of the section
\( B \) = Breadth of slab

\[ \therefore \tau = \frac{47.000}{300} \times 209 = 7.4960 \text{ Kg/cm}^2 \]

Maximum Permissible Shear Stress :-

\[ \tau_{\text{max}} = 2.50 \text{ Mpa for M40 and above concrete grade} \]

\[ \tau_{\text{max perm}} = 25.0 \text{ Kg/cm}^2 \]

Calculation of permissible shear Stress :-

\[ \frac{100 \times A_s}{b d} = 2.740 \]

\[ \therefore \tau_c = 4.2 \text{ Kg/cm}^2 \text{ value from table is } 2.740125171 \text{ value is } 4.2 \text{ Kg/cm}^2 \text{ Required} \]

3) Shear Reinforcement Reqd :-

Shear reinforcement is provided to carry a shear equivalent to

Design shear \( V_s = (V - \tau_c b d) \) = (47.00 - 26.33) T

\( A_s = \frac{V}{s \times x \times d} \) = \( \frac{47.00}{2000 \times 209.0} \)

\( s \), spacing = 125 mm (assumed)

\[ \therefore A_s = \frac{20.67}{1000 \times 12.5} = 0.6 \text{ cm}^2 \]

Using 12 mm dia stirrups 2 Legged at spacing 125 mm.

\[ A_s = 2.3 \text{ cm}^2 > 0.6 \text{ cm}^2 \]

Hence O.K.
Appendix D-3: Design of Substructure & Foundation
Design of Trestle Abutment and Pile Foundation

Design Data for Substructure:

<table>
<thead>
<tr>
<th>Type</th>
<th>Simply supported Precast Prestressed Girders and RCC slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>C/C Distance betw. piers</td>
<td>40000 mm</td>
</tr>
<tr>
<td>Carriageway Width</td>
<td>7500 mm</td>
</tr>
<tr>
<td>Overall Width of Deck.</td>
<td>17000 mm</td>
</tr>
<tr>
<td>Width of Crash Barrier</td>
<td>500 mm Both sides</td>
</tr>
<tr>
<td>Height of Crash Barrier</td>
<td>1050 mm</td>
</tr>
<tr>
<td>No of Bearings</td>
<td>3</td>
</tr>
<tr>
<td>Overall Depth of I.Girders</td>
<td>2200 mm</td>
</tr>
<tr>
<td>Depth of Deck Slab</td>
<td>250 mm</td>
</tr>
</tbody>
</table>

Design Data for Pier:

| Formation level at abutment locc       | 212.600 m                                                |
| R.L at abutment cap top                | 209.448 m                                                |
| Existing Road level                    | 207.656 m                                                |
| Pile cap top below existing road       | 500 mm                                                   |
| R.L at Pile cap top                    | 207.16 m                                                 |
| R.L at Pile cap bottom                 | 205.36 m                                                 |
| Depth of Pile below pilecap            | 25000 mm                                                 |
| Founding level for Piles.              | 180.36 m                                                 |
| Overall Height of Substructure         | 2292 mm                                                  |
| Diameter of pier                       | 1300 mm                                                  |
| Transverse width of pier               | 1300 mm                                                  |
| No of piers                            | 3                                                        |
| C/C distance between piers             | 3500 mm                                                  |
| Pier Cap Width in Long Dirn.           | 1600 mm                                                  |
| Pier Cap Length in Trans Dirn.         | 12290 mm                                                 |
| Straight Depth of Pier Cap             | 1000 mm                                                  |
| Type of Bearing                        | POT PTFE BEARING                                         |
| Size of Pedestals                      | 600 x 600 x 350                                          |
| Distance between Pedestals             | 4400 mm                                                  |
| Longitudinal width of pile cap         | 5100 mm                                                  |
| Transverse width of pile cap           | 8700 mm                                                  |
| Straight Depth of pile cap             | 1800 mm                                                  |
| Varying Depth of pile cap              | 0 mm                                                     |
| P.C.C Projections                      | 150 mm                                                   |
| Diameter of Pile                       | 1200 mm                                                  |
| Distance between Piles in transverse   | 3600 mm                                                  |
| Distance between Piles in longitudinal | 3600 mm                                                  |
| No of Piles                            | 6                                                        |
| Edge projection in longitudinal        | 150 mm                                                   |
| Edge projection in transverse          | 150 mm                                                   |
| Grade of Concrete                      | M 50                                                     |
| Permissible flexural stress            | 16.67 N/mm²                                               |
| Grade of Steel                         | Fe - 415                                                  |
| Permissible tensile stress             | 200 N/mm²                                                 |
| Total Height of Pier                   | 1292 mm                                                   |
| Density of Concrete                    | 24 kN/m³                                                   |
| Density of Concrete for PSC Girder     | 25 kN/m³                                                   |

References:

I.R.C.-21: 2000 - Permissible Stresses
I.R.C.-78: 2000 - Sub Structure and Foundation
Load Calculations:

1) DEAD LOADS
   Total Load = 7184.00 kN
2) SIDL
   - Wearing coat load = 429.00 kN
   - Median = 144.00 kN
   - Crash barrier load = 249.60 kN
   SIDL = 822.60 kN

3) LIVE LOAD
   Impact Factor = 1 considering 50% reduction as per clause 211.7 IRC 6-2000
   70 R

<table>
<thead>
<tr>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>170</td>
<td>170</td>
<td>170</td>
</tr>
<tr>
<td>170</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>80</td>
<td>3.05</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>866.6</td>
<td>38.4</td>
<td>133.4323</td>
</tr>
</tbody>
</table>

   \[ R_C = \frac{(170 \times 38.4 + 170 \times 37.03 + 170 \times 33.98 + 170 \times 32.61 + 120 \times 30.48 + 120 \times 28.96 + 80 \times 25)}{38.4} \]

   \[ R_C = 866.5677 \text{ kN} \]
   \[ R_D = 133.432 \text{ kN} \]

   70 R one lane = 866.57 kN
   Class A One lane = 425.60 kN

   Class A One lane

<table>
<thead>
<tr>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>114</td>
<td>38.4</td>
</tr>
<tr>
<td>1.2</td>
<td>23.9</td>
</tr>
<tr>
<td>425.60</td>
<td></td>
</tr>
</tbody>
</table>

   Class A One lane = 425.60 kN
   10% reduction of live load is considered for three lane as per Clause 208
   Class 70R = 866.57 kN
   Class A two lane = 851.21 kN
   Critical live load = 866.57 kN

   The design for critical live load case is done here and checked for all other cases.

Calculation of Dead Load from Sub - Structure:

1) From Pedestals:
   - Volume of 1 Pedestal = 0.6 x 0.6 x 0.35 = 0.126 m³
   - No of Pedestals = 3 x 1 = 3 Nos
   - Total Volume = 0.126 x 3 = 0.38 m³
   - Total Load = 0.378 x 24 = 9.07 kN

2) From Bed - Block:
   = \{ 1.6 x 1 \} = 1.6 m³

   Total C/S Area in longitudinal Dirn

   Width in the Transverse Dirn = 12.29 m
   - Total Volume = 1.60 x 12.29 = 19.66 m³
   - Total Load = 19.66 x 24 = 471.94 kN

3) From Pier:
   - Diameter of Pier = 1.3 m
   - Area of Pier = 1.327 m²
   - Height of Pier = 1.292 m
   - Total Volume = 1.327 x 1.3 = 1.714 m³
   - Total Load = 1.714 x 3 x 24 = 123.43 kN
4) From Pile Cap -:
   Volume of footing = (5.1 x 8.7 x 1.8) m³ = 79.866 m³
   Dead Load = 79.866 x 24 = 1916.78 kN
5) From Pile -:
   Diameter of Pile = 1.2 m
   Depth of Pier with Circular Xn = 25 m
   C/S Area of Circular Pier = 1.131 m²
   No of Piles = 6
   Total Volume of Concrete = 169.646 m³
   Total Load = 169.646 x 24 = 4071.50 kN
5) From Dirt Wall -:
   Volume of dirt wall = 3.152 x 12.0 x 0.30 = 11.349 m³
   Total Load = 11.349 x 24 = 272.376 kN
Options
A) Both Spans on
   Calculation of Longitudinal Moments at start of Pier Flaring & Pier Base & Pilecap Base
1) Due to Braking
   b) Braking:
      (Vide :- cl 214.2 (c) & (b) of I.R.C : 6 - 2000.)
      Braking: Since the movement of bearing under the girders on one side is restricted to move in the longitudinal direction half the effect of braking is considered in the design.
      1) 20 % of Ist Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.
      2) 20 % of Ist Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge. + 5 % of Loads on the lanes exceeding Two.
      3 Lanes of Class A Wheeled Vehicles:
      Total Load of 1 Vehicle = 554 kN one span
      Braking Force = 110.8 kN
      1 Lanes of Class 70R Wheeled Vehicles. + Class A one lane
      Total Load of 70R Vehicle = 1000 kN one span
      Total Load of Cl A Vehicle = 554 kN
      Braking Force = 200 kN
      Max Braking Force = 200 kN
      Vertical reaction due to braking = 200(1.2+0.075+2.45)/(40-0.6) = 17.5888 kN
      Longitudinal moment due to vertical load of braking
      Longitudinal Eccentricity = 0.45
      Moment due to long.Eccentricity = 7.91497 kNm
2) Due to Temp & Shrinkage of Bearings:
   (ii) FH + m (Rg + Rq) = 543.66 kN
      Total Force = 543.66 kN
      Moment at Pile Cap top = 544 x 3.69 = 2006.91 kNm
      Moment at Pile Cap bottom = 544 x 5.49 = 2985.50 kNm
3) Moment due to Longitudinal Eccentricity
   Longitudinal Eccentricity = 0.80 m
   Normal Case
   Seismic Case
   Due to DL = 0 kNm 0 kNm
   Due to SIDL = 0 kNm 0 kNm
   Due to LL = 693 kNm 347 kNm
   Moment = 693 kNm 347 kNm

Due to Live Load
   Transverse moment about the centre of the pier is calculated by finding the eccentricity.
Class 70R 1 lane

Moment in Transverse Dirn. = 4683.80 kNm

Class A 2 lane.

Moment in Transverse Dirn. = 5778.09 kNm

70 R One lane - both carriage ways

Moment in Transverse Direction = 2001.77 kNm

Class A two lane - both carriage ways

Moment in Transverse Direction = 1634.21 kNm

Critical Moment in Transverse Direction = 5778.09 kNm

Axial Load

Pile Cap Top = 7184 + 822.6 + 866.57 + 9.08 + 471.94 + 123.43 + 17.59
Pile Cap Bottom = 7184 + 822.6 + 866.57 + 9.08 + 471.94 + 123.43 + 1916.79

Longitudinal moment

Pile Cap Top = 2006.92 + 693.26
Pile Cap Bottom = 2985.51 + 693.26

Transverse moment

Pile Cap Top = 5778.1
Pile Cap Bottom = 5778.1

**Summary of Axial Loads & Moments**

<table>
<thead>
<tr>
<th>Descriptions Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>9750.0</td>
</tr>
<tr>
<td>Ml (Tm)</td>
<td>2700.2</td>
</tr>
<tr>
<td>Ml (Tm)</td>
<td>5778.1</td>
</tr>
</tbody>
</table>

C) Both Spans on Under Seismic in Longitudinal Direction.  

Case ( ii )

As per Modified Clause 222

\[ F_{eq} = A_h (\text{Dead load} + \text{Appropriate Live Load}) \]

\[ F_{eq} = \text{Seismic force to be resisted} \]

\[ A_h = \text{Horizontal Seismic coefficient} \]

\[ = \left( \frac{Z}{2} \right) \left( \frac{S_a}{g} \right) \]

Zone No = IV

Zone Factor, Z = 0.24

\[ S_a/g = \text{Average acceleration coefficient} \]

Response Modification factor, R = 3.3

Importance Factor, I = 1.2

\[ T = \frac{2D}{\sqrt{1000F}} \]

Dead load of the super structure, and appropriate live load in kN, D = 8006.60 kN

\[ F = \text{Horizontal force in kN required to be applied at the centre mass of the super structure for one mm horizontal deflection at the top of the pier / abutment along the considered direction of horizontal force.} \]

\[ = \left( \frac{3EI\delta}{F} \right) \]

Modulus of Elasticity of concrete, E = 3.54E+07 kN/m²

lxx = 0.42 m

Deflection, \( \delta \) = 0.001 m

Distance from bottom to the centre of mass of super structure, y = 0.97 m
Design of seismic force in longitudinal direction

Seismic force in transverse direction

Summary of Axial Loads & Moments
D) Both Spans on Under Seismic in Transverse Direction.

Case (iii)

Design Seismic force in trans
\[ D_{Feq} \] = 892.35 kN
Resultant force in trans dim
\[ \mathbf{R} \] = 1332.859 kN
CG of loads
\[ \mathbf{CG} \] = 3.492861 m

Moment, \( M_{eqy} \) at Pile Cap top
\[ M_{eqy} \] = 4655.492 kN
Moment, \( M_{eqy} \) at Pile Cap
\[ M_{eqy} \] = 7054.638 kN
Axial load is same as that of Case iii

Longitudinal moment
Pile Cap Top
\[ \mathbf{P} \times \mathbf{CG} = \{100 \times 0.5 + 46.8\} \times 2 \times 3.7 + 4655.49 \]
Pile Cap Bottom
\[ \mathbf{P} \times \mathbf{CG} = \{100 \times 0.5 + 46.8\} \times 2 \times 5.49 + 7054.63 \]

Transverse moment
Pile Cap Top
\[ \mathbf{P} \times \mathbf{CG} = 5778.09 \times 0.5 + 4655.5 \]
Pile Cap Bottom
\[ \mathbf{P} \times \mathbf{CG} = 5778.09 \times 0.5 + 7054.64 \]
As per Table 1 Load Combination, 50 % of LL is considered in seismic case

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Description</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>10441.34</td>
<td>12339.28</td>
</tr>
<tr>
<td>( M_y )</td>
<td>6744.49</td>
<td>9993.22</td>
</tr>
<tr>
<td>( M_z )</td>
<td>7544.54</td>
<td>9993.68</td>
</tr>
</tbody>
</table>

G) Service condition with Wind in Transverse direction

(Vide cl: 212.1 of I.R.C:6-2000)

Height of the exposed surface above ground level = 4.94 m
Exposed depth of C/Barrier & Superstructure = 2.45 m + 1.05 m = 3.50 m

Due to Crash Barrier
Avg Height of Crash barrier from GL = 5.47 m
Intensity of Wind pressure corrs : to height = 463.70 Kg/m²
Average Exposed Length = 40.00 m
Effective area of crash barrier = 42.00 m²
Force = \( P \times A \times G \times C_d \)
\[ G = 2 \]
\[ C_d = 1.30 \]
Force = 506.36 kN

Due to Deck Slab and girder
Average ht of deck slab and girder from GL = 2.67 m
Intensity of Wind pressure corrs : to height = 463.70 Kg/m²
Effective area of deck slab + girder = 98.00 m²
Force = \( P \times A \times G \times C_d \) kN
\[ G = 2 \]
\[ C_d = 1.95 \]
Force = 177.23 kN

Live Load
Effective length = 40.00 m
Depth = 1.30 m
Height of the exposed surface above ground level = 6.44 m
Area = 52.00 m²
Pressure = 463.70 N/m²
Force = \( P \times A \times G \times C_d \)
\[ G = 2 \]
\[ C_d = 1.20 \]
Force = 57.87 kN

Due to Pier
Average ht of pier and pier cap from GL = 1.15 m
Intensity of Wind pressure corrs : to height = 463.70 kg/m²
Effective area of pier and pier cap = 1.60 m²
Force = \( P \times A \times G \times C_d \) kN
\[ G = 2 \]
Cd = 0.50

Force = 0.74 kN

Total transverse force = 742.20 kN

Transverse Moment due to wind
Pile Cap Top = 3615.62 kNm
Pile Cap Bottom = 4017.76 kNm

Longitudinal Force
Crash barrier = 25 % of trans force = 126.59 kN
Superstructure = 25 % of trans force = 44.31 kN
On Live load = 25 % of trans force = 14.47 kN
Substructure = 25 % of trans force = 0.19

Total longitudinal force = 185.55 kN

Longitudinal Moment due to wind
Pile Cap Top = 829.48 kNm
Pile Cap Bottom = 1237.89 kNm

Vertical load = \( P_z \) \( A \) \( 3GCL \)

\[ P_z = \frac{463.70 \text{ kN/m}^2 \times 680.00}{2.00 \times 0.75} = 472.97 \text{ kN} \]

Axial loads and Longitudinal Moments are same as Case 1

**Summary of Axial Loads & Moments**

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pilecap Top</th>
<th>Pile cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>10222.95</td>
<td>2139.73</td>
</tr>
<tr>
<td>( M_a ) (Tm)</td>
<td>3529.65</td>
<td>4916.65</td>
</tr>
<tr>
<td>( M_t ) (Tm)</td>
<td>9393.71</td>
<td>9795.85</td>
</tr>
</tbody>
</table>

**Case ( iv b )**

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pilecap Top</th>
<th>Pile cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>9277.00</td>
<td>11193.79</td>
</tr>
<tr>
<td>( M_a ) (Tm)</td>
<td>3529.65</td>
<td>4916.65</td>
</tr>
<tr>
<td>( M_t ) (Tm)</td>
<td>9393.71</td>
<td>9795.85</td>
</tr>
</tbody>
</table>

**Effect of collision in longitudinal direction**

Axial load
Pile Cap Top = 7184+822.6+9.072+471.936+123.425 = 7184+822.6+9.072+471.936+123.425 + 1916.78
Pile Cap Bottom = 7184+822.6+9.072+471.936+123.425 + 1916.78

Longitudinal moment
Collision load in longitudinal direction = 500.00 kN acting at 1.5m above carriageway level of service road
Pile Cap Top = 500x(1.5+0.5) = 1000.00 kNm
Pile Cap Bottom = 500x(1.5+2.3000000) = 1900.00 kNm

Transverse moment
Pile Cap Top = 0.00 kNm
Pile Cap Bottom = 0.00 kNm

**Summary of Axial Loads & Moments**

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Footing Top</th>
<th>Footing bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>8611.03</td>
<td>10527.82</td>
</tr>
<tr>
<td>( M_a ) (Tm)</td>
<td>1000.00</td>
<td>1900.00</td>
</tr>
<tr>
<td>( M_t ) (Tm)</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
J) Effect of collision in transverse direction  

**Case vi**

**Axial load**
- Pile Cap Top = 7184 + 822.6 + 9.072 + 471.936 + 123.425
- Pile Cap Bottom = 7184 + 822.6 + 9.072 + 471.936 + 123.425 + 1916.78

**Longitudinal moment**
- Pile Cap Top = 0.00 kNm
- Pile Cap Bottom = 0.00 kNm

**Transverse moment**
- Collision load in longitudinal direction = 250.00 kN acting at 1.5m above carriageway level of service road
  - Pile Cap Top = 250 \times (1.5 + 0.5) = 500 kNm
  - Pile Cap Bottom = 250 \times (1.5 + 2.3) = 950.00 kNm

**Summary of Axial Loads & Moments**

<table>
<thead>
<tr>
<th>Description</th>
<th>Footing top</th>
<th>Footing bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>8611.03</td>
<td>10527.82</td>
</tr>
<tr>
<td>M, (Tm)</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>M, (Tm)</td>
<td>500.00</td>
<td>500.00</td>
</tr>
</tbody>
</table>

The stresses in concrete and steel is calculated by a programme developed for columns subjected to axial load and biaxial bending. In none of the load cases, the stresses in concrete and steel is exceeded beyond the permissible limits. Hence the section adopted is safe.

As per Table - 1 and CI-202.3 in IRC-6:2000, permissible stresses in concrete and steel are increased by 50% and 33% for seismic and wind conditions respectively.

**Calculation of Pier Reinforcements.**

**Longitudinal Reinforcements:**
- (Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000 )
  - a) Not less than 0.3 % & not more than 8 % the gross C/S Area of the Column.
  - b) 0.8 % of the minimum area of concrete required to resist the direct stresses.

**Transverse Reinforcements:**
- a) Diameter of Transverse Reinforcement shall not be less than 1/4th the Dia of Main Reinforcement & minimum being 8mm.
- b) Minimum of 8mm Diameter.
- Pitch of Transverse Reinforcement shall be the least of the following.
  - a) The least Lateral Dimension of the Column.
  - b) 12 Times the Diameter of the smallest Longitudinal Reinforcement.
  - c) Maximum allowable spacing of 300 mm

---

---
Assumed % of Longitudinal Reinforcement = 1.700

Longitudinal Reinforcement Provided = \( \frac{13273.23 \times 1.70}{100} = 225.649 \text{ cm}^2 \)

Using 32 mm f bars,

Using 32 mm f bars,

No of bars reqd: \( \frac{225.645}{8.04277} = 28.06 \text{ Nos} \)

Provided steel 32 mm f bars, at 30 no of bars

Transverse Reinforcements:

a) Diameter of Transverse Reinforcement

\[ \frac{1}{4} \times 32 = 8 \text{ mm} \]

b) Minimum Diameter

Pitch of Transverse Reinforcement

a) The least Lateral Dimension of the Column = 1300 mm
b) 32 x 32 = 384 mm

So provide 10 m f bars, at 300 mm C/C.

**Design of Pilecap & Pile**

Length of pile = 25 m
S.B.C of Pile = 4520 kN
Moment due to tilt of pile,

As per Cl 709.1.6 of IRC:78-2000, for vertical piles,
Permissible shift of pile = 75 mm
Permissible tilt of pile (1:150) = 166.7 mm
Moment due to tilt of pile = Axial load/pile \times \{166.7\} mm
Depth of pile = 7000 mm
Reduction factor = 0.82

a) Piles:

For piles subjected to direct load aswell as moments, the distribution of loads on individual pile is determined as per the equation stated below.

\[
\text{Load} / \pi = \frac{W}{n} + \frac{M \times y}{\Sigma y^2} + \frac{M \times x}{\Sigma x^2}
\]
\[ W = \text{Total Load} \]
\[ n = \text{No of piles} = 6 \]
\[ y = 1.800 \text{ m} \] Considering 75mm permissible shift

\[ x = 1.800 \text{ m} \]
\[ \Sigma x = 12.960 \text{ m}^2 \]
\[ \Sigma y = 19.440 \text{ m}^2 \]

Reduction Factor = 0.8

Considering the total load from the 4 piers

Effective spacing between the piles = 3.6 m Considering 75mm permissible shift

Summary of Axial loads, Moments, Stresses, Steel provided & resultant stresses.

<table>
<thead>
<tr>
<th>Description</th>
<th>(i)</th>
<th>(ii)</th>
<th>(iii)</th>
<th>(iv)</th>
<th>(ivb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load kN</td>
<td>2966.99</td>
<td>3128.01</td>
<td>3778.89</td>
<td>3654.79</td>
<td>3497.13</td>
</tr>
<tr>
<td>Horizontal load/pile- Long</td>
<td>90.61</td>
<td>512.66</td>
<td>32.27</td>
<td>90.61</td>
<td>90.61</td>
</tr>
<tr>
<td>Horizontal load/pile- Trans</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>123.70</td>
<td>123.70</td>
</tr>
<tr>
<td>Moment due to horizontal load- Long</td>
<td>260.05</td>
<td>1471.34</td>
<td>92.61</td>
<td>260.05</td>
<td>260.05</td>
</tr>
<tr>
<td>Moment due to horizontal load- Trans</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>355.02</td>
<td>355.02</td>
</tr>
<tr>
<td>Moment/pile due to fill of piles</td>
<td>499.50</td>
<td>521.34</td>
<td>629.81</td>
<td>609.13</td>
<td>582.86</td>
</tr>
<tr>
<td>Total moment</td>
<td>759.55</td>
<td>3464.02</td>
<td>815.03</td>
<td>1309.25</td>
<td>1282.98</td>
</tr>
<tr>
<td>eccentricity (M/N)</td>
<td>0.25</td>
<td>1.11</td>
<td>0.22</td>
<td>0.36</td>
<td>0.37</td>
</tr>
<tr>
<td>radius of column</td>
<td>0.60</td>
<td>0.60</td>
<td>0.60</td>
<td>0.60</td>
<td>0.60</td>
</tr>
<tr>
<td>radius of rein: ring</td>
<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
</tr>
<tr>
<td>Bar size in mm</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Perimeter in mm</td>
<td>3298.67</td>
<td>3298.67</td>
<td>3298.67</td>
<td>3298.67</td>
<td>3298.67</td>
</tr>
<tr>
<td>spacings in mm</td>
<td>180.0</td>
<td>180.0</td>
<td>180.0</td>
<td>180.0</td>
<td>180.0</td>
</tr>
<tr>
<td>No of bars</td>
<td>18.33</td>
<td>18.33</td>
<td>18.33</td>
<td>18.33</td>
<td>18.33</td>
</tr>
<tr>
<td>% of reinforcement</td>
<td>0.0051</td>
<td>0.0051</td>
<td>0.0051</td>
<td>0.0051</td>
<td>0.0051</td>
</tr>
<tr>
<td>modular ratio</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
</tr>
<tr>
<td>e / R</td>
<td>0.422</td>
<td>1.846</td>
<td>0.36</td>
<td>0.60</td>
<td>0.61</td>
</tr>
<tr>
<td>% R</td>
<td>0.875</td>
<td>0.875</td>
<td>0.875</td>
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<tr>
<td>n</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Longitudinal Reinforcements:

\[ \text{C/S Area of Pile Section.} = 1.131 \text{ m}^2 = 11309.73 \text{ cm}^2 \]

\[ \text{C/S Area of Pile Section.} = 1.131 \times 0.4 = 45.24 \text{ cm}^2 \]

Remarks Safe Safe Safe Safe Safe

Calculation of Pile Reinforcements:

**Longitudinal Reinforcements:**

- **Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000**
- a) Not less than 0.3 % & not more than 8 % the gross C/S Area of the Column.
- b) 0.8 % of the minimum area of concrete required to resist the direct stresses.

**Transverse Reinforcements:**

- a) Diameter of Transverse Reinforcement shall not be less than 1/4th the Dia of Main Reinforcement & minimum being 8mm.
- b) Minimum of 8mm Diameter.
- Pitch of Transverse Reinforcement shall be the least of the following.

- a) The least Lateral Dimension of the Column.
- b) 12 Times the Diameter of the smallest Longitudinal Reinforcement.
- c) Maximum allowable spacing of 300 mm

**Longitudinal Reinforcements:**

\[ \text{C/S Area of Pile Section.} = 1.131 \text{ m}^2 = 11309.73 \text{ cm}^2 \]

\[ \text{C/S Area of Pile Section.} = 1.131 \times 0.4 = 45.24 \text{ cm}^2 \]
Assumed % of Longitudinal Reinforcement = 0.509
\[ = \frac{11309.73 \times 0.509}{100} = 57.57 \text{ cm}^2 \]

Using 20 mm f bars, \[ A_s = \frac{57.573}{3.141593} = 18.33 \text{ Nos} \]

Provided steel 20 mm f bars, at 19 no of bars

Transverse Reinforcements:

a) Diameter of Transverse Reinforcement
\[ = \frac{1}{4} \times 20 = 5 \text{ mm} \]

b) Minimum Diameter
\[ = 10 \text{ mm} \]

c) Maximum Allowable Spacing
\[ = 300 \text{ mm} \]

So provide 10 mm f bars, at 240 mm C/C.

Pilecap:

Design of steel in the Longitudinal direction.

Check for Pile Cap depth
Maximum Moment \[ 2267.33 \text{ kNm} \]
Moment of resistance factor \[ 1.90 \text{ N/mm}^2 \]
Effective depth required, \[ d_{req} = 1092.40 \text{ mm} \]
Effective depth provided, \[ d_{pro} = 1649.00 \text{ mm} \]
Hence Safe

Design of reinforcement in longitudinal direction
\[ R = F \sin \theta \]

Providing 32 mm dia bar at bottom of pile cap,
\[ \tan \theta = 0.95 \]
\[ = 43.40 \]

Horizontal force \[ H = 3996.47 \text{ kN} \]
Ast Provided \[ 19982.4 \text{ mm}^2 \]
Provide 80% steel in each band (above pile) [Refer CI 307.2.5 IRC 21:2000]
Steel area \[ 15985.9 \text{ mm}^2 \]
No of bars Reqd: \[ 15985.9 \]
\[ = 804.25 \text{ Nos} \]

No of bars in each band \[ 20.00 \text{ Nos} \]
Provide 20 bars of 32 mm dia in each band in longitudinal direction at bottom
( providing \[ 15985.9 \text{ mm}^2 \] )

1.5 times the diameter is taken as band width as per CI 307.2.5.2 -IRC 21-2000
Ast Required in the remaining portion \[ 3996.5 \text{ mm}^2 \]

Assume 20 mm dia bars
Provide 20 mm dia bar @ 120 mm c/c in longitudinal direction at pile cap bottom in the remaining portion.

Provide min reinforcement of 0.06% in the pile cap top
Ast required in the pile cap top \[ 1080 \text{ mm}^2 \]
Provide 16 mm dia bar @ 150 mm c/c in at pile cap top in both directions
Design of reinforcement in transverse dirn
This has to be designed as cantilever bending due to pile load.

Moment, M = 4345.72 kNm
Ast required = 14190.6 mm
Min Ast required = 0.85bd/fy = 16820 mm

Assume 25 mm dia bars
spacing = 140

Provide 25 mm bars @ 140 mm c/c

Min Stirrup Reinforcement Reqd:
Assume 8 legged 10 mm diameter stirrups
Sv = 162.039 mm
Provide 8 legged 10 mm diameter stirrups @ 150 mm C/c

Design of Bed Block

Moment at face of Pier = 750.00 kNm Refer staad
Torsion due to Live load from one side = 56.41 kNm
Equivalent longitudinal moment due to torsion = 49.77 kNm

\[ \Sigma \text{Moments} = 56.41 \times \frac{(1+1/1.4)}{1.7} \]

Effective Depth Reqd "deff" = \[ \frac{M}{G \times B \times d} \]
Clear Co' = 50 mm
Bar Dia = 25 mm

deff reqd = \[ \frac{750.00 \times 1000000}{1.47 \times 1600} = 564.69 \text{ cms} \]
< 993.75 cms Hence O.K

Ast reqd = \[ \frac{M}{V} \]

\[ \therefore \text{Ast reqd} = 750.00 \times 1000000 = 4239.98 \text{ mm}^2 \]

490.8739
No of bars Reqd: = \[ \frac{4239.983}{490.8739} = 8.64 \text{ Nos} \]
Spacing of Bars = \[ \frac{1600}{9} = 185.2 \text{ mm} \]

So provi. 25 mm dia bars, 185 mm c/c spacing

2) Check for Shear at pier face.

Calculation of S. Force at face of Pier due to
Due to DL, SIDL and LL = 1014 kN Refer staad
Shear due to torsion = 56.41 kN
Shear = 1071.07 kN

\[ \tau = \frac{V}{B \times d} \]
V = The design shear across the section
B = Breadth of slab

\[ \therefore \tau = \frac{1071.1 \times 1000}{1600 \times 993.75} = 0.7 \text{ Kg/cm}^2 \]

Maximum Permissible Shear Stress:
\[ \tau_{\text{max}} = 2.3 \text{ Mpa} \]
\[ \tau_{\text{max perm}} = 23.0 \text{ Kg/cm}^2 \]

Calculation of permissible Shear Stress:

\[ d = 993.75 \text{ Effective Depth} \]
\[ p = \frac{100 \times A_s}{B \times d} \]
\[ \therefore \tau_e = \frac{4240.0 \times 100}{1600 \times 993.8} = 0.267 \text{ kg/cm}^2 \]

Since \( \tau > \tau_e \) Shear Reinforcement Reqd:
Shear Reinforcement Req'd :-

\[
A_{sw} = \frac{V}{s \cdot f_t} \times x \times s
\]

\[
V = 685.75 \text{ kN}
\]

\[
s, \text{ spacing} = 100 \text{ mm} \quad \text{(assumed)}
\]

\[
\therefore A_{sw} = \frac{685.75}{200} \times 1000 \times 100 = 345.0 \text{ mm}^2
\]

Using 12 mm dia stirrups 6 Legged at spacing 100 mm.

\[
A_{sw} = 678.6 \text{ cm}^2 > 345.0 \text{ mm}^2
\]

Hence O.K

B) Design of Cantilever Portion in the Longitudinal Direction.

The cantilevered portion of Bed block in this direction is very less. Even for one span off condition during construction time, the girders are not rested initially over this cantilever portion. So the nominal reinforcement need to be provided.
### Design Data for Substructure:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>PSC Girder</td>
</tr>
<tr>
<td>C/C Distance between piers</td>
<td>40000 mm</td>
</tr>
<tr>
<td>Carriageway Width</td>
<td>7500 mm</td>
</tr>
<tr>
<td>Overall Width of Deck</td>
<td>17000 mm</td>
</tr>
<tr>
<td>Width of Crash Barrier</td>
<td>500 mm</td>
</tr>
<tr>
<td>Width of Median</td>
<td>1000 mm</td>
</tr>
<tr>
<td>Height of Crash Barrier</td>
<td>1050 mm</td>
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<tr>
<td>No of Bearings</td>
<td>3</td>
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<tr>
<td>Overall Depth of I.Girders</td>
<td>2200 mm</td>
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<tr>
<td>Depth of Deck Slab</td>
<td>250 mm</td>
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<tr>
<td>Width of cross girder</td>
<td>2200 mm</td>
</tr>
<tr>
<td>Depth of cross girder</td>
<td>2200 mm</td>
</tr>
<tr>
<td>Overall Depth of substructure</td>
<td>6748 mm</td>
</tr>
<tr>
<td>Longitudinal width of Pier at bottom</td>
<td>1700 mm</td>
</tr>
<tr>
<td>Transverse width of pier at bottom</td>
<td>3000 mm</td>
</tr>
<tr>
<td>Transverse width of pier above pile cap</td>
<td>4000 mm</td>
</tr>
<tr>
<td>Longitudinal width of pile cap</td>
<td>2500 mm</td>
</tr>
<tr>
<td>Transverse width of pile cap</td>
<td>5000 mm</td>
</tr>
<tr>
<td>Overall Height of Substructure</td>
<td>6748 mm</td>
</tr>
<tr>
<td>Foundation level for Piles.</td>
<td>185.70 m</td>
</tr>
<tr>
<td>Height of varying portion of pile</td>
<td>3000.00 mm</td>
</tr>
<tr>
<td>Height of straight portion of pile</td>
<td>2547.50 mm</td>
</tr>
<tr>
<td>Existing Ground level</td>
<td>210.70 m</td>
</tr>
<tr>
<td>Pier Cap top below existing ground</td>
<td>500 mm</td>
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<tr>
<td>R.L at Pier cap top</td>
<td>210.20 m</td>
</tr>
<tr>
<td>R.L at Pier cap bottom</td>
<td>208.40 m</td>
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<tr>
<td>Depth of Pier below GL</td>
<td>25000 mm</td>
</tr>
<tr>
<td>Pier Cap Width in Long Dir.</td>
<td>2500 mm</td>
</tr>
<tr>
<td>Pier Cap Length in Trans Dir.</td>
<td>9700 mm</td>
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<tr>
<td>Straight Depth of Pier Cap</td>
<td>600 mm</td>
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<td>Varying Depth of Pier Cap</td>
<td>600 mm</td>
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<tr>
<td>Type of Bearing</td>
<td>Pot PTFE bearing</td>
</tr>
<tr>
<td>Size of Pedestals</td>
<td>600 x 600 x 350</td>
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<td>Distance between Pedestals</td>
<td>4400 mm</td>
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<td>Longitudinal width of pile cap</td>
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<td>P.C.C Projections</td>
<td>150 mm</td>
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<tr>
<td>Diameter of Pile</td>
<td>1200 mm</td>
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<td>Distance between Piles in longitudinal dir</td>
<td>3600 mm</td>
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<tr>
<td>Distance between Piles in transverse dir</td>
<td>3600 mm</td>
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<tr>
<td>No of Piles</td>
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<td>Edge projection in</td>
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<tr>
<td>Edge projection in transverse</td>
<td>150 mm</td>
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<tr>
<td>Grade of Concrete for pier</td>
<td>M 50</td>
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<tr>
<td>Permissible flexural stress</td>
<td>16.67 N/mm²</td>
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<tr>
<td>Grade of Steel</td>
<td>Fe - 415</td>
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<td>200 N/mm²</td>
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<td>5548 mm</td>
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<tr>
<td>Density of Concrete</td>
<td>24 kN/m³</td>
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<tr>
<td>Density of Concrete for Earth</td>
<td>18 kN/m³</td>
</tr>
</tbody>
</table>
References:
I.R.C. - 21 - 2000 - Permissible Stresses
I.R.C. - 78 - 2000 - Sub Structure and Foundation
# Load Calculations:

## 1) **DEAD LOADS**

- **Load from slab, girder & diaphragm** = 7184 kN for 40m span
- **Load from slab, girder & diaphragm** = 7184 kN for 40m span
- **Total dead load** = 14368.00 kN

## 2) **S.I.D.L**

- **Wearing coat load** = 20 x 0.065 x 15 x 22 for 40m span = 429.00 kN
- **Crash barrier load** = 249.60 kN for 40m span
- **Median load** = 144.00 kN for 40m span
- **SIDL for 40m span** = 822.60 kN
- **SIDL for 40m span** = 822.60 kN
- **S.I.D.L** = 1645.20 kN

## 3) **LIVE LOAD**

- **Impact Factor** = 1 considering 50 % reduction as per clause 211.7 IRC 6-2000

### 70 R

<table>
<thead>
<tr>
<th>B</th>
<th>170</th>
<th>170</th>
<th>170</th>
<th>170</th>
<th>120</th>
<th>120</th>
<th>80</th>
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<tr>
<td>R</td>
<td>867</td>
<td>38.4</td>
<td>38.4</td>
<td>133.4323</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

\[
R_C = \frac{(170 \times 38.4 + 170 \times 37.03 + 170 \times 33.98 + 170 \times 32.61 + 120 \times 30.48 + 120 \times 28.96 + 80 \times 25)}{38.4}
\]

\[
R_C = 866.568 \text{ kN}
\]

\[
R_D = 170 + 170 + 170 + 170 + 120 + 120 + 80 - 866.568
\]

\[
R_D = 133.432 \text{ kN}
\]

### 70 R one lane

\[
R = 866.57 \text{ kN}
\]
Class A One lane

$R_A = 400.67$ kN
$R_B = 153.33$ kN
$R_C = 430.32$ kN
$R_D = 123.68$ kN

Reaction at pier Support = 583.65 kN
Class A One lane = 583.65 kN
Class 70R = 866.57 kN
Class A two lane = 1167.29 kN
Class A One lane (One span dislodged) = 430.32 kN
Critical live load = 866.57 kN
Calculation of Dead Load from Sub - Structure :-

1) From Pedestals
   - Volume of 1 Pedestal = 0.6 x 0.6 x 0.35 = 0.126 m³
   - No: of Pedestals = 6
   - Total Volume = 0.126 x 6 = 0.76 m³
   - Total Load = 0.756 x 24 = 18.14 kN

2) From Bed - Block :
   - Total C/S Area in transverse Dirn = 11.25 m²
   - Width in the longitudinal Dirn = 2.5 m
   - Total Volume = 28.13 m³
   - Total Load = 28.13 x 24 = 675.00 kN

3) From Pier :
   - Area at top = 10.62 m²
   - Area at bottom = 4.35 m²
   - Averaged area = 7.485 m²
   - Height of varying portion = 3 m
   - Volume of varying portion = 22.455 m³
   - Height of straight portion = 1.5475 m
   - Volume of straight portion = 5.78 m³
   - Area of pier above pilecap = 5.78 m²
   - Volume of tapered portion = 5.78 m³
   - Total Volume = 33.967 m³
   - Total Load = 33.967 x 24 = 809.20 kN

4) From Pile Cap :
   - Volume of pile cap = (8.7 x 8.7 x 1.8) = 136.242 m³
   - Dead Load = 136.242 x 24 = 3269.81 kN

5) From Pile :
   - Diameter of Pile = 1.2 m
   - Depth of Pier with Circular Xn = 25 m
   - C/S Area of Circular Pier = 1.131 m²
   - No of Piles = 8.00
   - Total Volume of Concrete = 226.195 m³
   - Total Load = 226.195 x 24 = 5428.67 kN

Options
A) Both Spans on.

Calculation of Longitudinal Moments at start of Pier Flaring & Pier Base & Pilecap Base

1) Due to Braking
   b) Braking
      - [Vide :- cl 214.2 (a) & (b) of I.R.C : 6 - 2000 .]
      - Braking - Since the movement of bearing under the girders on one side is restricted to move in the longitudinal direction half the effect of braking is considered in the design.
      1) 20 % of Ist Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.
      2) 20 % of Ist Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.
      + 5 % of Loads on the lanes exceeding Two.

   2 Lanes of Class A Wheeled Vehicles,
   - Total Load of 1 Vehicle = 554 kN one span
   - : Braking Force = 110.8 kN

   1 Lane of Class 70 R Wheeled Vehicles
   - Total Load of 70R Vehicle = 1000 kN one span
   - : Braking Force = 200 kN
   - Max Braking Force = 200 kN

   Vertical reaction due to braking = 200(1.2+0.065+2.45)/(40 -1.6)
   = 19.35 kN
Longitudinal moment due to vertical load of braking

Longitudinal Eccentricity = 0.800 m
Moment due to long. Eccentri = 15 kNm

2) **Due to Temp & Shrinkage of Bearings :**

**Fixed Bearing**
- Coefficient of Friction, \( m \) = 0.05
- (i) \( F_h - m (R_g + R_q) \) = -200.33 kN
- (ii) \( F_h + m (R_g + R_q) \) = 543.66 kN

**Free Bearing**
- Coefficient of Friction, \( m \) = 0.05
- (i) \( m (R_g + R_q) \) = 400.33 kN
- Total Force = 943.99 kN
- \( \therefore \) Moment at Pile Cap top = \( 944 \times 7.20 = 6794.36 \) kNm
- Moment at Pile Cap bottom = \( 944 \times 9.00 = 8493.54 \) kNm

3) **Moment due to Longitudinal Eccentricity**

Longitudinal Eccentricity = 0.80 m

<table>
<thead>
<tr>
<th>Normal Case</th>
<th>Seismic Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Due to DL</td>
<td>0 kNm</td>
</tr>
<tr>
<td>Due to SIDL</td>
<td>0 kNm</td>
</tr>
<tr>
<td>Due to LL</td>
<td>693 kNm</td>
</tr>
<tr>
<td>Moment</td>
<td>693 kNm</td>
</tr>
</tbody>
</table>

4) **Centrifugal force**

- Live load = 867 kN
- Design Speed = 100 km/hr
- Radius of curvature = 2350.00 m
- Centrifugal force = \( \frac{WV^2}{127R} \)
- = 29 T

Centrifugal force acts at 1.2 m above the level of carriageway. Increase in impact effect is not considered (Refer CI 215-IRC 6:2000)

- \( \therefore \) Moment at Pile Cap top = 29 \( \times \) 10.91 = 316.85 kNm
- \( \therefore \) Moment at Pile Cap bottom = 29 \( \times \) 12.72 = 369.41 kNm

**Due to Live Load**

Transverse moment about the centre of the pier is calculated by finding the eccentricity

**Class 70R 1 lane**

\[ \begin{align*}
433.28 & \quad 433.3 \\
2.13 & \quad 1.93 \\
\end{align*} \]

\[ \begin{align*}
\frac{c/l}{i\text{ of pier}} \\
8.5 \quad 7.5 \quad 17 \quad 7.5 \quad 8.50 \quad 0.5 \\
\end{align*} \]

Moment in Transverse Dirn = 4683.80 kNm
Class A 2 lane.

Moment in Transverse Dirn. = 2001.77 kNm

Class A 1 lane.

Moment in Transverse Dirn. = 5778.09 kNm

Due to Class A One lane

Moment in Transverse Dirn. = 1634.21 kNm

Transverse Eccentricity = 3.80 m

Moment in Transverse Dirn. = 2883.11 kNm

Axial Load
Pile Cap Top = 14368 + 1645.2 + 866.57 + 18.15 + 675 + 839.2 + 19.35
Pile Cap Bottom = 14368 + 1645.2 + 866.57 + 18.15 + 675 + 839.2 + 3269.81 + 19.35

Longitudinal moment
Pile Cap Top = 6794.36 + 693.26 + 15.48
Pile Cap Bottom = 8493.54 + 693.26 + 15.48

Transverse moment
Pile Cap Top = 6094.94
Pile Cap Bottom = 6147.50

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>18431.5</td>
<td>21701.3</td>
</tr>
<tr>
<td>Mx (Tm)</td>
<td>7503.1</td>
<td>9202.3</td>
</tr>
<tr>
<td>Mx (Tm)</td>
<td>6094.9</td>
<td>6147.5</td>
</tr>
</tbody>
</table>
8) **One Span Dislodged with one lane of class A**

**Case (ii)**

Calculation of Longitudinal Moments at Pier Base & Pile cap Base

1) Due to Temp & Shrinkage of Bearings:

   \[ \frac{F_h + m (R_g + R_q)}{2} = 499.06 \text{ kN} \]

   Vertical reaction due to braking

   \[ = \frac{110.8(1.2+0.065+2.45)/(40 -1.6)}{10.72 \text{ kN}} \]

   Longitudinal moment due to vertical load of braking

   Longitudinal Eccentricity \( = 0.800 \text{ m} \)

   Moment due to long.Eccentri \( = 9 \text{ kNm} \)

2) Due to Eccentricity of Dead Load and SIDL:

   Eccentricity of Loading \( = 0.8 \text{ m} \)

   Dead Load + SIDL \( = 8006.60 \text{ kN} \)

   Longitudinal Moment \( = 6405.28 \text{ kNm} \)

   Transverse moment for sild \( = 0.00 \text{ kNm} \)

   Axial Load

   Pile Cap Top \( = 822.6 + 7184 + 430.32 + 18.15 + 675 + 839.2 + 10.72 \)

   Pile Cap Bottom \( = 822.6 + 7184 + 430.32 + 18.15 + 675 + 839.2 + 3269.81 + 10.72 \)

   Longitudinal moment

   Pile Cap Top \( = 6405.28 + 430.32 \times 1.125 + 499.06 \times 7.2 + 8.58 \)

   Pile Cap Bottom \( = 6405.28 + 430.32 \times 1.125 + 499.06 \times 9 + 8.58 \)

   Transverse moment

   Pile Cap Top \( = 2883.12+871.2+157.34 \)

   Pile Cap Bottom \( = 2883.12+871.2+183.43 \)

4 Centrifugal force

   Live load \( = 430 \text{ kN} \)

   Design Speed \( = 100 \text{ km/hr} \)

   Radius of curvature \( = 2350.00 \text{ m} \)

   Centrifugal force \( = \frac{WV^2}{2R} \)

   \( = 14 \text{ kN} \)

   Centrifugal force acts at 1.2m above the level of carriage way. Increase in impact effect is not considered. (Refer Cl 215-IRC 6:2000)

   Moment at Pile Cap top \( = 14 \times 10.91 = 157.34 \text{ kNm} \)

   Moment at Pile Cap bottc \( = 14 \times 12.72 = 183.44 \text{ kNm} \)

**Summary of Axial Loads & Moments**

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>9979.98</td>
<td>13249.79</td>
</tr>
<tr>
<td>M_L (Tm)</td>
<td>10350.08</td>
<td>11248.39</td>
</tr>
<tr>
<td>M_T (Tm)</td>
<td>3754.31</td>
<td>3754.31</td>
</tr>
</tbody>
</table>

**Case (iii)**

As per Modified Clause 222

\[ F_{eq} = A_c \ (\text{Dead load} + \text{Appropriate Live Load}) \]

\[ F_{eq} = \text{Seismic force to be resisted} \]
\[ A_h = \frac{Z}{Z} \left( \frac{Sa}{g} \right) \]

Zone No = IV
Zon Factor, Z = 0.24
\[ Sa/g = \text{Average acceleration coefficient} \]
Response Modification factor, R = 3.3
Importance Factor, I = 1.2
\[ T = \frac{D}{1000F} \]

Dead load of the super structure, and appropriate live load in kN, D
\[ F = 16013.20 \text{ kN} \]

Horizontal force in kN required to be applied at the centre mass of the super structure for one mm horizontal deflection at the top of the pier /abutment along the considered direction of horizontal force.
\[ F = \frac{3EI \delta}{I^3} \]

Modulus of Elasticity of concrete, E = 3.0E+07 kN/m³
\( I = 0.41 \text{ m}^4 \)
Deflection, \( \delta \) = 0.001 m
Distance from bottom to the centre of mass of super structure, y = 1.23 m
Height of substructure, h = 6.75 m
Total height measured from the centre of mass of super structure, l = 8.27 m

\[ F = 64.27 \text{ kN} \]
\[ T = 1.00 \]
Soil type = II
\[ Sa/g = 1.362 \]
\[ A_h = 0.18 \]
\[ F_{eqz} = 3141.24 \text{ kN} \]
Design Seismic force in long d = \( F_{eqz} / R \)
\[ DF_{eqz} = 951.89 \text{ kN} \]

Seismic force in transverse direction
The seismic force due to live load shall be considered when acting in the direction perpendicular to traffic. The horizontal seismic force in the direction perpendicular to traffic shall be computed by taking 20% of live load (excluding impact factor).
Dead load and appropriate live load, Feq
\[ Feq = A_h (\text{Dead load} + \text{Appropriate Live Load}) \]
Iyy = 0.41 m^4
F = 64.27 kN
T = 1.00
Soil type = II
Sa/g = 1.35
Ah = 0.2
Feqx = 3158.19 kN
= 957.03 kN
Seismic vertical component, V = 638.02 2/3 * 957.03
Resultant force in longi dim = 1430.40 kN DFeqz+0.3 DFeqx+ 0.3Vf
CG of loads = 7.52

Moment, M_{eq} at Pile Cap top = 10756.85 kNm
Moment, M_{eq} at Pile Cap bottom = 13331.58 kNm
Seismic vertical load [Acting downwards-considering critical condition]
At pile cap top = 1210.69 kN 0.3DFeqz+0.3 DFeqx+ Vf
At pile cap bottom = 1513.37 kN (0.3DFeqz+0.3 DFeqx+ Vf ) *1.25

Axial load
Pile Cap Top = 14368+1645.2+866.57 x 0.5+18.15+675+839.2 + 19.35
Pile Cap Bottom = 14368+1645.2+866.57 x 0.5+18.15+675+839.2 + 3269.81 + 19.35

Longitudinal moment
Pile Cap Top = 5778.09 x 0.5 +0+316.85x 0.5
Pile Cap Bottom = 5778.09 x 0.5 +0+369.4x 0.5

As per Table 1 Load Combination, 50 % of LL is considered in seismic case

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>16787.48</td>
<td>19754.62</td>
</tr>
<tr>
<td>M_{t} (Tm)</td>
<td>17397.51</td>
<td>21542.42</td>
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<tr>
<td>M_{t} (Tm)</td>
<td>3047.47</td>
<td>3073.75</td>
</tr>
</tbody>
</table>

D) Both Spans on Under Seismic in Transverse Direction.

The Seismic effect on Live Load is taken in this Case

Design Seismic force in trans dim, DFeq 957.03 kN
Resultant force in trans dim = 1434.00 kN 0.3DFeqz+ DFeqx+ 0.3Vf
CG of loads = 7.52 m

Moment, M_{eq} at Pile Cap top = 10783.89 kNm
Moment, M_{eq} at Pile Cap bottom = 13365.09 kNm

Axial load is same as that of Case iii

Longitudinal moment
Pile Cap Top = [100 x 0.5 + 46.8] x 2 x 7.197 +10756.84 + 346.627 + 15.48
Pile Cap Bottom = [100 x 0.5 + 46.8] x 2 x 8.99 +13331.57 + 346.627 + 15.48

Transverse moment
Pile Cap Top = 3047.47 + 10783.9
Pile Cap Bottom = 3047.47 + 13365.1

As per Table 1 Load Combination, 50 % of LL is considered in seismic case
Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>16787.48</td>
<td>19754.02</td>
</tr>
<tr>
<td>Mx (Tm)/MM (Tm)</td>
<td>6625.18</td>
<td>8195.36</td>
</tr>
<tr>
<td>Mz (Tm)/MM (Tm)</td>
<td>13989.79</td>
<td>16570.99</td>
</tr>
</tbody>
</table>

E) One Spans Dislodged Under Seismic in Longitudinal Direction.  

Case (v)

D = 8006.60 kN
T = 0.71
Soil type = II
Sa/g = 1.93
Ah = 0.28
F_{eq} = 2221.19 kN
Design Seismic force in long d = F_{eq} / R
DF_{eq} = 673.09 kN

Seismic force in transverse direction-One span dislodged

Dead load and appropriate
F_{eq} = 8092.66 kN
F = 64.27 kN
T = 0.71
Soil type = II
Sa/g = 1.92
Ah = 0.28
F_{eq} = 2233.10 kN
Design Seismic force in trans = 676.70 kN
Seismic vertical load (Acting downwards-considering critical condition)
At pile cap top = 856.07 kN
At pile cap bottom = 1070.08 kN

Axial load
Pile Cap Top = 8006.6+430.32*0.5+18.15+675+839.2 + 10.72
Pile Cap Bottom = 8006.6+430.32*0.5+18.15+675+839.2+3269.81 + 10.72

Longitudinal moment
Pile Cap Top = 6405.28 + (50 + 46.8)*x 7.2 + 172.13 + 7348.98 + 8.58
Pile Cap Bottom = 6405.28 + (50 + 46.8)*x 8.99 + 172.13 + 9169.56 + 8.58

Transverse moment
Pile Cap Top = 2883.12 x 0.5+0
Pile Cap Bottom = 2883.12 x 0.5+0

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>8908.75</td>
<td>11964.55</td>
</tr>
<tr>
<td>Mx (Tm)</td>
<td>17281.23</td>
<td>19938.67</td>
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<tr>
<td>Mz (Tm)</td>
<td>1520.23</td>
<td>1533.27</td>
</tr>
</tbody>
</table>

The seismic force caused due to dead load of the girders placed over the Sand Jacks are considered.
F) One Span Dislodged Under Seismic in Transverse Direction.

Case (vi)

- Design Seismic force in trans: $DF_{eq} = 676.70$ kN
- Resultant force in trans: $1013.96$ kN
- CG of loads: $7.27$ m

Moment $M_{eq}$ at Pile Cap top: $7367.33$ kNm
Moment $M_{eq}$ at Pile Cap bottom: $9192.46$ kNm

Axial load is same as for Case v

Longitudinal moment
- Pile Cap Top: $6405.28 + (50 + 46.8) \times 7.197 + 172.13$
- Pile Cap Bottom: $6405.28 + (50 + 46.8) \times 8.99 + 172.13$

Transverse moment
- Pile Cap Top: $2883.12 \times 0.5 + 0 + 7367.33$
- Pile Cap Bottom: $2883.12 \times 0.5 + 0 + 9192.46$

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
</tr>
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<tbody>
<tr>
<td>Axial Load</td>
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<td>$M_1$ (Tm)</td>
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<td>$M_2$ (Tm)</td>
<td>8966.22</td>
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G) Service condition with Wind in Transverse direction

Case (vii)

Transverse direction

- Height of the exposed surface above ground level: $6.25$ m
- Exposed depth of C/Barrier & Superstructure: $2.45 + 1.05 = 3.50$ m

Due to Crash Barrier
- Avg Height of Crash barrier from GL: $6.77$ m
- Intensity of Wind pressure corres to height: $463.70$ Kg/m²
- Average Exposed Length: $40.00$ m
- Effective area of crash barrier: $42.00$ m²
- Force: $P_z A \cdot G \cdot C_d$
  - $G = 2$
  - $C_d = 1.30$
  - Force: $506.36$ kN

Due to Deck Slab and girder
- Average ht of deck slab and girder from GL: $7.12$ m
- Intensity of Wind pressure corres to height: $463.70$ Kg/m²
- Effective area of deck slab + girder: $98.00$ m²
- Force: $P_z A \cdot G \cdot C_d$
  - $G = 2$
  - $C_d = 1.95$
  - Force: $177.23$ kN

Live Load
- Effective length: $40.00$ m
- Depth: $3.00$ m
- Height of the exposed surface above ground level: $7.75$ m
- Area: $120.00$ m²
- Pressure: $463.70$ N/m²
- Force: $P_z A \cdot G \cdot C_d$
  - $G = 2$
  - $C_d = 1.20$
  - Force: $133.55$ kN
Due to Pier
Average ht of pier and pier cap from GL = 3.37 m
Intensity of Wind pressure comes : to height = 463.70 kg/m²
Effective area of pier and pier cap = 10.73 m²
Force = P₀A₀GCₐ kN
G = 2
Cₐ = 0.80
Force = 7.96 kN
Total transverse force = 825.09 kN

Transverse Moment due to wind
Pile Cap Top = 5753.12 kNm
Pile Cap Bottom = 6495.71 kNm

Longitudinal Force
Crash barrier 25 % of trans force = 126.59 kN
Superstructure 25 % of trans force = 44.31 kN
On Live load 25 % of trans force = 33.99 kN
Substructure 25 % of trans force = 1.99 kN
Total longitudinal force = 206.27 kN

Longitudinal Moment due to wind
Pile Cap Top = 1348.38 kNm
Pile Cap Bottom = 1809.57 kNm

Vertical load = P₀A₀GCₐ

Axial load = 463.70 N/m²
A₀ = 680.00
G = 2.00
Cₐ = 0.75
= 472.97 kN Acting upwards or downwards

Axial loads and Longitudinal Moments are same as Case 1

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
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<tbody>
<tr>
<td>Axial Load</td>
<td>18904.43</td>
<td>22174.24</td>
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<tr>
<td>M₀ (Tm)</td>
<td>8941.37</td>
<td>11011.84</td>
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<tr>
<td>M₀ (Tm)</td>
<td>11848.07</td>
<td>12643.21</td>
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<table>
<thead>
<tr>
<th>Descriptions</th>
<th>Pile Cap Top</th>
<th>Pile Cap Bottom</th>
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</thead>
<tbody>
<tr>
<td>Axial Load</td>
<td>17938.49</td>
<td>21228.29</td>
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<td>M₀ (Tm)</td>
<td>8941.37</td>
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<tr>
<td>M₀ (Tm)</td>
<td>11848.07</td>
<td>12643.21</td>
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Case ( vii-b )

Effect of collision in longitudinal direction

Axial load
Pile Cap Top = 14368+1645.2+18.144+675+839.199
Pile Cap Bottom = 14368+1645.2+18.144+675+839.199 + 3269.8

Longitudinal moment
Collision load in longitudinal direction = 500.00 kN acting at 1.5m above carriageway level of service road
Pile Cap Top = 500x(1.5+0.5) = 1000.00 kNm
Pile Cap Bottom = 500x(1.5+2.30) = 1900.0 kNm
Transverse moment
Pile Cap Top = 0.00 kNm
Pile Cap Bottom = 0.00 kNm

Summary of Axial Loads & Moments

<table>
<thead>
<tr>
<th>Descriptions</th>
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<th>Footing bottom</th>
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<tr>
<td>Axial Load</td>
<td>17545.54</td>
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<tr>
<td>M&lt;sub&gt;l&lt;/sub&gt; (Tm)</td>
<td>1000.00</td>
<td>1900.00</td>
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<tr>
<td>M&lt;sub&gt;t&lt;/sub&gt; (Tm)</td>
<td>0.00</td>
<td>0.00</td>
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</table>

J) Effect of collision in transverse direction
Case ix

Axial load
Pile Cap Top = 14368+1645.2+18.144+675+839.199
Pile Cap Bottom = 14368+1645.2+18.144+675+839.199 + 3269.8

Longitudinal moment
Pile Cap Top = 0.00 kNm
Pile Cap Bottom = 0.00 kNm

Transverse moment
Collision load in longitudinal direction = 250.00 kN acting at 1.5m above carriageway level of service road
Pile Cap Top = 250x(1.5+0.5) = 500.00 kNm
Pile Cap Bottom = 250(1.5+2.30) = 950.00 kNm

Summary of Axial Loads & Moments

Summary of loads and moment for all 7 cases

<table>
<thead>
<tr>
<th>Description</th>
<th>(i)</th>
<th>(ii)</th>
<th>(iii)</th>
<th>(iv)</th>
<th>(v)</th>
<th>(vi)</th>
<th>(vii)</th>
<th>(viii)</th>
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<tbody>
<tr>
<td>At Pile Cap top</td>
<td></td>
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<tr>
<td>Axial Load (T)</td>
<td>18431.46</td>
<td>9979.98</td>
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<td>16787.48</td>
<td>8908.75</td>
<td>8908.75</td>
<td>17958.49</td>
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<tr>
<td>Moment (Long-T)</td>
<td>7503.09</td>
<td>10350.08</td>
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<td>13989.79</td>
<td>8966.22</td>
<td>8966.22</td>
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<td>% steel assumed</td>
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<td>1.89</td>
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<tr>
<td>Stress in concrete</td>
<td>9.20</td>
<td>10.18</td>
<td>15.40</td>
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<td>14.1</td>
<td>12.6</td>
<td>12.8</td>
<td>12.8</td>
<td>9.1</td>
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<tr>
<td>Stress in steel (N/mm&lt;sup&gt;2&lt;/sup&gt;)</td>
<td>28.80</td>
<td>97.04</td>
<td>145.60</td>
<td>67.4</td>
<td>203.6</td>
<td>130.3</td>
<td>68.3</td>
<td>72.9</td>
<td>59.5</td>
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<td>At Pile Cap bottom</td>
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<td>Axial Load (T)</td>
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<td>19754.62</td>
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<td>22174.24</td>
<td>22174.24</td>
<td>22174.24</td>
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<td>Moment (Long-T)</td>
<td>9202.27</td>
<td>11248.39</td>
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<td>11011.84</td>
<td>11011.84</td>
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<td>Moment (Trans-T)</td>
<td>6147.50</td>
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<td>10817.4</td>
<td>12643.21</td>
<td>12643.21</td>
<td>12643.21</td>
</tr>
</tbody>
</table>

The stresses in concrete and steel is calculated by a programme developed for columns subjected to axial load and biaxial bending. In none of the load cases, the stresses in concrete and steel is exceeded beyond the permissible limits. Hence the section adopted is safe.

As per Table - I and CI-202.3 in IRC-6:2000, permissible stresses in concrete and steel are increased by 50% and 33% for seismic and wind conditions respectively.

Calculation of Pier Reinforcements.

Longitudinal Reinforcements:
- a) Not less than 0.3% & not more than 8% the gross C/S Area of the Column.
- b) 0.8% of the minimum area of concrete required to resist the direct stresses.

Transverse Reinforcements:
- a) Diameter of Transverse Reinforcement shall not be less than 1/4th the Dia of Main Reinforcement & minimum being 8mm.
b) Minimum of 8mm Diameter.
Pitch of Transverse Reinforcement shall be the least of the following.

a) The least Lateral Dimension of the Column.
b) 12 Times the Diameter of the smallest Longitudinal Reinforcement.
c) Maximum allowable spacing of 300 mm

Longitudinal Reinforcements:

C/S Area of Pier Section. = 2.270 m² = 22698.01 cm²
a) 0.3 % C/S Area. = 22698.007 x 0.3 = 68.09402 cm²
b) b) Direct Stress = \( \frac{P}{A} \) = \( \frac{22174.24}{22698.007} \) = 1.890
\( \sigma_{cbc} = \frac{16.666667}{A} \) Kg/cm²
\therefore Area = \( \frac{2217424.2}{166.6667} \) = 13305 cm²
0.8 % of Min C/S Area. = \( \frac{13304.545}{100} \) x 0.8 = 106.436 cm²

Assumed % of Longitudinal Reinforcement = 1.890
\therefore Longitudinal Reinforcement Provided.
= 22698.007 x 1.89 = 428.9923 cm²

Using 32 mm f bars, As1 = 8.042 cm²
No of bars reqd: = \( \frac{428.992}{8.042} \) = 53.34 Nos

Provided steel 32 mm f bars, at 54 nos of bars

Transverse Reinforcements:

a) Diameter of Transverse Reinforcement = \( \frac{1}{4} \) x 32 = 8 mm

b) Minimum Diameter = 10 mm

Pitch of Transverse Reinforcement

a) The least Lateral Dimension of the Column. = 1700 mm
b) 12 x 32 = 384 mm
c) Maximum Allowable Spacing = 300 mm

So provide 10 mm f bars, at 300 mm C/C.
Length of pile = 25 m
S.B.C of Pile = 4520 kN
Moment due to tilt of pile
As per Cl 709.1.6 of IRC:78-2000, for vertical piles,
Permissible shift of pile = 75 mm
Permissible tilt of pile (1:150) = 166.7 mm
Depth of fixity = 7000 mm
Reduction factor = 0.82

a) Piles:
For piles subjected to direct load as well as moments, the distribution of loads on individual pile
is determined as per the equation stated below.

\[
\frac{W}{n} + \frac{M x y}{\Sigma y^2} + \frac{M y x}{\Sigma x^2}
\]

\[
W = \text{Total Load}
\]
\[
n = \text{No of piles} = 8
\]
\[
y = 3.60 \text{ m} \quad \text{Considering 75mm permissible shift}
\]
\[
x = 3.60 \text{ m}
\]
\[
\Sigma x^2 = 77.76 \text{ m}^2
\]
\[
\Sigma y^2 = 77.76 \text{ m}^2
\]

Reduction Factor = 0.82
Considering the total load from the pier
Effective spacing between the piles = 3.6 m Considering 75mm permissible shift

<table>
<thead>
<tr>
<th>Description</th>
<th>(I)</th>
<th>(II)</th>
<th>(III)</th>
<th>(IV)</th>
<th>(V)</th>
<th>(VI)</th>
<th>(VII)</th>
<th>(VIII)</th>
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<tbody>
<tr>
<td>Axial Load kN</td>
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<td>2309.20</td>
<td>3415.85</td>
<td>3423.72</td>
<td>3423.72</td>
<td>2366.01</td>
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<td>3734.25</td>
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<tr>
<td>Horizontal load/pile-Long</td>
<td>118.00</td>
<td>62.38</td>
<td>416.85</td>
<td>24.20</td>
<td>289.75</td>
<td>12.10</td>
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<td>Horizontal load/pile-Trans</td>
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<td>0.00</td>
<td>0.00</td>
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<tr>
<td>Moment due to horizontal load-Long</td>
<td>338.66</td>
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<td>34.73</td>
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<td>Mr = M_L + M_T</td>
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<td>Total moment</td>
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<tr>
<td>Radius of reinforcement (r)</td>
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<td>Perimeter in mm</td>
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</table>
Calculation of Pile Reinforcements.

Longitudinal Reinforcements:

\[ \text{Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000} \]

a) Not less than 0.4% & not more than 8% the gross C/S Area of the Column.
b) 0.8% of the minimum area of concrete required to resist the direct stresses.

Transverse Reinforcements:

a) Diameter of Transverse Reinforcement shall not be less than 1/4th the Dia of Main Reinforcement & minimum being 8mm.
b) Minimum of 8mm Diameter.

Pitch of Transverse Reinforcement shall be the least of the following.
a) The least Lateral Dimension of the Column.
b) 12 Times the Diameter of the smallest Longitudinal Reinforcement.
c) Maximum allowable spacing of 300 mm

Longitudinal Reinforcements:

<table>
<thead>
<tr>
<th>C/S Area of Pile Section.</th>
<th>[ 1.131 \text{ m}^2 ]</th>
<th>[ 11309.73 \text{ cm}^2 ]</th>
</tr>
</thead>
</table>
a) 0.4% C/S Area.          | \[ 11309.73 \times 0.4 = 45.24 \text{ cm}^2 \] |

b) Direct Stress

\[ P_{\text{max}} = \frac{A}{\text{s cbc}} \]

\[ P_{\text{max}} = 3734.25 \text{ kN} = 373425 \text{ Kg} \]

\[ s_{\text{cbc}} = 16.667 \text{ Kg/cm}^2 \]

\[ P_{\text{max}}\ \\ \text{s cbc} = 373425 \times 16.667 \]

\[ = 2240.55 \text{ cm}^2 \]

\[ 0.8 \% \text{ of Min C/S Area}. = 2240.55 \times 0.8 = 17.92 \text{ cm}^2 \]

Assumed % of Longitudinal Reinforcement = 0.611

:. Longitudinal Reinforcement Provided.

\[ = 11309.73 \times 0.611 = 69.09 \text{ cm}^2 \]

Using 20 mm f bars, \[ A_{s1} = 3.142 \text{ cm}^2 \]

No of bars reqd: \[ = \frac{69.09}{3.1415927} = 21.99 \text{ Nos} \]

Provided steel 20 mm dia bars, at 22 mm c/c.

Transverse Reinforcements:

a) Diameter of Transverse Reinforcement

\[ = \frac{1}{4} \times 20 = 5 \text{ mm} \]

b) Minimum Diameter

\[ = 10 \text{ mm} \]

Design of Pilecap :

Provide min reinforcement of 0.06% in the pile cap top

Ast required in the pile cap top 1080 mm²

Provide 16 mm dia bar @ 150 mm c/c in at pile cap top in both directions
Design of reinforcement in transverse dim
This has been designed as cantilever bending due to pile load.
Load coming on three piles (normal case) 8981 kN
Moment, M 14369.6 kNm
Ast required 46922.7 mm²

Design of reinforcement in longitudinal dim
Moment, M 24697.8 kNm
Ast required 123095 mm²
Min Ast required = 0.85bd/fy 28693 mm²

Assume 32 mm dia bars spacing 120
Provide 32 mm bars @120mm c/c in transverse direction
Assume 32 mm dia bars spacing 50
Provide 32 mm bars @50mm c/c in longitudinal direction

Min Stirrup Reinforcement Reqd -
Assume 8 legged 16 mm diameter stirrups
Sv 166.881 mm
Provide 8 legged 16 mm diameter stirrups @ 150 mm C/c

Design of Bed Block
Girder + Deck slab 2394.67
Live Load 289
SIDL 274

Design of Cantilever Portion in the Transverse Direction.
Calculation of B Moments at face of Pier due to:

a) Due to self weight
   Width of Bedblock in long: dir " = 2.5 m
   Cantilevered Length = 2.35 m
   Average depth of cantilever part = 0.9 m
   Volume = 5.288 m³
   U.D.L /m Length = 5.288 x 24 = 126.90 KN
   C.G of Load = 1.044 m
   D.Load Moment at face of Pier = 126.9 x 1.044 = 132.54 KNm

b) Due to Dead load of Girder & Deck Slab
   Moment at face of Pier = 4549.87 Tm

c) Due to S.I.D.Load
   Moment at face of Pier = 520.98 Tm

d) Due to Vehicular Live Load
   Moment at face of Pier = 548.83 kNm
Torsion due to Live load from one side = 71.47 kNm
Equivalent longitudinal moment due to torsion = 62.22 kNm

\[ \Sigma \text{Moments} = 5814.43 \text{ kNm} \]

Effective Depth Reqd **def**  = \( \sqrt{\frac{M}{Q \times B}} \),
Clear Co -  = 50 mm
Bar Dia  = 32 mm

def reqd: \( = \frac{5814.43 \times 1000000}{1.90 \times 2500} \) = 1106.39 cms < 1193.40 mm Hence O.K

def provided = \( = 1.6 \) = 1193.40 mm

Ast reqd = \( = \frac{M}{Q \times B} \times \frac{d}{j} \times d \)

\[ \therefore \text{Ast reqd} = \frac{5814.43 \times 1000000}{200 \times 0.88 \times 1193.40} = 27683 \text{ mm}^2 \]

No of bars Reqd: = 27683 = 34 Nos

Spacing of Bars = 2500 = 73 mm

So provide two layers 32 dia bars, 17 nos

2) Check for Shear at pier face.
Calculation of S. Force at face of Pier due to

a) Due to self weight
def = 1.1934 m
Shear due to self wt: = 48.18 KN

b) D. load of Girder, Deck Slab = 2394.67 KN

c) Due to S.J.D.Load = 274 KN

d) Due to LL = 289 KN

Equivalent shear due to tc = 45.74 kN
S Shear = 3051.65 kN

\[ \text{Shear stress} , \quad \tau = \frac{V}{B \times d} \quad (\text{Vide cl - 304.7.1.1 of I.R.C:-21-2000}) \]

\[ V = \text{The design shear across the section} \]
\[ d = \text{Effective depth of the section} \]
\[ B = \text{Breadth of slab} \]

\[ \therefore \tau = \frac{3051.6 \times 1000}{2500 \times 1193.40} = 1.0 \text{ Mpa} \]

Maximum Permissible Shear Stress :-
\[ \tau_{\text{max}} = 2.3 \text{ Mpa} \]

which ever is less

fck = 35 N/mm²

Calculation of permissible Shear Stress :-
\[ \text{Vide cl - 304.7.3.1 of I.R.C:-21-2000} \]

\[ p = \frac{100 \times As}{B \times d} \quad \frac{As}{As} = 27682.72 \text{ mm}^2 \]

\[ p = 27682.7 \times 100 = 0.928 \]

\[ 2500 \times 1193.40 \]

\[ \therefore \tau_c = 0.4 \text{ N/mm}^2 \]

Since \( \tau > \tau_c \): Shear Reinforcement Reqd:

Shear Reinforcement Reqd :-
\[ \text{Vide cl - 304.7.4.2 of I.R.C:-21-2000} \]

\[ \text{Asw} = \frac{V \times s \times d}{s \times d} \]

\[ \text{V} = 1846.41 \text{ kN} \]

\[ s , \text{ spacing} = 150 \text{ mm (assumed)} \]

\[ \therefore \text{Asw} = \frac{1846.41 \times 1000 \times 150}{200 \times 1193.40} = 1160.4 \text{ mm}^2 \]

Using 12 mm dia stirrups

Using 12 Legged at spacing 150 mm.

\[ \text{Asw} = 1357.2 \text{ cm}^2 \]

Hence O.K

B) Design of Cantilever Portion in the Longitudinal Direction.

The cantilevered portion of Bed block in this direction is very less. Even for one span off condition during construction time the girders are not rested initially over this cantilever portion. So the nominal reinforcement need to be provided.