GOVERNMENT OF ORISSA
WORKS DEPARTMENT

ORISSA STATE ROAD PROJECT

FINAL DETAILED ENGINEERING REPORT
FOR PHASE-I ROADS
DESIGN REPORT OF BRIDGES
(BHAWANIPATNA TO KHARIAR)

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INTRODUCTION
INTRODUCTION

This report presents the design of bridges for Bhawanipatna to Khariar on SH-16. The total width of carriageway is 12.0 m with clear carriageway width of 11.0 m and crash barrier of 0.5 m on each side in minor bridges does not lie in village vicinity. Footpath of 1.5 m shall be provided on both sides for major bridges and minor bridges lies in the village vicinity. The bridge having 7.5 m width in good condition has been retained without any widening as per the discussion and decision with PIU.

There are total 21 Nos of existing bridges in this stretch. The existing and new proposal has been summarized on the next page.

From Ch. 27/600 to 29/400, there are total 6 nos. of bridges on Tel River. 3 Nos. of bridges are towards Bhawanipatna side w.r.t. Main Tel River bridge and 2 bridges are towards Khariar side. As per the drawing received from Executive Engineer Bhawanipatna, HFL observed on 3rd July 2006 was around 1.5 m above bridge deck w.r.t. bridge at Ch. 27/850. The level difference between main bridge on Tel and the above bridge is about 6 m within a distance of around 125 m. As per highway alignment, it is not possible to keep the bridge at the same level. Due to submergence, difficulty in highway alignment, Detailed hydrology report, Review committee meeting and Inspection of Bhawanipatna-Khariar Road followed by Inspection note vide letter no. PIU(WB) 25/2006, it is decided to dismantle the existing bridge at Ch. 27/800 and at Ch. 27/850 and construction of new bridge of around 175 m clear vent way. It is proposed to construct 8 spans of 32.2 m PSC Girder and substructure resting on Pile foundations as per geotechnical and hydrology report. As per the discussion with PIU, raising of other minor bridges at Ch. 27/600, 28/900 and 29/400 has been suggested provided the parameter like SBC and reinforcement mentioned in working drawings are met.

Main Tel bridge has been kept as it is as per detailed inspection and performance of the bridge. For bridge at Ch. 29/400 besides raising, 2 additional spans of 9.2 m on both sides has been added. Initially 2 spans of 32.2 m was proposed for the above bridge.

Detailed analysis of existing bridges to be raised has been made on the basis of modern day loading and dimension detail drawing No. D-III/25/1-78-79 provided by PIU.

In final DPR, rehabilitation measures of bridge at Ch. 69/300 has been proposed as per the review committee inspection note vide letter no. PIU(WB) 25/2006, however as the NDT results are not satisfactory, it is suggested to do core cutting of slab during execution to know the residual strength of concrete.

Slab type superstructure of 10.0 m and RCC girder of 12.0 m, 14.0 m and 21.0 m has been taken from the MOST standard drawings.

Box type structure has also been taken from the MOST – standard drawings.

Substructure of the bridges has been designed as per the program based on latest IRC codes on standard excel sheet-moving live load using standard software STAAD-PRO.

Following live loads has been considered for 12.0 m overall width.

1. Single lane of Class 70-R wheeled+ Class A
2. Single lane of class 70-R Tracked + class A
3. Class-A, 3 lane

The structural drawing has been prepared in AUTOCAD.

Following changes has been done in Final DPR.

- Tel bridge and its approaches designs has been revised as per the actual HFL as per detailed hydrology report and waterway as per the Inspection note vide letter no. PIU(WB) 25/2006 and letter no. PIU (WB)/36/05/part/7227/22/2/07.
- Bridge at Ch. 69/300 reconstruction case in draft DPR has been revised to Rehabilitation case as per the Inspection note vide letter no. PIU(WB) 25/2006.
Discrepancy in Soil report has been taken care as per the comments received on 30.12.06 and compliance on dated 17-01-07.

Bed protection and length of apron has been incorporated as per discussion.

Seismic analysis as per letter no. PIU(WB)43/2006/11973/dated 20\textsuperscript{th} march 2007, has been done in our sample calculation for bridge at Ch. 29/500 in Berhampur-Rayagada road and are not critical for seismic zone II and as per our experience in this regard, hence in other designs it is ignored.

Coherence in list of culverts in the drawing, report and scheduled of culverts has been maintained with proper design chainages as per letter no. PIU (WB)/36/05/part/7227/22/2/07.

Code of Reference

1. IRC : 21 - 2000
2. IRC : 6 - 2004
3. IRC : 78 - 2000
4. IRC : 89 - 1997
5. IRC : 89 - 1997 (Part II)
6. SP : 13 – 2004
7. Specifications for Road and Bridge works (MOST Book)
8. MOST standard plans Culverts with or without Cushion
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<th>Sl. No.</th>
<th>Location/Chainage</th>
<th>Existing Span Arrangement</th>
<th>Existing Carriage way width</th>
<th>Type of Bridge</th>
<th>Recommended for</th>
<th>Proposed Spand Arrangement</th>
<th>Type of foundation</th>
<th>Type of Substructure</th>
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Bridge Design Report
BRIDGE AT CH:13+750
DESIGN OF SUBSTRUCTURE
DESIGN DATA

Formation Level = 235.100 m
Ground Level = 229.452 m
Lowest Water Level = 225.824 m
Highest Flood Level = 232.670 m
Founding Level = 225.824 m
Thickness of bearing & pedestal = 0.300 m
Width of abutment = 12.000 m
Bouyancy factor = 1.0
Safe Bearing Capacity = 86.530 t/sqm
Dry density of earth = 1.800 t/cum
Submerged density of earth = 1.0 t/cum
Saturated density of earth = 2.000 t/cum
Coefficient of base friction = 0.7
Span (c/c of exp. joint) = 14.600 m
Overall Width of deck slab = 12.000 m
Width of carriageway = 11.000 m
Width of crash barrier = 0.500 m
Depth of Superstructure = 1.400 m
Thickness of wearing coat = 0.056 m
Unit wt of concrete = 2.400 t/m$^3$
Grade of Reinforcement = M 25
Grade of Concrete = Fe 415 (HYSD)
Live Load = One Lane of 70R Wheeled + Class A
- 3 lanes of Class A
Permissible Compressive stress in Concrete = 850 t/m$^2$
Permissible Tensile stress in Steel = 20400 t/m$^2$
Modular ratio, m = 10
factor, k = 0.294
Lever arm factor, j = 0.902
Moment of Resistance = 113 t/m$^2$
Thickness of returnwall = 0.5 m

COEFFICIENT OF ACTIVE EARTH PRESSURE

As per Coulomb's theory, the coefficient of active earth pressure is

$$\begin{align*}
K_a &= \frac{\sin^2 \phi \cdot \sin(\alpha - \delta)}{1 + \sqrt{\frac{\sin(\alpha + \phi) \cdot \sin(\phi + \bar{\alpha})}{\sin(\alpha - \delta) \cdot \sin(\phi + \bar{\alpha})}}} \\
\text{WHERE} \\
\phi &= \text{Angle of internal friction of earth} \\
\alpha &= \text{Angle of inclination of back of wall} \\
\delta &= \text{Angle of internal friction between wall & earth} \\
t &= \text{Angle of inclination of backfill} \\
\text{HERE} \\
\phi &= 30^\circ = 0.524 \text{ Radian} \\
\alpha &= 90^\circ = 1.571 \text{ Radian} \\
\delta &= 20^\circ = 0.349 \text{ Radian} \\
t &= 0^\circ = 0 \text{ Radian} \\
K_a &= 0.2973 \\
\end{align*}$$

Therefore, Horizontal coefficient of active earth pressure = $K_a \cos \phi = 0.2794$
HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 9.276 m
For DESIGN purpose, the height of abutment is considered as, say, = 9.280 m

CALCULATION OF ACTIVE EARTH PRESSURE

DRY. condition

a) Service Condition

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<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
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<td>74.64</td>
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<td>334.26</td>
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b) Span Dislodge Condition

Net force = 334.26 - 74.64 = 259.62 t
Net moment = 1357.64 - 346.17 = 1011.48 tm

H.F.L. condition

a) Service Condition

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<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
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b) Span Dislodge Condition

Net force = 284.54 - 74.64 = 209.91 t
Net moment = 1063.01 - 346.17 = 716.84 tm
# Forces & moments due to Abutment (Concrete) components

## DRY Case:

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment about toe (t.m)</th>
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<tbody>
<tr>
<td>1</td>
<td>Toe Slab</td>
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<td>12.00</td>
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## H.F.L. Case:

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<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment about toe (t.m)</th>
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- **H.F.L. Moment about Toe**: 953.47
### Forces & moments due to Earth and LL surcharge

#### DRY Case:
**Self weight of Earth**

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<tr>
<th>Element No</th>
<th>Component</th>
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<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
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#### H.F.L Case:

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<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
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#### L.L.SURCHARGE

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<th>Length</th>
<th>Width</th>
<th>Height</th>
<th>Density</th>
<th>Weight</th>
<th>C.G. from toe</th>
<th>Moment @ Toe</th>
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#### SUMMARY OF FORCES AND MOMENTS:

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<tr>
<th>LOAD CASE</th>
<th>L.W.L.</th>
<th>H.F.L.</th>
</tr>
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<tbody>
<tr>
<td>Vertical load from superstructure including LL</td>
<td>Service Cond.</td>
<td>Span dislodged</td>
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<tr>
<td>Vertical load from substructure (b)</td>
<td>211.04</td>
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<td>Total Vertical Load</td>
<td>1087.41</td>
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<td>Total Horizontal Force (H)</td>
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<td>Moment @ toe due to (a)</td>
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<tr>
<td>Moment @ toe due to (b)</td>
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<td>Total Moment @ toe (M)</td>
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<td>Dist. of C.G. of V from toe (Z) = M/V</td>
<td>3.830</td>
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<td>Earth Pressure</td>
<td>0.530</td>
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<td>Relieving Moment @ c/l base (M1)</td>
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<td>overturning moment due to</td>
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<tr>
<td>Horz. braking force</td>
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<tr>
<td>Total overturning Moment (M2)</td>
<td>1488.36</td>
<td>1193.73</td>
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<td>Net moment (M2-M1) = Ml</td>
<td>800.52</td>
<td>746.53</td>
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**Factor of Safety**

<table>
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<th>Service Cond.</th>
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<th>Service Cond.</th>
<th>Span dislodged</th>
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<td>2.86</td>
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<tr>
<td>Against sliding</td>
<td>2.595</td>
<td>2.091</td>
<td>2.290</td>
<td>Safe against sliding</td>
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I.R.C 78-2000:cl

706.3.4
Area of base (A) = $6.600 \times 12.00 = 79.20$ m$^2$

$Z_L = 87.12$ m$^3$

$Z_T = 158.40$ m$^3$

**CHECK FOR BASE PRESSURE:**

<table>
<thead>
<tr>
<th>Base Pressure</th>
<th>LWL CASE</th>
<th>HFL CASE</th>
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<td></td>
<td>Service Cond.</td>
<td>Span dislodged</td>
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<tr>
<td>P/A</td>
<td>16.39</td>
<td>13.73</td>
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<td>$M_L/Z_L$</td>
<td>9.19</td>
<td>2.02</td>
</tr>
<tr>
<td>$M_T/Z_T$</td>
<td>1.04</td>
<td>0.00</td>
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<tr>
<td>(A) (P/A + $M_L/Z_L + M_T/Z_T$)</td>
<td>26.62</td>
<td>15.75</td>
</tr>
<tr>
<td>(B) (P/A + $M_L/Z_L - M_T/Z_T$)</td>
<td>24.54</td>
<td>15.75</td>
</tr>
<tr>
<td>(C) (P/A - $M_L/Z_L + M_T/Z_T$)</td>
<td>8.25</td>
<td>11.71</td>
</tr>
<tr>
<td>(D) (P/A - $M_L/Z_L - M_T/Z_T$)</td>
<td>6.165</td>
<td>11.71</td>
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</table>

Max. Base Pressure = $26.62$ t/m$^2$ < 86.53 Hence O.K.

Min. Base Pressure = $1.73$ t/m$^2$ > 0 Hence O.K.

**DESIGN OF TOE SLAB**
**BENDING MOMENT AT FACE OF STEM**

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
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<tbody>
<tr>
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Bending Moment at face of stem 46.016 tm/m  
Effective depth required 0.639 m  
Effective depth provided at face of stem 1.115 m  
Area of Reinforcement required 2243 mm$^2$  
Minimum steel required 1673 mm$^2$ I.R.C 78-2000 Clause:707.2.7  
Distribution steel 669 mm$^2$/m  
mainsteel 2243 mm$^2$  

Hence provide,  
25 $\phi$, @ 175 C/C  
0 $\phi$, @ 175 C/C  

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.  
Minimum steel reinforcement as per above clause 250 mm$^2$/m  
provide  
12 $\phi$ , @ 150 C/C  
753.9822

**Check for Shear**

**SHEAR FORCE AT "d" FROM FACE OF STEM**

<table>
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<th>L.A.</th>
<th>Moment</th>
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<td>1.062</td>
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<td>-9.416</td>
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Effective depth (d') at distance d 0.725 m  
Shear force at critical section 20.7 t  
Bending Moment at critical section 9.42 tm  
\[\tan \beta = 0.36\]  
Net shear force S-M*\tan\beta/d' 15.96 t  
Hence, Shear stress 22.02 t/m$^2$  
% of reinforcement 0.39  
Permissible shear stress 27.92 t/m$^2$ Hence O.K.
DESIGN OF HEEL SLAB

BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

<table>
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<th>Element</th>
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<th>L.A.</th>
<th>moment</th>
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<td>4.284</td>
<td>2.267</td>
<td>9.710</td>
</tr>
<tr>
<td></td>
<td>8a</td>
<td>1</td>
<td>49.425</td>
<td>1.700</td>
<td>84.023</td>
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<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>4.284</td>
<td>2.267</td>
<td>4.855</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>Rect.</td>
<td>1</td>
<td>-20.960</td>
<td>1.700</td>
<td>-35.631</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-16.094</td>
<td>1.133</td>
<td>-18.240</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>34.081</td>
<td>65.439</td>
<td></td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 65.439 tm

Effective depth required 0.762 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3191.65 mm$^2$
Minimum steel reinforcement as per above clause 250 mm$^2$/m

Distribution steel 669.00 mm$^2$/m
mainsteel 3191.65 mm$^2$

Hence provide , 12 φ , @ 150 C/C 753.9822
0 φ , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.
Minimum steel reinforcement as per above clause provide 12 φ , @ 150 C/C 753.9822
**Check for Shear**  
(Critical section at face of stem)

Shear force at face of stem 34.08 t

\[
\tan \beta = 0.206
\]

Bending moment at face of stem 65.439 tm

Net shear force \(S - M \times \tan \beta / d\) 22.00 t

Hence, Shear stress 19.73 t/m²

% of reinforcement 0.29

Permissible shear stress 24.63 t/m² Hence O.K.

**DESIGN OF STEM WALL**

![Diagram of stem wall](image)

**SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT**

R.L. OF SECTION = 227.024 m

**DRY Condition**

a) Service Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Area factor</th>
<th>Height of E.P. diagram</th>
<th>Earth Pressure</th>
<th>Force</th>
<th>L.A.</th>
<th>Moment tm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>7.726</td>
<td>0.603</td>
<td>55.9</td>
<td>3.863</td>
<td>216.13</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>7.726</td>
<td>3.885</td>
<td>180.1</td>
<td>3.245</td>
<td>584.44</td>
</tr>
<tr>
<td>HORIZONTAL FORCE</td>
<td></td>
<td>16.00</td>
<td>6.620</td>
<td>105.92</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td>252.06</td>
<td></td>
<td>906.488</td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

= 208.051 + 12.15 + 10.37 + 211.04

= 441.604 t

Longitudinal Moment = 906.488 t-m

Transverse Moment = 164.933 t-m
b) Span Dislodged Condition

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 208.051 + 12.15 + 10.37 + 0.00
= 230.567 t

Longitudinal Moment = 800.568 t-m
Transverse Moment = 0.000 t-m

H.F.L. Condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.603</td>
<td>55.95</td>
<td>3.863</td>
<td>216.13</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>2.024</td>
<td>1.13</td>
<td>13.73</td>
<td>6.321</td>
<td>86.81</td>
</tr>
<tr>
<td>3</td>
<td>SubmgEarth</td>
<td>1.0</td>
<td>5.646</td>
<td>1.13</td>
<td>76.62</td>
<td>2.823</td>
<td>216.31</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>5.646</td>
<td>1.58</td>
<td>53.44</td>
<td>1.882</td>
<td>100.57</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>199.74</td>
<td>619.815</td>
<td></td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 126.749 + 12.15 + 10.37 + 211.04
= 360.301 t

Longitudinal Moment = 725.735 t-m
Transverse Moment = 164.933 t-m

b) Span Dislodged Condition

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 126.749 + 12.15 + 10.37 + 0.00
= 149.265 t

Longitudinal Moment = 619.815 t-m
Transverse Moment = 0.000 t-m

Cross Sectional area = 1.2 m²
For horizontal reinforcement area of steel required for the stem at the section/metre = 173.9186 mm²

Providing 12 ø @ 650.29 c/c say, 150 C/C as horizontal reinforcement
Movement of Deck:
Total Length of Bridge = 14.6 m
= 0.0005 x 7300 = 3.65 mm

**Longitudinal Force:**

Size of bearing = 500 x 250 x 50 mm
Strain in bearing = \( \frac{3.65}{50} = 0.073 \)

Shear modulus = 1.0 Mpa
Shear force per Bearing = 0.073 x 1.0 x 500 x 250 = 9125 N = 0.930 t

Total shear force for 4 bearings (with 5% increase)
= 0.930 x 4 x 1.05 = 3.907 t

Refer IRC : 6 clause 214.5.1.5:
10% increase for variation in movement of span
Total shear force = 1.1 x 3.907 = 4.297 t

As per clause 214.2 of IRC:6, horizontal braking force \( F_h \), for each span is:

**For Class A Single lane**
\[ F_h = 0.2 \times 50 \]
= 10.000 t

**For class 70R wheeled**
\[ F_h = 0.2 \times 100 \]
= 20.000 t

**For class A 3 lane**
\[ F_h = 0.2 \times 50 + 0.05 \times 50 = 12.5 \text{ t} \]

**For class 70R wheeled + class A 1 lane**
\[ F_h = 0.2 \times 100 + 0.05 \times 50 = 22.5 \text{ t} \]

**For class A 3 lane**
Longitudinal horizontal force = \( \frac{12.500 + 4.297}{2} = 10.55 \text{ t} \) Say, 11.0 t

**For class 70R wheeled + class A 1 lane**
Longitudinal horizontal force = \( \frac{22.500 + 4.297}{2} = 15.5 \text{ t} \) Say, 16.0 t

**Span dislodged condition**
Longitudinal horizontal force = \( \frac{0.000 + 4.297}{2} = 4.297 \text{ t} \) Say, 5.0 t

**Summary of Longitudinal Forces:**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 3 lane</td>
<td>11.00</td>
</tr>
<tr>
<td>70R+class A 1 lane</td>
<td>16.00</td>
</tr>
<tr>
<td>Span dislodged condition</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Transverse eccentricity**

Class 70RW+ Class A
\( e = 1.7207 \)
Class A 3 lane

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>e = 0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3</td>
<td>0.7</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load Reaction:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SIDL</td>
<td>24.46</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td>L.L</td>
<td>90.73</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td>70R + Class A</td>
<td>95.85</td>
<td>t</td>
<td>164.93</td>
</tr>
<tr>
<td>Class A 3 Lane</td>
<td>90.93</td>
<td>t</td>
<td>63.65</td>
</tr>
<tr>
<td></td>
<td>211.04</td>
<td></td>
<td>164.93</td>
</tr>
</tbody>
</table>
Summary of Loads at Abutment Shaft bottom:

1 DRY condition

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>With L.L</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Check for Cracked/Uncracked Section

Length of section = 12000 mm
Width of section = 1200 mm
Gross Area of section $A_g = 14400000 \text{ mm}^2$
Gross M.O.I of section $I_{gxx} = 1.728\times10^{12} \text{ mm}^4$
Gross M.O.I of section $I_{gyy} = 1.728\times10^{14} \text{ mm}^4$

Transformed sectional properties of section:
Adopting

- Modular ratio $m = 10$
- Cover 70 68
- Dia of Bars = 20 16
- No of bars in tension face (longer) = 160
- No of bars in compression face = 80
- No of bars in shorter direction = 8
- Total bars in section = 256
Consulting Engineers Group Ltd., Jaipur

Bridge Design Report

Steel Area
\[ A_s = 69567 \, \text{mm}^2 \]
% of Steel
\[ \% = 0.4831 \%

Area of concrete
\[ A_c = A_g - A_s = 14330433 \, \text{mm}^2 \]
C.G of Steel placed on longer face
\[ = 530 \, \text{mm} \]
C.G of Steel placed on shorter face
\[ = 5932 \, \text{mm} \]
Transformed Area of Section
\[ A_{\text{fin}} = 15026107 \, \text{mm}^2 \]

Transformed Moment of Inertia
\[ I_{\text{txx}} = \frac{1}{12} (m - \bar{y}) A_s \bar{x}^2 + 2 \left[ \frac{1}{2} A_c \bar{x}^2 \right] \]
\[ = 1.98215 \times 10^9 \, \text{mm}^4 \]

Permissible stresses
Minimum Gross Moment of Inertia
\[ I_{\text{min}} = 1.728 \times 10^9 \, \text{mm}^4 \]
Area of section
\[ = 14400000 \, \text{mm}^2 \]
Effective length
\[ r = 346.41016 \, \text{mm} \]

Effective length of Abutment shaft
(IRC:21-2000 cl: 306.2.1)

Permissible stresses
Stress reduction coefficient
\[ \beta = 1 \]

Type of member
1 Short Column
2 Long Column
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>DRY Case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loads and Moments With L.L.</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>( P )</td>
<td>441.60 t</td>
</tr>
<tr>
<td>2</td>
<td>( M_{t} )</td>
<td>906.49 t-m</td>
</tr>
<tr>
<td>3</td>
<td>( M_{T} )</td>
<td>164.93 t-m</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

|   | \( \sigma_{\text{co,cal}} \) | \( \frac{P}{A_{\text{ftm}}} \) | 0.293890878 |
|   | \( \sigma_{\text{cbc,cal}} \) | \( \frac{M_{t}}{Z_{xx}} \) | 2.743949739 |
|   | \( \sigma_{\text{cbc,cal}} \) | \( \frac{M_{T}}{Z_{yy}} \) | 0.056932836 |
|   | \( \sigma_{\text{cbc,cal}} = 5 + 6 \) | \( \sigma_{\text{cbc,cal}} \) | 2.800882575 |

**Permissible Stresses**

|   | \( \sigma_{\text{cbc}} \) | 8.3333333 |
|   | \( \sigma_{\text{co}} \) | 6.25 |

**Check for Minimum steel area** mm²

|   | Conc.Area Required for directstress |
|   | \( \frac{(1)}{(9)} \) | 706565.72 |
|   | 0.8% of area required | 5652.5257 |
|   | 0.3% of \( A_{g} \) | 43200 |
|   | Governing steel mm² | 43200 |
|   | Provided Steel area mm² | 69567.428 |

**Check for safety of section**

\[
\frac{\sigma_{\text{co,cal}}}{\sigma_{\text{co}}} + \frac{\sigma_{\text{cbc,cal}}}{\sigma_{\text{cbc}}} < 1
\]

|   | 0.3831284 |

**Check for Cracked /Uncracked section**

\[
\sigma_{\text{co,cal}} - \sigma_{\text{cbc,cal}}
\]

|   | -2.506992 |

Permissible Basic tensile stress in concrete -0.61

Section to be designed as Cracked
Width of Solid return wall \((a)\) = 3.40
Width of Cantilever return wall = 4.00
Avg Height of Solid return wall \((b)\) = 8.426
Height of Cantilever return at Tip = 0.75
Height of Cantilever return at Root = 2.666667
Thickness of Solid Return at farther end = 0.5
Thickness of Solid Return at Root = 0.5
Thickness of Solid Return at bottom = 0.5
Thickness of Solid Return at top = 0.5
Thickness of Cantilever return = 0.5
Unit wt of Soil = 1.8 t/m³
Grade of concrete = M 30
\(\sigma_{\text{cb}}\) = 1020 t/m²
m = 10
\(\sigma_{\text{st}}\) = 20400 t/m²
k = 0.333333
j = 0.888889
R = 151.1111 t/m²

**Case (1) For uniformly distributed load over entire plate**

<table>
<thead>
<tr>
<th>(\frac{a}{b})</th>
<th>0.4035129</th>
<th>0.375</th>
<th>0.353</th>
<th>0.398</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\beta_1)</td>
<td>0.416413</td>
<td>0.5</td>
<td>0.631</td>
<td>0.632</td>
</tr>
<tr>
<td>(\beta_2)</td>
<td>0.451376</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Live Load Surcharge:

\[ q = 0.2794 \times 1.8 \times 1.2 = 0.603504 \text{ t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.4164128 \times 0.603504 \times 71.00}{0.25} = 71.36859 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ Z = \frac{1000 \times 250000}{6} = 0.041667 \text{ m}^3 \]

Hence Moment /m width along Y direction

\[ M_Y /\text{m width} = 71.368586 \times 0.041667 = 2.973691 \text{ t-m/m} \]

\[ \sigma_{a_{\text{max}}} = \frac{0.4513762 \times 0.603504 \times 71.00}{0.25} = 77.36094 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ Z = \frac{1000 \times 250000}{6} = 0.041667 \text{ m}^3 \]

Hence Moment /m width along X direction

\[ M_X /\text{m width} = 77.360938 \times 0.041667 = 3.223372 \text{ t-m/m} \]

Case (2) For Triangular loading due to earth pressure

\[ \frac{a}{b} = 0.4035129 \quad \text{For } \frac{a}{b} = 0.375 \quad \beta_1 = 0.212 \quad \beta_2 = 0.148 \]

\[ \frac{a}{b} = 0.4035129 \quad \beta_1 = 0.328 \quad \beta_2 = 0.200 \]

Earth pressure:

\[ q = 0.2794 \times 1.8 \times 8.426 = 4.237604 \text{ t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.23846 \times 4.237604 \times 71.00}{0.25} = 286.9715 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ Z = \frac{1000 \times 250000}{6} = 0.041667 \text{ m}^3 \]
Hence Moment /m width along Y direction

\[ M_y = \frac{286.97153 \times 0.041667}{0.25} = 11.95715 \ t\cdot m/m \]

\[ \sigma_{\text{max}} = \frac{0.1598614 \times 4.237604 \times 71.00}{6} = 192.3831 \ t/m^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \ mm^3 \]

\[ = 0.041667 \ m^3 \]

Hence Moment /m width along X direction

\[ M_x = \frac{192.38306 \times 0.041667}{0.25} = 8.015961 \ t\cdot m/m \]

Total Moment in Solid Return /m height

Along X-direction

\[ = 11.23933 \ t\cdot m/m \]

Along Y-direction

\[ = 14.93084 \ t\cdot m/m \]

Moment due to Cantilever Return:

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 \]
\[ + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 \]
\[ + 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times X^2 \times dx \times \left( 4.00 - X \right) \]
\[ + 0.2794 \times 1.8 \times 1.95 \times 0.66667 \times X \times dx \times \left( 4.00 - X \right) \]

\[ = 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.6667 \]

\[ = 14.11063 \ t\cdot m \]

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

Effective depth available

\[ = 500 - 50 - 20 - 6 = 424 \ mm \]

\[ M = R \times b \times d^2 \]
\[ = 151.1111 \times 2.666667 \times 0.179776 = 72.4431 \ t\cdot m \]

\[ A_{\text{st}} = \frac{14.11063 \times 10^6}{20400 \times 0.888889 \times 0.424} = 1835.28 \ mm^2 \]

\[ A_{\text{st}}/m = 688.231 \ mm^2/m \]

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm² on earth face.

Provide 12 mm dia @ 180 mm c/c providing 628.3185 mm² on other face.

Along Horizontal direction.
Design of Solid Return:

Moment due to Cantilever Return:

\[
M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 5.40 \\
+ 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 5.40 \\
+ 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times X^2 \times dx \times \left(7.40 - X\right) \\
+ 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left(7.40 - X\right)
\]

\[
= 9.776765 + 3.055239 + 0.11176 \times 93.86667 + 0.653796 \times 37.8667 \\
= \text{48.07962} \ t-m
\]

Moment in Solid Return /m height 
\[
= 11.23933 + \frac{48.07962}{8.426} = \text{16.9454} \ t-m/m
\]

Moment in Solid Return /m width  
\[
= \text{14.9308} \ t-m/m
\]

Design of face B-B'

Moment in Solid Return /m height
\[
= \text{16.9454} \ t-m/m
\]

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 20 - 10 = 420 mm

\[
M = R \times b \times d^2 \\
= 151.1111 \times 1 \times 0.1764 = \text{26.656} \ t-m
\]

\[
A_{st} = 16.94544 \times 10^6 \\
\frac{20400 \times 0.888889 \times 0.42}{2224.98} = \text{2224.98} \ mm^2
\]

\[
A_{at}/m = \text{2224.98} \ mm^2/m
\]

Provide 20 mm dia @ 150 mm c/c providing 2094.395 mm² on earth face.

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm² on other face.

Along Horizontal direction.

Design of face A'-B'

Moment in Solid Return /m width
\[
= \text{14.9308} \ t-m/m
\]

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 0 - 10 = 440 mm

\[
M = R \times b \times d^2 \\
= 151.1111 \times 1 \times 0.1936 = \text{29.2551} \ t-m
\]

\[
A_{st} = 14.93084 \times 10^6 \\
\frac{20400 \times 0.888889 \times 0.44}{2224.98} = \text{1871.35} \ mm^2
\]

\[
A_{at}/m = \text{1871.35} \ mm^2/m
\]

Provide 20 mm dia @ 160 mm c/c providing 1963.495 mm² on earth face.

Provide 12 mm dia @ 160 mm c/c providing 706.8583 mm² on other face.

Along Vertical direction.
1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure \( K_a = 0.279384 \)

FOR NORMAL CASE, \( F_a = 1.0 \)

unit wt of soil \( \gamma = 1.8 \)

\( \delta = 20 \)

clear span between return wall = 11.000 m

width of return wall = 0.5

Avg. cover to reinforcement. (FOR 2-3 LAYERS) = 0.150 m

\( H \) (From formation level) = 8.076 m

DESIGN FOR BENDING MOMENT

<table>
<thead>
<tr>
<th>S.NO.</th>
<th>UNITS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>H (FROM TOP)</td>
<td>m 8.076</td>
</tr>
<tr>
<td>WIDTH OF RETURN AT THIS LEVEL</td>
<td>m 3.400</td>
</tr>
<tr>
<td>FORCE DUE TO EARTH PRESSURE</td>
<td>t 92.464</td>
</tr>
<tr>
<td>MOMENT DUE TO EARTH PRESSURE</td>
<td>tm 313.631</td>
</tr>
<tr>
<td>FORCE DUE TO L.L. SURCHARGE</td>
<td>t 27.478</td>
</tr>
<tr>
<td>MOMENT DUE TO L.L. SURCHARGE</td>
<td>tm 110.957</td>
</tr>
<tr>
<td>TOTAL MOMENT</td>
<td>tm 424.588</td>
</tr>
<tr>
<td>DESIGN MOMENT</td>
<td>tm 424.588</td>
</tr>
<tr>
<td>REQUIRED EFFECTIVE DEPTH</td>
<td>m 2.650</td>
</tr>
<tr>
<td>EFF. DEPTH AVAILABLE</td>
<td>m 3.250</td>
</tr>
<tr>
<td>AREA OF STEEL REQUIRED</td>
<td>cm(^2) 72.036</td>
</tr>
<tr>
<td>DIAMETER OF BAR PROVIDED</td>
<td>mm 32</td>
</tr>
<tr>
<td>TOTAL NO. OF BARS</td>
<td>no. 12</td>
</tr>
<tr>
<td>AREA OF STEEL PROVIDED</td>
<td>cm(^2) 96.51</td>
</tr>
<tr>
<td></td>
<td>O.K.</td>
</tr>
<tr>
<td></td>
<td>34.0</td>
</tr>
</tbody>
</table>

CHECK FOR SHEAR STRESS

| SHEAR FORCE | t 124.942 |
| SHEAR STRESS | t/m\(^2\) 76.888 |
| Ast/bd x 100 | % 0.443 |
| PERMISSIBLE SHEAR STRESS | MPa 0.29 |
| PERMISSIBLE SHEAR STRESS | t/m\(^2\) 29.77 |
| Asv/sv | cm\(^2\)/m 7.107 |
| \( k_1 \) | ... 0.5 |
| \( k_2 \) | ... 1.0 |
| PERMISSIBLE SHEAR STRESS | t/m\(^2\) 10 Tor 2-legged 1|

1 Tor 2-legged
Design of Abutment Cap:

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl: 710.8.2 of IRC:78-2000 is 200 mm.

However the thickness of abutment cap is = 300 mm
Assuming a cap thickness of = 300 mm

Volume of Abutment cap = 0.3 x 1.20 x 12 = 4.32 m³

Quantity of steel = 1 % of volume = 1 x 4.32

= 0.0432 m³

Quantity of steel to be provided at top = 0.0216 m³
Quantity of steel to be provided at bottom = 0.0216 m³

Top & bottom face:

Quantity of steel to be provided in Longitudinal dir = 0.0108 m³
Assuming a clear cover of = 50 mm
Length of bar 12.00 - 0.100 = 11.9 m

Area of steel required in Longitudinal direction

\[ \frac{0.0108}{11.9} = 907.563 \text{ mm}^2 \]

Provide 9 nos of bars 12 mm dia at top & bottom face.

= 1017.88 mm²

Transverse steel:

Quantity of steel to be provided in Longitudinal dir = 0.0108 m³
Assuming a clear cover of = 50 mm
Assuming a dia of bar = 12 mm
Length of bar 1.20 - 0.100 = 1.1 m

Volume of each stirrup = 0.00012 m³

no of stirrups required for m/length = 8 nos
Required Spacing = \( \frac{1000}{8} \)

= 125 mm

Provide 12 mm dia bar 125 mm c/c stirrups through in length of abutment cap.

904.779 mm²
Design of Dirt wall:
Dirt wall designed as a vertical cantilever.

Intensity for rectangular portion = \( 0.2794 \times 2.00 \times 1.2 = 0.6706 \, \text{t/m}^2 \)
F1 = \( 0.67056 \times 12.00 \times 1.41 = 11.31369 \, \text{t} \)

Intensity for triangular portion = \( 0.2794 \times 2.00 \times 1.41 = 0.7857 \, \text{t/m}^2 \)
F2 = \( 0.785673 \times 12.00 \times 1.41 = 6.627936 \, \text{t} \)
M1 = \( 6.627936 \times 0.59052 = 3.913929 \, \text{t-m} \)
M2 = \( 11.86745 \, \text{t-m} \)
Total moment at base of dirt wall /m length = \( 0.988954 \, \text{t-m/m} \)

Thickness of dirtwall = 0.3 m
Assuming a clear cover on either face = 50 mm

**Vertical steel on earth face:**
dia of steel bar = 12 mm
Available effective depth = 300 - 50 - 6 = 244 mm

Effective depth req = \( 0.988954 \times 1.51 \times 1000 = 80.92815 \, \text{mm} \)
Ast req = \( 0.988954 \times 200 \times 0.889 \times 244 = 227.9579 \, \text{mm}^2/\text{m} \)

Minimum steel = \( 0.12 \times 300 \times 1000 = 360 \, \text{mm}^2/\text{m} \)
Provide 12 mm dia bar 200 mm c/c as vertical steel at earth face.

**Distribution steel on earth face:**
dia of steel bar = 12 mm
Available effective depth = 300 - 50 - 12 = 238 mm

\( 0.3M = 0.3 \times 0.988954 = 0.296686 \, \text{t-m/m} \)
Ast req = \( 0.296686 \times 200 \times 0.889 \times 238 = 70.11142 \, \text{mm}^2/\text{m} \)

Minimum steel as per IRC:21-200 cl:305.10 = 250 mm²/m
Governing steel at earth face
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

**Vertical steel on earth face**
As per IRC:21-200 cl:305.10 All faces provide minimum steel of
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

**Distribution steel:**
As per IRC:21-200 cl:305.10 All faces provide minimum steel of
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.
Bridge Design Report

7.400 0.3 0.900
235.100
0.75
11
1.756
slop V1:H 1.5
2.667 233.344
10 0.300
8d 0.000
1.20 233.044
4.000 0.00
0.000 233.044
H.F.L 232.67
3.400
9b
6.020
L.W.L
225.824
5
8c
6.020
227.024 1.20 0.00 227.024
0.700 8 9 0.700
4 2 Founding 7 Lvl
3.400 1.200 2.000
6.60
Wind Force Calculation

Span c/c of Abutment 14.60 m

Section details

Deck level at abutment 235.100 m
Wearing coat + Girder depth + Bearing thickness + Pedestal ht. 1.552 m
Abutment cap level 233.548 m
Foundation level adopted 225.900 m
Foundation top level 227.100
Average bed level 229.420 m
Height of dirt wall 1.552 m
Thickness of abutment cap 0.300 m
Ht. of abutment from foundation base to top of cap 233.548-225.900 7.648 m
Thickness of foundation slab at junction 1.200 m
Thickness of foundation slab at edge 0.500 m
Net ht. of abutment from top of fnd. slab to bottom of cap 7.648-1.200-0.300 6.148 m
Ht. of back fill from bottom of foundation 235.10-225.90 9.20 m
Surcharge height as per IRC:78-2000, clause 710.4.4 1.20 m
Thickness of dirt wall (as per clause 710.6.4 of IRC:78-2000) 0.300 m
Width of abutment cap 0.430+0.320+0.300 1.200 m
Height of return wall above top of foundation slab 235.100-225.900-1.200 8.000 m

Wind Forces

On live load

Intensity of wind force on moving live load 300 kg/m
Exposed length of live load 14.60 m
Horizontal wind force 43.8 kN
Height of action of this force above deck 1.5 m
Height of action of this force from top of foundation 9.500 m
Height of action of this force from bottom of foundation 10.700 m

On deck

Girder depth + crash barrier 2.45 m
Exposed area of deck per span 17.89 m²
Average height above foundation top 9.325 m
Wind pressure at this height 88.00 kg/m²
Wind force on deck per span 15.74 kN

Total on superstructure

Wind force on loaded superstructure 29.77 kN
Min. force on loaded superstructure @ 450 kg/m 32.85 kN
Hence applicable wind force on loaded superstructure 32.85 kN
Height of combined c.g. of the forces above foundation top 9.454 m
Height of combined c.g. of the forces at foundation bottom 10.654 m
Level of action of this force 236.554 m
Wind force on unloaded superstructure 15.74 kN
Min. force on unloaded superstructure @ 240 kg/m² 42.92 kN
Hence applicable wind force on unloaded superstructure 42.92 kN
Height of c.g. of the forces above foundation top 9.32 m
Height of c.g. of the forces at foundation bottom 10.53 m
Level of action of this force 236.43 m
**On substructure**

Exposed area of abutment & cap \( 4.95 \text{ m}^2 \)

Average height above top of foundation \( 5.584 \text{ m} \)

Wind pressure at this height \( 71.00 \text{ kg/m}^2 \)

Wind force on substructure \( 3.52 \text{ kN} \)

Average height from bottom of foundation \( 5.584 \text{ m} \)

Average height from bottom of abutment shaft \( 4.384 \text{ m} \)

<table>
<thead>
<tr>
<th>Moment at the bottom of foundation</th>
<th>Total</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded condition</td>
<td>369.61</td>
<td>369.61</td>
<td>0.00 kN.m</td>
</tr>
<tr>
<td>Unloaded condition</td>
<td>471.41</td>
<td>471.41</td>
<td>0.00 kN.m</td>
</tr>
</tbody>
</table>

Transverse moment due to wind forces are say 47 t·m.

Moments at the bottom of abutment shaft bottom

| Loaded condition                  | 325.97  | 325.97     |
| Unloaded condition                | 415.69  | 415.69     |

Transverse moment due to wind forces are say 42 t·m.
INPUT FILE: 70r.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
9. 1 0.00 0 0; 2 0.3 0 0; 3 14.3 0 0; 4 14.6 0 0
10. MEMBER INCIDENCES
11. 1, 2, 3
12. MEMBER PROPERTY CANADIAN
13. 1 TO 3 PRI YD 1.0 ZD 1.0
14. CONSTANT
15. E CONCRETE ALL
16. DENSITY CONCRETE ALL
17. POISSON CONCRETE ALL
18. SUPPORT
19. 2 3 PINNED
20. DEFINE MOVING LOAD
21. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
22. LOAD GENERATION 175
23. TYPE 1 -13.4 0.0 0.0 XINC .2
24. PERFORM ANALYSIS
25. LOAD LIST 74
26. PRINT SUPPORT REACTION
   SUPPORT REACTION

SUPPORT REACTIONS —UNIT MTON METE   STRUCTURE TYPE = PLANE
----------------
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM Z
2 74 0.00 34.46 0.00 0.00 0.00 0.00
3 74 0.00 65.54 0.00 0.00 0.00 0.00

27. FINISH
INPUT FILE: CLASS A.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
9. 1 0.00 0 0; 2 0.3 0 0; 3 14.3 0 0; 4 14.6 0 0
10. MEMBER INCIDENCES
11. 1 1 2 3
12. MEMBER PROPERTY CANADIAN
13. 1 TO 3 PRI YD 1.0 ZD 1.0
14. CONSTANT
15. E CONCRETE ALL
16. DENSITY CONCRETE ALL
17. POISSON CONCRETE ALL
18. SUPPORT
19. 2 3 PINNED
20. DEFINE MOVING LOAD
21. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
22. LOAD GENERATION 202
23. TYPE 1 -18.8 0 0 XINC .2
24. PERFORM ANALYSIS
25. LOAD LIST 74
26. PRINT SUPPORT REACTION
   SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE
-----------------
  JOINT  LOAD  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM-Z
  2   74 0.00 30.31 0.00 0.00 0.00 0.00
  3   74 0.00 19.69 0.00 0.00 0.00 0.00

************* END OF LATEST ANALYSIS RESULT *************

27. FINISH
Abutment SHAFT ..DRY NORMAL

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Section</td>
<td>1.200 m</td>
</tr>
<tr>
<td>Width of Section</td>
<td>12.000 m</td>
</tr>
<tr>
<td>along width-compression face</td>
<td></td>
</tr>
<tr>
<td>- no of bar: compression face</td>
<td>80</td>
</tr>
<tr>
<td>- no of bar: tension face</td>
<td>160</td>
</tr>
<tr>
<td>Dia (mm)</td>
<td>16</td>
</tr>
<tr>
<td>Cover (cm)</td>
<td>7.50</td>
</tr>
<tr>
<td>along depth-compression face</td>
<td></td>
</tr>
<tr>
<td>- no of bar: compression face</td>
<td>8</td>
</tr>
<tr>
<td>- no of bar: tension face</td>
<td>8</td>
</tr>
<tr>
<td>Dia (mm)</td>
<td>16</td>
</tr>
<tr>
<td>Cover (cm)</td>
<td>7.50</td>
</tr>
<tr>
<td>Modular Ratio : Compression</td>
<td>10.0</td>
</tr>
<tr>
<td>Modular Ratio : Tension</td>
<td>10.0</td>
</tr>
<tr>
<td>Allowable Concrete Stress</td>
<td>85.00 Kg/cm^2</td>
</tr>
<tr>
<td>Allowable Steel Stress</td>
<td>2040.00 Kg/cm^2</td>
</tr>
<tr>
<td>Axial Load</td>
<td>441.604 T</td>
</tr>
<tr>
<td>Mxx</td>
<td>906.488 Tm</td>
</tr>
<tr>
<td>Myy</td>
<td>164.933 Tm</td>
</tr>
<tr>
<td>Intercept of Neutral axis : X axis</td>
<td>192.893 m</td>
</tr>
<tr>
<td>: y axis</td>
<td>0.330 m</td>
</tr>
</tbody>
</table>

Concrete Stress Governs Design

Stress in Concrete due to Loads = 57.84 Kg/cm^2
Stress in Steel due to Loads = 1374.71 Kg/cm^2
Percentage of Steel = 0.48%

Abutment SHAFT ..DRY SPAN DISLODGED

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Section</td>
<td>1.200 m</td>
</tr>
<tr>
<td>Width of Section</td>
<td>12.000 m</td>
</tr>
<tr>
<td>Modular Ratio : Compression</td>
<td>10.0</td>
</tr>
<tr>
<td>Modular Ratio : Tension</td>
<td>10.0</td>
</tr>
<tr>
<td>Allowable Concrete Stress</td>
<td>85.00 Kg/cm^2</td>
</tr>
<tr>
<td>Allowable Steel Stress</td>
<td>2040.00 Kg/cm^2</td>
</tr>
<tr>
<td>Axial Load</td>
<td>230.567 T</td>
</tr>
<tr>
<td>Mxx</td>
<td>800.568 Tm</td>
</tr>
<tr>
<td>Myy</td>
<td>0.000 Tm</td>
</tr>
<tr>
<td>Intercept of Neutral axis : X axis</td>
<td>****** m</td>
</tr>
<tr>
<td>: y axis</td>
<td>0.294 m</td>
</tr>
</tbody>
</table>
Steel Stress Governs Design

Stress in Concrete due to Loads = 49.21 Kg/cm^2
Stress in Steel due to Loads = 1341.50 Kg/cm^2
Percentage of Steel = .48 %

Abutment SHAFT ..HFL NORMAL

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 360.301 T
Mxx = 725.735 Tm
Myy = 164.933 Tm

Intercept of Neutral axis : X axis : = 156.141 m
: y axis : = .334 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 46.67 Kg/cm^2
Stress in Steel due to Loads = 1098.35 Kg/cm^2
Percentage of Steel = .48 %

Abutment SHAFT ..HFL SPAN DISLODGED

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 149.265 T
Mxx = 619.815 Tm
Myy = .000 Tm

Intercept of Neutral axis : X axis : = ******* m
: y axis : = .288 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 37.99 Kg/cm^2
Stress in Steel due to Loads = 1064.81 Kg/cm^2
Percentage of Steel = .48 %
DESIGN OF SUPERSTRUCTURE
For Design of Superstructure of RCC Girder 14.0 m span refer MOST STANDARD Drawing titled “STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and Slab Superstructure)” Drg. No. SD/250 to SD/256.
BRIDGE AT CH:21+000
DESIGN OF SUBSTRUCTURE
**DESIGN DATA**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Formation Level</td>
<td>221.200 m</td>
</tr>
<tr>
<td>Ground Level</td>
<td>217.178 m</td>
</tr>
<tr>
<td>Lowest Water Level</td>
<td>210.678 m</td>
</tr>
<tr>
<td>Highest Flood Level</td>
<td>218.807 m</td>
</tr>
<tr>
<td>Founding Level</td>
<td>210.678 m</td>
</tr>
<tr>
<td>Thickness of bearing &amp; pedestal</td>
<td>0.300 m</td>
</tr>
<tr>
<td>Width of abutment</td>
<td>12.000 m</td>
</tr>
<tr>
<td>Bouyancy factor</td>
<td>1.0</td>
</tr>
<tr>
<td>Safe Bearing Capacity</td>
<td>45.300 t/sqm</td>
</tr>
<tr>
<td>Dry density of earth</td>
<td>1.800 t/cum</td>
</tr>
<tr>
<td>Submerged density of earth</td>
<td>1.0 t/cum</td>
</tr>
<tr>
<td>Saturated density of earth</td>
<td>2.000 t/cum</td>
</tr>
<tr>
<td>Coefficient of base friction</td>
<td>0.5</td>
</tr>
<tr>
<td>Span (c/c of exp. joint)</td>
<td>12.600 m</td>
</tr>
<tr>
<td>Overall Width of deck slab</td>
<td>12.000 m</td>
</tr>
<tr>
<td>Width of carriageway</td>
<td>11.000 m</td>
</tr>
<tr>
<td>Width of crash barrier</td>
<td>0.500 m</td>
</tr>
<tr>
<td>Depth of Superstructure</td>
<td>1.200 m</td>
</tr>
<tr>
<td>Thickness of wearing coat</td>
<td>0.056 m</td>
</tr>
<tr>
<td>Unit wt of concrete</td>
<td>2.400 t/m³</td>
</tr>
<tr>
<td>no. of elastomeric bearing</td>
<td>4</td>
</tr>
<tr>
<td>size of elastomer brgs.</td>
<td>500 x 250 x 50 mm</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M 25</td>
</tr>
<tr>
<td>Grade of Reinforcement</td>
<td>415 (HYSD)</td>
</tr>
<tr>
<td>Live Load</td>
<td>One Lane of 70R Wheeled + Class A</td>
</tr>
<tr>
<td></td>
<td>3 lanes of Class A</td>
</tr>
<tr>
<td>Permissible Compressive stress in Concrete</td>
<td>850 t/m²</td>
</tr>
<tr>
<td>Permissible Tensile stress in Steel</td>
<td>20400 t/m²</td>
</tr>
<tr>
<td>Modular ratio, m</td>
<td>10</td>
</tr>
<tr>
<td>factor, k</td>
<td>0.294</td>
</tr>
<tr>
<td>Lever arm factor, j</td>
<td>0.902</td>
</tr>
<tr>
<td>Moment of Resistance</td>
<td>113 t/m²</td>
</tr>
<tr>
<td>Thickness of returnwall</td>
<td>0.5 m</td>
</tr>
</tbody>
</table>

**COEFFICIENT OF ACTIVE EARTH PRESSURE**

As per Coulomb’s theory, coefficient of active earth pressure is:

\[ K_a = \frac{\sin^2(\alpha + \phi)}{1 + \sqrt{\frac{\sin(\alpha - \delta) \cdot \sin(\phi - \tau)}{\sin(\alpha - \delta) \cdot \sin(\phi + \tau)}}} \]

Where:
- \( \phi \) = Angle of internal friction of earth
- \( \alpha \) = Angle of inclination of back of wall
- \( \delta \) = Angle of internal friction between wall & earth
- \( \tau \) = Angle of inclination of backfill

Here:
- \( \phi = 30^\circ \) = 0.524 Radian
- \( \alpha = 90^\circ \) = 1.571 Radian
- \( \delta = 20^\circ \) = 0.349 Radian
- \( \tau = 0^\circ \) = 0 Radian

\[ K_a = 0.2973 \]

Therefore, horizontal coefficient of active earth pressure = \( K_a \cos \phi = \frac{K_a}{2} \approx 0.2794 \)
HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 10.522 m
For DESIGN purpose, the height of abutment is considered as, say, = 10.520 m

CALCULATION OF ACTIVE EARTH PRESSURE

DRY. condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>84.66</td>
<td>5.261</td>
<td>445.41</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>10.522</td>
<td>5.29</td>
<td>334.06</td>
<td>4.419</td>
<td>1476.28</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>1.0</td>
<td>0.000</td>
<td>5.29</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td>SubmergedEarth</td>
<td>0.5</td>
<td>0.000</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>418.72</td>
<td>1921.69</td>
<td></td>
</tr>
</tbody>
</table>

b) Span Dislodge Condition

Net force = 418.72 - 84.66 = 334.06 t
Net moment = 1921.69 - 445.41 = 1476.28 tm

H.F.L. condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>84.66</td>
<td>5.261</td>
<td>445.41</td>
</tr>
<tr>
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<td></td>
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<td>1451.07</td>
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b) Span Dislodge Condition

Net force = 345.07 - 84.66 = 260.40 t
Net moment = 1451.07 - 445.41 = 1005.66 tm
### Forces & moments due to Abutment (Concrete) components

#### DRY Case:

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G.from toe (m)</th>
<th>Moment about toe</th>
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**TOTAL** 627.67   2217.09

#### H.F.L. Case:

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<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G.from toe (m)</th>
<th>Moment @ Toe</th>
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**TOTAL** 366.76   1309.84
Forces & moments due to Earth and LL surcharge

**DRY Case :** Self weight of Earth

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<th>Element No</th>
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<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
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**H.F.L Case :**

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<th>Height (m)</th>
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**L.L.SURCHARGE**

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<th>Height (m)</th>
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<th>C.G. from toe (m)</th>
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**SUMMARY OF FORCES AND MOMENTS:**

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<th>LOAD CASE</th>
<th>Case. L.W.L.</th>
<th>Case. H.F.L.</th>
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<tr>
<td>Vertical load from superstructure including LL</td>
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<td>181.78</td>
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<tr>
<td>Vertical load from substructure (b)</td>
<td>1317.09</td>
<td>825.59</td>
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<tr>
<td>Total Vertical Load V = (a) + (b)</td>
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<td>825.59</td>
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<td>Total Horizontal Force H =</td>
<td>429.72</td>
<td>260.40</td>
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<tr>
<td>Moment @ toe due to (a)</td>
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<td>Moment @ toe due to (b)</td>
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<td>Total Moment @ toe (M)</td>
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<td>Dist. of C.G. of V from toe Z = M/V</td>
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<td>Relieving Moment @ c/l base (M1)</td>
<td>1116.50</td>
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<td>overturning moment due to Horz. braking force</td>
<td>105.78</td>
<td>0.00</td>
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<td>Earth Pressure</td>
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<td>1005.66</td>
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<td>Total overturning Moment ( M2)</td>
<td>2027.47</td>
<td>1005.66</td>
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<td>Net moment ( M2-M1) = M1</td>
<td>910.96</td>
<td>828.54</td>
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<td>Factor of Safety</td>
<td>Safe against overturning</td>
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<td>Against overturning ( M / M2)</td>
<td>3.250</td>
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<tr>
<td>Against sliding ( µ x V / H )</td>
<td>1.744</td>
<td>1.585</td>
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I.R.C 78-2000:cl
706.3.4
Area of base \((A)\) = 7.300 \(\times\) 12.00 = 85.20 \(\text{m}^2\)

\[Z_L = 106.58 \text{ m}^3\]
\[Z_T = 175.20 \text{ m}^3\]

**CHECK FOR BASE PRESSURE:**

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<tr>
<th>Base Pressure</th>
<th>LWL CASE</th>
<th>HFL CASE</th>
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<td>Span dislodged</td>
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<tr>
<td>(P/A)</td>
<td>17.59</td>
<td>15.46</td>
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<tr>
<td>(M_L/Z_L)</td>
<td>8.55</td>
<td>2.44</td>
</tr>
<tr>
<td>(M_T/Z_T)</td>
<td>0.87</td>
<td>0.00</td>
</tr>
<tr>
<td>((A)) ((P/A + M_L/Z_L + M_T/Z_T))</td>
<td>27.01</td>
<td>17.90</td>
</tr>
<tr>
<td>((B)) ((P/A + M_L/Z_L - M_T/Z_T))</td>
<td>25.27</td>
<td>17.90</td>
</tr>
<tr>
<td>((C)) ((P/A - M_L/Z_L + M_T/Z_T))</td>
<td>9.91</td>
<td>13.02</td>
</tr>
<tr>
<td>((D)) ((P/A - M_L/Z_L - M_T/Z_T))</td>
<td>8.177</td>
<td>13.02</td>
</tr>
</tbody>
</table>

Max. Base Pressure = 27.01 \(\text{t/m}^2\) < 45.30 Hence O.K.
Min. Base Pressure = 3.18 \(\text{t/m}^2\) > 0 Hence O.K.

**DESIGN OF TOE SLAB**
### BENDING MOMENT AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
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<tbody>
<tr>
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<tr>
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<td></td>
<td>-51.231</td>
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</tbody>
</table>

Bending Moment at face of stem 62.031 tm/m

Effective depth required 0.742 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3024 mm$^2$

Minimum steel required 1673 mm$^2$ IRC 78-2000 Clause:707.2.7

Distribution steel

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<tr>
<th>Area of Reinforcement</th>
<th>Then provide</th>
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<td>Rect.</td>
<td>669 mm$^2$/m</td>
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<tr>
<td>mainsteel</td>
<td>3024 mm$^2$</td>
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<td></td>
<td>12 φ @ 150 C/C</td>
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<tr>
<td></td>
<td>753.9822</td>
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Hence provide, 25 φ @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinforcement as per above clause 250 mm$^2$/m

Provide 12 φ @ 150 C/C 753.9822

### Check for Shear

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
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<td>-17.171</td>
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Effective depth (d') at distance d 0.776 m

Shear force at critical section 28.2 t

Bending Moment at critical section 17.17 tm

$\tan \beta = 0.34$

Net shear force S-M*\tan\beta/d' 20.72 t

Hence, shear stress 26.71 t/m$^2$

% of reinforcement 0.42

Permissible shear stress 29.05 t/m$^2$ Hence O.K.
DESIGN OF HEEL SLAB

![Diagram of heel slab with dimensions and loadings]

BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads.</td>
<td>3</td>
<td>1</td>
<td>4.320</td>
<td>1.800</td>
<td>7.776</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>3.024</td>
<td>1.200</td>
<td>3.629</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.5</td>
<td>4.536</td>
<td>2.400</td>
<td>10.886</td>
</tr>
<tr>
<td></td>
<td>8a</td>
<td>1</td>
<td>60.407</td>
<td>1.800</td>
<td>108.732</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>4.536</td>
<td>2.400</td>
<td>5.443</td>
</tr>
<tr>
<td></td>
<td>8a</td>
<td>1</td>
<td>7.468</td>
<td>1.800</td>
<td>13.442</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>Rect.</td>
<td>1</td>
<td>-29.438</td>
<td>1.800</td>
<td>-52.989</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-15.174</td>
<td>1.200</td>
<td>-18.209</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>39.678</td>
<td></td>
<td>78.710</td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 78.710 \(\text{tm}\)

Effective depth required 0.836 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3838.91 mm\(^2\)
Minimum steel 1672.50 mm\(^2\) I.R.C 78-2000 Clause:707.2.7

Distribution steel 669.00 mm\(^2\)/m
mainsteel 3838.91 mm\(^2\)

Hence provide:

\[25 \phi \text{ , @ } 125 \text{ C/C} \]

\[753.9822 \]

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinforcement as per above clause 250 mm\(^2\)/m

Provide:

\[12 \phi \text{ , @ } 150 \text{ C/C} \]

753.9822
Check for Shear

Shear force at face of stem 39.68 t

\[ \tan \beta = 0.194 \]

Bending moment at face of stem 78.710 tm

Net shear force \( S - M \tan \beta / d \)

25.95 t

Hence, Shear stress \( 23.27 \text{ t/m}^2 \)

% of reinforcement 0.34

Permissible shear stress \( 26.52 \text{ t/m}^2 \)

Hence O.K.

DESIGN OF STEM WALL

SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT

R.L. OF SECTION = 211.878 m

DRY Condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Area factor</th>
<th>Height of E.P. diagram</th>
<th>Earth Pressure</th>
<th>Force</th>
<th>L.A.</th>
<th>Moment tm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>8.972</td>
<td>0.603</td>
<td>65.0</td>
<td>4.486</td>
<td>291.46</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>8.972</td>
<td>4.512</td>
<td>242.9</td>
<td>3.788</td>
<td>915.25</td>
</tr>
<tr>
<td>HORIZONTAL FORCE</td>
<td></td>
<td></td>
<td>11.00</td>
<td>8.066</td>
<td></td>
<td>88.73</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>318.86</td>
<td></td>
<td></td>
<td>1295.44</td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

= 301.029 + 10.42 + 10.37 + 181.78

= 503.592 t

Longitudinal Moment = 1295.444 t-m

Transverse Moment = 152.042 t-m
b) Span Dislodged case

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 301.029 + 10.42 + 10.37 + 0.00
= 321.817 t

Longitudinal Moment = 1206.718 t-m
Transverse Moment = 0.000 t-m

H.F.L. Condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.603</td>
<td>64.97</td>
<td>4.486</td>
<td>291.46</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>1.987</td>
<td>1.11</td>
<td>13.24</td>
<td>7.591</td>
<td>100.48</td>
</tr>
<tr>
<td>3</td>
<td>SubmgEarth</td>
<td>1.0</td>
<td>6.929</td>
<td>1.11</td>
<td>92.32</td>
<td>3.465</td>
<td>319.83</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>6.929</td>
<td>1.94</td>
<td>80.48</td>
<td>2.310</td>
<td>185.88</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>251.01 897.66</td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 169.888 + 10.42 + 10.37 + 181.78
= 372.451 t

Longitudinal Moment = 986.389 t-m
Transverse Moment = 152.042 t-m

b) Span Dislodged case

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure
= 169.888 + 10.42 + 10.37 + 0.00
= 190.676 t

Longitudinal Moment = 897.663 t-m
Transverse Moment = 0.000 t-m

Cross Sectional area = 1.3 m²
For horizontal reinforcement area of steel required for the stem at the section/metre = 215.3876 mm²

Providing 12 @ 525.09 c/c say, 150 C/C as horizontal reinforcement
Movement of Deck:
Total Length of Bridge = 12.6 m
= 0.0005 x 6300 = 3.15 mm

Longitudinal Force:
Size of bearing = 500 x 250 x 50 mm
Strain in bearing = 3.15 x 50 = 0.063
Shear modulus = 1.0 Mpa
Shear force per Bearing = 0.063 x 1.0 x 500 x 250 = 7875 N
= 0.803 t

Total shear force for 4 bearings (with 5% increase)
= 0.803 x 4 x 1.05 = 3.372 t

Refer IRC: 6 clause 214.5.1.5;
10% increase for variation in movement of span
Total shear force = 1.1 x 3.372 = 3.709 t

As per clause 214.2 of IRC:6, horizontal braking force $F_h$, for each span is:
For Class A Single lane: $F_h = [0.2 \times 28.3] = 5.666$ t
For Class 70R wheeled: $F_h = [0.2 \times 60.5] = 12.106$ t
For Class A 3 lane: $F_h = 0.2 \times 28.3 + 0.05 \times 28.3 = 7.085$ t
For Class 70R wheeled + Class A 1 lane:
For Class A 3 lane
Longitudinal horizontal force $= \frac{7.085 + 3.709}{2} = 7.25$ t Say, 8.0 t

For class 70R wheeled + Class A 1 lane
Longitudinal horizontal force $= \frac{13.523 + 3.709}{2} = 10.5$ t Say, 11.0 t

Span dislodged condition
Longitudinal horizontal force $= \frac{0.000 + 3.709}{2} = 3.709$ t Say, 4.0 t

Summary of Longitudinal Forces:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 3 lane</td>
<td>8.00</td>
</tr>
<tr>
<td>70R+Class A 1 lane</td>
<td>11.00</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>4.00</td>
</tr>
</tbody>
</table>

Transverse eccentricity
Class 70RW + Class A

\[ e = \frac{3.095 \times 6.84}{2} = 1.711 \]

Class A 3 lane
### Summary of Dead load & Live loads from Superstructure (STAAD Pro)

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead Load Reaction</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SIDL</td>
<td>71.81</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td>L.L. Max Reaction Case</td>
<td>21.10</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td>70R + Class A</td>
<td>88.86</td>
<td>t</td>
<td>152.04</td>
</tr>
<tr>
<td>Class A 3 Lane</td>
<td>84.99</td>
<td>t</td>
<td>59.49</td>
</tr>
<tr>
<td></td>
<td><strong>181.78</strong></td>
<td></td>
<td><strong>152.04</strong></td>
</tr>
</tbody>
</table>
**Summary of Loads at Abutment Shaft bottom:**

<table>
<thead>
<tr>
<th>1 DRY condition</th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>With L.L.</td>
<td>503.59</td>
<td>1295.44</td>
<td>152.04</td>
</tr>
</tbody>
</table>

**Check for Cracked/Uncracked Section**

- Length of section = 12000 mm
- Width of section = 1200 mm
- Gross Area of section $A_g$ = 14400000 mm²
- Gross M.O.I of section $I_{gxx}$ = 1.728E+12 mm⁴
- Gross M.O.I of section $I_{gyy}$ = 1.728E+14 mm⁴

---

**Transformed sectional properties of section:**

Adopting

- Modular ratio $m$ = 10
- Cover = 70 68
- Dia of Bars = 20 16
- No of bars in tension face (longer) = 200
- No of bars in compression face = 100
- No of bars in shorter direction = 8
- Total bars in section = 316
Steel Area \( A_s \) = 86155 \( \text{mm}^2 \)

% of Steel = 0.5983 \%

\[ A_{sx} = 62832 \ \text{mm}^2 \]
\[ A_{sy} = 1608.5 \ \text{mm}^2 \]

Area of concrete \( A_c = A_g - A_s \) = 14313845 \( \text{mm}^2 \)

C.G of Steel placed on longer face = 530 mm

C.G of Steel placed on shorter face = 5932 mm

Transformed Area of Section \( A_{tm} \) = 15175395 \( \text{mm}^2 \)

\[ \text{Transformed } M.I_{xx} = I_{gxx} + 2 \left( m - \frac{1}{2} A_s \right) a x^2 \]
\[ = 2.04569E+12 \ \text{mm}^4 \]

\[ Z_{xx} = \frac{M.I_{txx}}{d/2} = 3.409E+09 \ \text{mm} \]

\[ \text{Transformed } M.I_{yy} = I_{gyy} + 2 \left( m - \frac{1}{2} A_s \right) a y^2 \]
\[ = 1.73819E+14 \ \text{mm}^4 \]

\[ Z_{yy} = \frac{M.I_{tyy}}{d/2} = 2.897E+10 \ \text{mm}^3 \]

**Permissible stresses**

Minimum Gross Moment of inertia \( I_{\text{min}} \) = 1.728E+12 \( \text{mm}^4 \)

Area of section \( r \) = 14400000 \( \text{mm}^2 \)

Effective length of Abutment shaft (IRC:21-2000 cl: 306.2.1)

Abutment shaft height \( L \) = 9.426 m

Effective length \( L_{\text{eff}} \) = 11.311 m

Slenderness ratio = 32.653 < 50

Type of member = 1

1 Short Column

2 Long Column

**Stress reduction coefficient** (IRC:21-2000 cl: 306.4.2,3) \( \beta = 1 \)

**Permissible stresses : concrete**

\[ \sigma_{\text{cbc}} = 8.3333 \ \text{N/mm}^2 \]
\[ \sigma_{\text{co}} = 6.25 \ \text{N/mm}^2 \]

Tensile stress = 0.61 \( \text{N/mm}^2 \)

**Permissible stresses : Steel**

\[ \sigma_{\text{st}} = 200 \ \text{N/mm}^2 \]
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>DRY Case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loads and Moments</td>
<td>With L.L</td>
</tr>
<tr>
<td>1</td>
<td>P</td>
<td>503.59</td>
</tr>
<tr>
<td>2</td>
<td>M&lt;sub&gt;L&lt;/sub&gt;</td>
<td>1295.44</td>
</tr>
<tr>
<td>3</td>
<td>M&lt;sub&gt;T&lt;/sub&gt;</td>
<td>152.042</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>σ&lt;sub&gt;co,cal&lt;/sub&gt; = P/A&lt;sub&gt;dim&lt;/sub&gt;</td>
<td>0.331847713</td>
</tr>
<tr>
<td>5</td>
<td>σ&lt;sub&gt;cbc,cal&lt;/sub&gt; = M&lt;sub&gt;L&lt;/sub&gt;/Z&lt;sub&gt;xx&lt;/sub&gt;</td>
<td>3.799531164</td>
</tr>
<tr>
<td>6</td>
<td>σ&lt;sub&gt;cbc,cal&lt;/sub&gt; = M&lt;sub&gt;T&lt;/sub&gt;/Z&lt;sub&gt;yy&lt;/sub&gt;</td>
<td>0.052483082</td>
</tr>
<tr>
<td>7</td>
<td>σ&lt;sub&gt;cbc,cal&lt;/sub&gt; = 5 + 6</td>
<td>3.852014246</td>
</tr>
</tbody>
</table>

**Permissible Stresses**

<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>σ&lt;sub&gt;cbc&lt;/sub&gt;</td>
<td>8.3333333</td>
</tr>
<tr>
<td>9</td>
<td>σ&lt;sub&gt;co&lt;/sub&gt;</td>
<td>6.25</td>
</tr>
</tbody>
</table>

**Check for Minimum steel area mm<sup>2</sup>**

<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Conc.Area Required for directstress</td>
<td>(1)/(9) 805747.24</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
<td>6445.9779</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of A&lt;sub&gt;t&lt;/sub&gt;</td>
<td>43200</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel mm&lt;sup&gt;2&lt;/sup&gt;</td>
<td>43200</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area mm&lt;sup&gt;2&lt;/sup&gt;</td>
<td>86155.037</td>
</tr>
</tbody>
</table>

**Check for safety of section**

\[
\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}} < 1
\]

**Check for Cracked /Uncracked section**

\[
\sigma_{co,cal} - \sigma_{cbc,cal} = -3.520167
\]

Permissible Basic tensile stress in concrete: -0.61

Section to be designed as: **Cracked**
Width of Solid return wall \( (a) \) = 3.60
Width of Cantilever return wall = 4.00
Avg Height of Solid return wall \( (b) \) = 9.672
Height of Cantilever return at Tip = 0.75
Height of Cantilever return at Root = 2.666667
Thickness of Solid Return at farther end = 0.5
Thickness of Solid Return at Root = 0.5
Thickness of Solid Return at bottom = 0.5
Thickness of Solid Return at top = 0.5
Thickness of Cantilever return = 0.5
Unit wt of Soil = 1.8 \( \text{t/m}^3 \)
Grade of concrete = M 30

\( \sigma_{obc} = 1020 \ \text{t/m}^2 \)
\( m = 10 \)
\( \sigma_{oi} = 20400 \ \text{t/m}^2 \)
\( k = 0.333333 \)
\( j = 0.888889 \)
\( R = 151.1111 \ \text{t/m}^2 \)

Case (1) For uniformly distributed load over entire plate

\[ a/b = 0.3722084 \]
For \( a/b = 0.25 \)
\[ \beta_1 = 0.182 \quad \beta_2 = 0.188 \]
For \( a/b = 0.375 \)
\[ \beta_1 = 0.353 \quad \beta_2 = 0.398 \]

\[ a/b = 0.3722084 \]
\[ \beta_1 = 0.349181 \]
\[ \beta_2 = 0.39331 \]
Live Load Surcharge:

\[ q = 0.2794 \times 1.8 \times 1.2 = 0.603504 \text{ t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.3491811 \times 0.603504 \times 93.55}{0.25} = 78.85396 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ = 0.041667 \text{ m}^3 \]

Hence Moment /m width along Y direction

\[ M_Y /\text{m width} = 78.853959 \times 0.041667 = 3.285582 \text{ t-m/m} \]

\[ \sigma_{a_{\text{max}}} = \frac{0.3933102 \times 0.603504 \times 93.55}{0.25} = 88.81941 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ = 0.041667 \text{ m}^3 \]

Hence Moment /m width along X direction

\[ M_X /\text{m width} = 88.819413 \times 0.041667 = 3.700809 \text{ t-m/m} \]

Case (2) For Triangular loading due to earth pressure

\[ a/b = 0.3722084 \]

For a/b = 0.25, \[ \beta_1 = 0.133, \beta_2 = 0.09 \]

For a/b = 0.375, \[ \beta_1 = 0.212, \beta_2 = 0.148 \]

Earth pressure:

\[ q = 0.2794 \times 1.8 \times 9.672 = 4.864242 \text{ t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.2102357 \times 4.864242 \times 93.55}{0.25} = 382.6611 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]

\[ = 0.041667 \text{ m}^3 \]
Hence Moment /m width along Y direction

\[ M_{Y/m\text{ width}} = 382.66108 \times 0.041667 = 15.94421 \text{ t-m/m} \]

\[ \sigma_{\text{max}} = 0.1467047 \times 4.864242 \times 93.55 \times 0.25 \]

\[ = 267.0249 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \text{ mm}^3 \]

\[ = 0.04167 \text{ m}^3 \]

Hence Moment /m width along X direction

\[ M_{X/m\text{ width}} = 267.02494 \times 0.041667 = 11.12604 \text{ t-m/m} \]

Total Moment in Solid Return /m height

\[ = 14.82685 \text{ t-m/m} \]

Along X-direction

Total Moment in Solid Return /m width

\[ = 19.22979 \text{ t-m/m} \]

Along Y-direction

Moment due to Cantilever Return:

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 \]

\[ + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 \]

\[ + 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times 4.00 - X \]

\[ + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times 4.00 - X \]

\[ = 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.66667 \]

\[ = 14.11063 \text{ t-m} \]

Design of cantilever Return:

Assuming 50 mm cover and 12 mm dia bars.

Effective depth available = 500 - 50 - 20 - 6 = 424 mm

\[ M = R \times b \times d^2 \]

\[ = 151.1111 \times 2.666667 \times 0.179776 = 72.44307 \text{ t-m} \]

\[ A_u = 14.11063 \times 10^6 \times 0.424 \]

\[ = 1835.283 \text{ mm}^2 \]

\[ A_u/m = \frac{20400 \times 0.888889 \times 0.424}{2} = 688.231 \text{ mm}^2/m \]

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm² on earth face.

Provide 12 mm dia @ 180 mm c/c providing 628.3185 mm² on other face.

Along Horizontal direction.
Design of Solid Return:

Moment due to Cantilever Return:

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 5.60 + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 5.60 + 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times x^2 \times dx \times \left( \frac{7.60}{X} - X \right) + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left( \frac{7.60}{X} - X \right) \]

\[ = 10.13887 + 3.168396 + 0.11176 \times 98.13333 + 0.653796 \times 39.46667 \]

\[ = 50.07779 \text{ t-m} \]

Moment in Solid Return /m height = \[ \frac{14.82685 + 50.07779}{9.672} \text{ t-m/m} \]

Moment in Solid Return /m width = 19.22979 t-m/m

Design of face B-B:

Moment in Solid Return /m height = 20.00445 t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 20 - 13 = 418 mm

\[ M = R \times b \times d^2 \]

\[ = 151.1111 \times 1 \times 0.174306 \text{ = 26.33961 t-m} \]

\[ A_{st} = \frac{20.00445 \times 10^6}{20400 \times 0.888889 \times 0.4175} \text{ = 2642.363 mm}^2 \]

\[ A_{sf/m} = \frac{2642.363}{264.2363} \text{ mm}^2/m \]

Provide 25 mm dia @ 150 mm c/c providing 3272.492 mm$^2$ on earth face.
Provide 16 mm dia @ 150 mm c/c providing 1340.413 mm$^2$ on other face.

Along Horizontal direction.

Design of face A-B:

Moment in Solid Return /m width = 19.22979 t-m/m

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 0 - 13 = 438 mm

\[ M = R \times b \times d^2 \]

\[ = 151.1111 \times 1 \times 0.191406 \text{ = 28.92361 t-m} \]

\[ A_{st} = \frac{19.22979 \times 10^6}{20400 \times 0.888889 \times 0.4375} \text{ = 2423.924 mm}^2 \]

\[ A_{sf/m} = \frac{2423.924}{264.2363} \text{ mm}^2/m \]

Provide 25 mm dia @ 160 mm c/c providing 3067.962 mm$^2$ on earth face.
Provide 16 mm dia @ 160 mm c/c providing 1256.637 mm$^2$ on other face.

Along Vertical direction.
1. DESIGN FOR FORCES IN LONGITUDINAL DIRECTION

Active earth pressure \( K_a \) = 0.279384

FOR NORMAL CASE, \( FA \) = 1.0

unit wt of soil \( \gamma \) = 1.8

\( \delta \) = 20

clear span between return wall = 11.000 m

width of return wall = 0.5

Avg. cover to reinforcement. (FOR 2-3 LAYERS) = 0.150 m

\( H \) (From formation level) = 9.322 m

DESIGN FOR BENDING MOMENT

<table>
<thead>
<tr>
<th>S.NO.</th>
<th>UNITS</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (FROM TOP)</td>
<td>m</td>
<td>9.322</td>
</tr>
<tr>
<td>WIDTH OF RETURN AT THIS LEVEL</td>
<td>m</td>
<td>3.600</td>
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<tr>
<td>FORCE DUE TO EARTH PRESSURE</td>
<td>t</td>
<td>123.197</td>
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<tr>
<td>MOMENT DUE TO EARTH PRESSURE</td>
<td>tm</td>
<td>482.344</td>
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<td>FORCE DUE TO L.L. SURCHARGE</td>
<td>t</td>
<td>31.718</td>
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<td>MOMENT DUE TO L.L. SURCHARGE</td>
<td>tm</td>
<td>147.836</td>
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<td>TOTAL MOMENT</td>
<td>tm</td>
<td>630.180</td>
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<td>DESIGN MOMENT</td>
<td>tm</td>
<td>630.180</td>
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<tr>
<td>REQUIRED EFFECTIVE DEPTH</td>
<td>m</td>
<td>3.229</td>
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<td>EFF. DEPTH AVAILABLE</td>
<td>m</td>
<td>3.450</td>
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<tr>
<td>AREA OF STEEL REQUIRED</td>
<td>cm²</td>
<td>100.720</td>
</tr>
<tr>
<td>DIAMETER OF BAR PROVIDED</td>
<td>mm</td>
<td>32</td>
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<tr>
<td>TOTAL NO. OF BARS</td>
<td>no.</td>
<td>16</td>
</tr>
<tr>
<td>AREA OF STEEL PROVIDED</td>
<td>cm²</td>
<td>128.68</td>
</tr>
</tbody>
</table>

O.K.

27.8

CHECK FOR SHEAR STRESS

| SHEAR FORCE | t | 159.914 |
| SHEAR STRESS | t/m² | 92.704 |
| Ast/bd x 100 | % | 0.584 |
| PERMISSIBLE SHEAR STRESS | MPa | 0.33 |
| PERMISSIBLE SHEAR STRESS | t/m² | 33.67 |
| Asv/sv | cm²/m | 8.387 |
Design of Abutment Cap:

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl: 710.8.2 of IRC:78-2000 is 200 mm.

However the thickness of abutment cap is = 300 mm
Assuming a cap thickness of = 300 mm
Volume of Abutment cap = 0.3 x 1.20 x 12
= 4.32 m³
Quantity of steel = 1 % of volume
= 1/100 x 4.32
= 0.0432 m³
Quantity of steel to be provided at top = 0.0216 m³
Quantity of steel to be provided at bottom = 0.0216 m³

Top & bottom face:
Quantity of steel to be provided in Longitudinal direction = 0.0108 m³
Assuming a clear cover of = 50 mm
Length of bar = 12.00 - 0.100 = 11.9 m
Area of steel required in Longitudinal direction


Provide 9 nos of bars 12 mm dia at top & bottom face.

= 1017.88 mm²

Transverse steel:
Quantity of steel to be provided in Longitudinal direction = 0.0108 m³
Assuming a clear cover of = 50 mm
Assuming a dia of bar = 12 mm
Length of bar = 1.20 - 0.100 = 1.1 m

Volume of each stirrup = 0.00012 m³

no of stirrups required for m/length = 8 nos
Required Spacing = 1000
= 125 mm

Provide 12 mm dia bar 125 mm c/c stirrups throught in length of abutment cap.

904.779 mm²
Design of Dirt wall:
Dirt wall designed as a vertical cantilever.

Intensity for rectangular portion = \(0.2794 \times 2.00 \times 1.2\) = 0.67056 t/m²

\(F_1\) = \(0.67056 \times 12.00 \times 1.21\) = 9.704344 t

Intensity for triangular portion = \(0.2794 \times 2.00 \times 1.21\) = 0.673913 t/m²

\(F_2\) = \(0.673913 \times 2.00 \times 1.21\) = 4.876433 t

\(M_1\) = \(9.704344 \times 0.60\) = 5.85172 t-m

\(M_2\) = \(4.876433 \times 0.50652\) = 2.470011 t-m

\(M_1 + M_2\) = 8.32173 t-m

Total moment at base of dirt wall /m length = 0.693478 t-m/m

Thickness of dirtwall = 0.3 m

Assuming a clear cover on either face = 50 mm

**Vertical steel on earth face:**
dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 6 = 244 mm

effective depth req = \(0.693478 \times \frac{1.51}{1000}\) = 67.76848 mm

Ast req = \(0.693478 \times \frac{200}{0.889 \times 244}\) = 159.8493 mm²/m

Minimum steel = \(0.12 \times \frac{300}{100}\) x 1000 = 360 mm²/m

Provide 12 mm dia bar 200 mm c/c as vertical steel at earth face.

565.4867 mm²/m

**Distribution steel on earth face:**
dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 12 = 238 mm

\(0.3M\) = \(0.3 \times 0.693478\) = 0.208043 t-m/m

Ast req = \(0.208043 \times \frac{200}{0.889 \times 238}\) = 49.16374 mm²/m

Minimum steel as per IRC:21-200 cl:305.10 = 250 mm²/m

Governing steel at earth face
Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm²/m

**Vertical steel on earth face**

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm²/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm²/m

**Distribution steel:**
As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm²/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm²/m
INPUT FILE: 70r.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
9. 1 0.00 0 0; 2 0.3 0 0; 3 14.3 0 0; 4 14.6 0 0
10. MEMBER INCIDENCES
11. 1 1 2 3
12. MEMBER PROPERTY CANADIAN
13. 1 TO 3 PRI YD 1.0 ZD 1.0
14. CONSTANT
15. E CONCRETE ALL
16. DENSITY CONCRETE ALL
17. POISSON CONCRETE ALL
18. SUPPORT
19. 2 3 PINNED
20. DEFINE MOVING LOAD
21. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
22. LOAD GENERATION 175
23. TYPE 1 -13.4 0.0 0 DIS XINC .2
24. PERFORM ANALYSIS
25. LOAD LIST 74
26. PRINT SUPPORT REACTION
   SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE
------------------

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
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<td>34.46</td>
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<td>74</td>
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<td>0.00</td>
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<td>0.00</td>
</tr>
</tbody>
</table>

27. FINISH
INPUT FILE: CLASS A.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
   9. 1 0.00 0 0;2 0.3 0 0;3 14.3 0 0;4 14.6 0 0
9. MEMBER INCIDENCES
10. 1 1 2 3
11. MEMBER PROPERTY CANADIAN
12. 1 TO 3 PRI YD 1.0 ZD 1.0
13. CONSTANT
14. E CONCRETE ALL
15. DENSITY CONCRETE ALL
16. POISSON CONCRETE ALL
17. SUPPORT
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
20. LOAD GENERATION 202
21. TYPE 1 -18.8 0. 0. XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 74
24. PRINT SUPPORT REACTION
25. SUPPORT REACTION
26. SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
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<th>MOM-Y</th>
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<tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

************** END OF LATEST ANALYSIS RESULT **************

27. FINISH
Abutment SHAFT ..DRY NORMAL

Depth of Section = 1.400 m
Width of Section = 12.000 m

along width-compression face- no of bar: 100  tension face- no of bar: 200
Dia (mm)  16  20
Cover (cm)  7.50  10.5

along depth-compression face- no of bar: 8  tension face- no of bar: 8
Dia (mm)  16  16
Cover (cm)  7.50  7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 503.592 T
Mxx = 1295.444 Tm
Myy = 152.042 Tm

Intercept of Neutral axis : X axis : = 248.283 m
: y axis : = .390 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 57.50 Kg/cm^2
Stress in Steel due to Loads = 1360.23 Kg/cm^2
Percentage of Steel = .51 %

Abutment SHAFT ..DRY SPAN DISLOGED

Depth of Section = 1.400 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 321.817 T
Mxx = 1206.718 Tm
Myy = .000 Tm

Intercept of Neutral axis : X axis : = ****** m
: y axis : = .358 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 51.94 Kg/cm^2
Stress in Steel due to Loads = 1358.48 Kg/cm^2
Percentage of Steel = .51 %
Abutment SHAFT ..HFL NORMAL

Depth of Section = 1.400 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 372.451 T
Mxx = 986.389 Tm
Myy = 152.042 Tm

Intercept of Neutral axis : X axis = 189.496 m
: y axis = .391 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 44.09 Kg/cm^2
Stress in Steel due to Loads = 1046.55 Kg/cm^2
Percentage of Steel = .51 %

Abutment SHAFT ..HFL SPAN DISLODGED

Depth of Section = 1.400 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 190.676 T
Mxx = 897.663 Tm
Myy = .000 Tm

Intercept of Neutral axis : X axis = ****** m
: y axis = .349 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 38.47 Kg/cm^2
Stress in Steel due to Loads = 1044.81 Kg/cm^2
Percentage of Steel = .51 %
DESIGN OF SUPERSTRUCTURE
For Design of Superstructure of RCC Girder 12.0 m span refer MOST STANDARD Drawing titled “STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and Slab Superstructure)” Drg. No. SD/260 to SD/266.
BRIDGE AT CH:27+600 & 28+900
STABILITY CHECK & DESIGN OF SUBSTRUCTURE (ABUTMENT PIER)
**Design Data:**
For design purposes, following parameters have been considered.

- **Grade of concrete:** M - 20
- **Abutment Cap:** M - 25
- **Grade of reinforcement steel:** Fe - 415
- **Centre to Centre distance of A / Expansion joints:** 9.200 m
- **Centre to Centre distance of Bearing:** 8.800 m
- **Depth of superstructure:** 660 mm
- **Thickness of wearing coat:** 56.00 mm
- **Formation level along C of carriage way:** 216.189 m
- **Soffit level:** 215.448
- **Pedestal top level:** 215.473
- **Height of bearing and Pedestal:** 0.025 m
- **L.W.L./Bed level:** 208.272 m
- **H.F.L:** 214.172 m
- **M.S.L:** 205.452 m
- **Founding Level:** 205.834 m
- **Abutment cap top level:** 215.448 m

**Live Load**

- **(a)** Class A two Lane
- **(b)** Class 70R wheeled

**Bearing** neoprene but during raising Tar paper bearing may be kept.

**Seismic zone** = II

The following codes are used for the design of substructure:

1. IRC : 6 - 2000
2. IRC : 21 - 2000
3. IRC : 78 - 2000
Longitudinal translation due to creep, shrinkage & temperature = 0.0005
Horizontal movement = 0.0005 x 4.60 x 1000 = 2.300 mm

**Longitudinal Force:**
- Size of bearing = 400 x 250 x 50 mm
- Strain in bearing = \( \frac{2.300}{50} = 0.046 \)
- Shear modulus = 1.0 Mpa
- Shear force per Bearing = 0.046 x 1.0 x 400 x 250 = 4600 N = 0.469 t

Total shear force for 4 bearings (with 5% increase)
= 0.469 x 4 x 1.05 = 1.969 t

Refer IRC : 6 clause 214.5.1.5;
10% increase for variation in movement of span
Total shear force = 1.1 x 1.969 = 2.166 t

As per clause 214.2 of IRC:6, horizontal braking force \( F_h \), for each span is:
**For Class A 2 lane** : \( F_h = 0.2 \times 49.5 = 9.896 \) t
**For class 70R wheeled** : \( F_h = 0.2 \times 47.72 = 9.544 \) t

<table>
<thead>
<tr>
<th>Summary of Longitudinal Forces:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case</td>
</tr>
<tr>
<td>Class A 2 lane</td>
</tr>
<tr>
<td>70R</td>
</tr>
<tr>
<td>Span dislodged condition</td>
</tr>
</tbody>
</table>
### Dry Condition with L.L

**a) Vertical load and their moments about C/L of Foundation base.**

<p>| | | | | | | |</p>
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<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>1</td>
<td>D.L. Reaction</td>
<td>P</td>
<td>e_L</td>
<td>M_L</td>
<td>e_T</td>
<td>M_T</td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>58.00</td>
<td>0.225</td>
<td>13.05</td>
<td>0</td>
<td>0</td>
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<tr>
<td>2</td>
<td>S.I.D.L.</td>
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<tr>
<td>a</td>
<td>Left span</td>
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<td>2.97</td>
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<td><strong>71.2</strong></td>
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<td>4</td>
<td>L.L.</td>
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<tr>
<td>a</td>
<td>Left span</td>
<td>70R Wheeled Class A 2 Lane</td>
<td>47.72</td>
<td>0.225</td>
<td>10.737</td>
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<td>49.48</td>
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<td><strong>118.92</strong></td>
<td><strong>26.757</strong></td>
<td><strong>52.492</strong></td>
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</table>

#### Substructure

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<tr>
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<td>e_L</td>
<td>M_L</td>
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<td>Left span</td>
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<td>3</td>
<td>Abut shaft</td>
<td>up to G.L</td>
<td>14.461</td>
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<td>below G.L</td>
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<td>8.9762</td>
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<td>48.103</td>
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<td>5</td>
<td>Earth above footing</td>
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<td>1.8</td>
<td>92.408</td>
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<td>7</td>
<td>Earth wt on heel side</td>
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**b) Horizontal Forces and Moments with respect to Base**

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<table>
<thead>
<tr>
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<tbody>
<tr>
<td>1</td>
<td>Longitudinal Forces at bearing level</td>
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<td>H_T</td>
<td>e_L</td>
<td>M_L</td>
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</table>

**Summary**

- P = 321.75 t
- M_L = 131 t-m
- M_T = 52.492 t-m
- A = 30.813
- Z_L = 37.232
- Z_T = 21.826
Check for Maximum Allowable Base Pressure:

\[ P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} \]

\[ = \frac{321.75}{30.813} + \frac{131.26}{37.232} + \frac{52.492}{21.826} = 16.373 \text{ t/m}^2 \]

\[ P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} \]

\[ = \frac{321.75}{30.813} - \frac{131.26}{37.232} - \frac{52.492}{21.826} = 4.5116 \text{ t/m}^2 \]

Design of Toe slab

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area factor</th>
<th>Force L.A</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1</td>
<td>1.017</td>
<td>1.4125</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>2.7798</td>
<td>0.9417</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>3</td>
<td>1</td>
<td>-11.22</td>
<td>1.4125</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>-3.8808</td>
<td>0.9417</td>
</tr>
</tbody>
</table>
Net bending moment at face of stem = 15.449 t-m/m
Effective depth required = 0.4453 m
Effective depth Provided at face of stem = 0.885 m
Area of reinforcement required = 933.49 mm²
Minimum steel req = 1327.5 mm²
Provide 16 φ 140 mm c/c = 1436.2 mm²

Shear
Effective depth d of footing = 0.887 m
Total depth of section at distance d from Abutment = 0.7125 m
Effective depth at distance d from face of Abutment = 0.6215 m

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area factor</th>
<th>Force</th>
<th>L.A</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1</td>
<td>0.6977</td>
<td>0.969</td>
<td>0.6761</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>1.907</td>
<td>0.646</td>
<td>1.2319</td>
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<tr>
<td>Upward base pressure</td>
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<td>1</td>
<td>-12.083</td>
<td>0.969</td>
<td>-11.708</td>
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<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>-1.8264</td>
<td>0.646</td>
<td>-1.1798</td>
</tr>
</tbody>
</table>

Effective depth at a distance of d eff = 0.6215 m
Shear at critical section = 11.30 t
Bending moment at critical section = 10.98 t-m
\( \tan(\beta) = 0.423 \)
Net shear force = 3.83 t
Shear stress = 6.1617 t/m²
% of reinforcement = 0.2311
Permissible shear stress = 26.279 t/m²
NO SHEAR REINFORCEMENT REQ

Design of Heel slab

Heel

Base pressure diagram
<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area factor</th>
<th>Force</th>
<th>L.A</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
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<td>1.017</td>
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<td>2.7798</td>
<td>0.9417</td>
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<tr>
<td></td>
<td>earth</td>
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<td>51.892</td>
<td>1.4125</td>
<td>73.298</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>3</td>
<td>1</td>
<td>-6.9166</td>
<td>1.4125</td>
<td>-9.7697</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>-3.8808</td>
<td>0.9417</td>
<td>-3.6544</td>
</tr>
</tbody>
</table>

Net bending moment at face of stem = 63.928 t-m/m  
Effective depth required = 0.9058 m  
Effective depth at face of stem = 0.9175 m  
Area of reinforcement required = 3726 mm²  
Minimum steel required = 1376.3 mm²  
Provide 25 mm c/c = 3927 mm²  

Shear

Effective depth d of footing = 0.8825 m  
Total depth of section at distance d from Abutment = 0.7138 m  
Effective depth at distance d from face of Abutment = 0.6138 m

width b = 4.25

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area factor</th>
<th>Force</th>
<th>L.A</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1</td>
<td>0.6977</td>
<td>0.969</td>
<td>0.6761</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>1.907</td>
<td>0.646</td>
<td>1.2319</td>
</tr>
<tr>
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<td>earth</td>
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<td>35.599</td>
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</tr>
<tr>
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<td>1</td>
<td>-6.9166</td>
<td>0.969</td>
<td>-6.7022</td>
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<td>4</td>
<td>0.5</td>
<td>-1.8264</td>
<td>0.646</td>
<td>-1.1798</td>
</tr>
</tbody>
</table>

Effective depth at a distance of d eff = 0.6138 m  
Shear at critical section = 29.46 t  
Bending moment at critical section = 28.52 t-m  
\(\tan(\beta) = 0.423\)

Net shear force = 9.80 t  
Shear stress = 15.967 t/m²  
% of reinforcement = 0.6397  
Permissible shear stress = 31.411 t/m²  

NO SHEAR REINFORCEMENT REQ
### Dry Condition with L.L

#### a) Vertical load and their moments about Abutment Shaft bottom

<table>
<thead>
<tr>
<th>Substructure</th>
<th>V</th>
<th>p</th>
<th>P</th>
<th>e&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>e&lt;sub&gt;T&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left span</td>
<td>0.0096</td>
<td>2.4</td>
<td>0.0231</td>
<td>0.225</td>
<td>0.0052</td>
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<td>0</td>
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<tr>
<td>2 Abut cap</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>R</td>
<td>1.7888</td>
<td>2.4</td>
<td>4.293</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>Abut cap</td>
<td></td>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Tr</td>
<td>5.37</td>
<td>2.4</td>
<td>12.888</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Abutment shaft</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>up to G.L</td>
<td>14.461</td>
<td>2.4</td>
<td>34.706</td>
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<td>below G.L</td>
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<tr>
<td>4 Earth pressure</td>
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</tr>
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<td>60.886</td>
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<td>280</td>
<td>280</td>
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#### b) Horizontal Forces and Moments with respect to Abutment Shaft bottom

<table>
<thead>
<tr>
<th>Longitudinal Forces at bearing level</th>
<th>H&lt;sub&gt;L&lt;/sub&gt;</th>
<th>H&lt;sub&gt;T&lt;/sub&gt;</th>
<th>e&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>e&lt;sub&gt;T&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6.94</td>
<td>0</td>
<td>8.669</td>
<td>60.149</td>
<td>8.644</td>
<td>0</td>
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</tbody>
</table>

**Summary for design of Abutment**

- **P** = 179.81 t
- **M<sub>L</sub>** = 367.35 t-m
- **M<sub>T</sub>** = 52.492 t-m
### H.F.L Condition with L.L

**a) Vertical load and their moments about C/L of Foundation base.**

<table>
<thead>
<tr>
<th></th>
<th>D.L. Reaction</th>
<th>P</th>
<th>e&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>e&lt;sub&gt;T&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>a Left span</td>
<td>58.000</td>
<td>0.225</td>
<td>13.050</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>S.I.D.L</td>
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<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>13.200</td>
<td>0.225</td>
<td>2.970</td>
<td>0.000</td>
<td>0.000</td>
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<td></td>
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<td><strong>71.200</strong></td>
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<td><strong>16.020</strong></td>
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<td><strong>0.000</strong></td>
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<tr>
<td>3</td>
<td>L.L</td>
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<td>a</td>
<td>Left span</td>
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</tr>
<tr>
<td></td>
<td>70R Wheeled</td>
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<td>10.737</td>
<td>1.100</td>
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</tr>
<tr>
<td></td>
<td>Class A 2 Lane</td>
<td>49.480</td>
<td>0.225</td>
<td>11.133</td>
<td>0.625</td>
<td>30.925</td>
</tr>
</tbody>
</table>

**Summary**

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td>118.920</td>
<td>26.757</td>
<td>52.492</td>
<td></td>
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</tr>
</tbody>
</table>

### II Substructure

<table>
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<tr>
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<th>V</th>
<th>p</th>
<th>P</th>
<th>e&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>e&lt;sub&gt;T&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
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<tbody>
<tr>
<td>1</td>
<td>Pedestal</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>0.010</td>
<td>2.400</td>
<td>0.023</td>
<td>0.225</td>
<td>0.005</td>
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<tr>
<td>2</td>
<td>Abut cap R</td>
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<td>2.400</td>
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<tr>
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<td>Abut cap Tr</td>
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<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3</td>
<td>Abut shaft up to H.F.L</td>
<td>0.000</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>below H.F.L</td>
<td>18.201</td>
<td>1.400</td>
<td>25.481</td>
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<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4</td>
<td>Footing</td>
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<td>6</td>
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<td>7</td>
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<td>83.277</td>
<td>-2.213</td>
<td>-184.29</td>
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</tr>
</tbody>
</table>

**Summary**

|   |   |   |   |
|---|---|---|
|   | 123.515 | 46.096 | 0.000 |

### b) Horizontal Forces and Moments with respect to Base

**1 Longitudinal Forces at bearing level**

<table>
<thead>
<tr>
<th>H&lt;sub&gt;L&lt;/sub&gt;</th>
<th>H&lt;sub&gt;T&lt;/sub&gt;</th>
<th>e&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>e&lt;sub&gt;T&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.938</td>
<td>0.000</td>
<td>9.639</td>
<td>66.879</td>
<td>9.614</td>
<td>0.000</td>
</tr>
</tbody>
</table>

**Summary**

|   |   |   |   |
|---|---|---|
|   | 6.938 | 0.000 | 66.879 | 0.000 |

**P** = 242.435 t

**M<sub>L</sub>** = 139.732 t-m

**M<sub>T</sub>** = 52.492 t-m
Check for Maximum Allowable Base Pressure:

\[ P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} \]
\[ = \frac{242.435}{30.813} + \frac{139.732}{37.232} + \frac{52.492}{21.826} = 14.026 \text{ t/m}^2 \]

\[ P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} \]
\[ = \frac{242.435}{30.813} - \frac{139.732}{37.232} - \frac{52.492}{21.826} = 1.710 \text{ t/m}^2 \]

H.F.L Condition with L.L

(a) Vertical load and their moments about Abutment Shaft bottom

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>V</th>
<th>p</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>1 Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>0.010</td>
<td>2.400</td>
<td>0.023</td>
<td>0.225</td>
<td>0.005</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>2 Abut cap R</td>
<td>1.789</td>
<td>2.400</td>
<td>4.293</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Abut cap Tr</td>
<td>5.370</td>
<td>2.400</td>
<td>12.888</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>3 Abut shaft up to H.F.L</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>below H.F.L</td>
<td>18.201</td>
<td>1.400</td>
<td>25.481</td>
<td>0.000</td>
<td>0.000</td>
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<td>42.685</td>
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</table>

(b) Horizontal Forces and Moments with respect to Abutment Shaft bottom

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
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<tr>
<td>6.938</td>
<td>0.000</td>
<td>8.669</td>
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<td>0.000</td>
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<tr>
<td>Earth pressure</td>
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<td></td>
<td></td>
<td>179.197</td>
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<table>
<thead>
<tr>
<th></th>
<th></th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.938</td>
<td>0.000</td>
<td>239.346</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Summary for design of Abutment shaft

\[ P = 161.605 \text{ t} \]
\[ M_L = 266.108 \text{ t-m} \]
\[ M_T = 52.492 \text{ t-m} \]
Summary of Loads at Abutment Shaft bottom:

<table>
<thead>
<tr>
<th></th>
<th>DRY condition</th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
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<tbody>
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<td>1</td>
<td>With L.L</td>
<td>179.81</td>
<td>367.35</td>
<td>52.49</td>
</tr>
<tr>
<td>2</td>
<td>H.F.L</td>
<td>161.61</td>
<td>266.11</td>
<td>52.49</td>
</tr>
</tbody>
</table>
Dimensions of Substructure & Foundation

1 Pedestal
Length = 0.7
Width = 0.55 Volume = 0.00963
Height = 0.025

2 Pier cap
a Top uniform portion
Width = 1.00
Depth = 0.225 Volume = 1.78875 m³
Length = 7.95

b Top uniform portion
Width = 1.00
Depth = 0.075 Volume = 0.59625 m³
Length = 7.95

c Bottom trapezoidal portion
Width = 1.00
Depth = 1.2 Volume = 5.37 m³
Length = 7.95
Area at level 215.148 m = 7.95 x 1 = 7.95 m²
Area at level 213.948 m = 1 x 1 = 1 m² = 7.75 m³

H.F.L
Pier cap = 1.78875 m³
= 0.59625 m³
= 5.37 m³

3 Abutment shaft
Area at abutment shaft bottom = 3.45062 m²
Area at abutment shaft top = 1.64485 m²
Average area = 2.54773 m²
Height of Abutment shaft = 7.14
Height above Ground level = 5.676 m Volume = 14.4609
Height below Ground level = 1.468 m Volume = 3.74007
= 18.201 m³

Height above H.F.L = 0.000 m Volume = 0
Height below H.F.L = 7.14 m Volume = 18.201
= 18.201 m³

4 Pedestal at footing top
Width = 1.6
Length = 2.50 Volume = 0 m³
Height = 0
5 Footing

at foundation level

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>7.250  m</td>
</tr>
<tr>
<td>Length</td>
<td>4.25   m</td>
</tr>
<tr>
<td>Thickness at Root</td>
<td>0.97   m</td>
</tr>
<tr>
<td>Thickness at Tip</td>
<td>0.15   m</td>
</tr>
</tbody>
</table>

= 20.043 $m^3$

Volume of Overburden earth below ground level

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total volume</td>
<td>7.250 x 4.250 x 2.438 = 75.1209 $m^3$</td>
</tr>
<tr>
<td>Net volume below Ground level</td>
<td>51.3378 $m^3$</td>
</tr>
</tbody>
</table>

Wt of earth on Heel side above ground level = 83.2769 t

Sectional Properties of Footing

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>7.250 x 4.250 = 30.813 $m^2$</td>
</tr>
<tr>
<td>Z_L</td>
<td>4.250 x 8.76042 = 37.232 $m^3$</td>
</tr>
<tr>
<td>Z_T</td>
<td>7.250 x 3.01042 = 21.826 $m^3$</td>
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</tbody>
</table>
DESIGN OF ABUTMENT CAP:

For Outer bearing

\[
\begin{align*}
a &= 2.125 \\ D &= 1.50 \\
\text{Available effective depth} \\
\text{using } 25 \text{ mm dia of bars} \\
\text{d} &= 1.425 \text{ m} \\
a/d &= \frac{2.125}{1.425} = 1.4912 > 1 \\
\end{align*}
\]

= Pier cap designed as Cantilever
Bearings A1 & A4 are effective.

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<thead>
<tr>
<th></th>
<th>A1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>14.5</td>
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<tr>
<td>SIDL</td>
<td>3.3</td>
</tr>
<tr>
<td>LL</td>
<td>7.3807</td>
</tr>
<tr>
<td>70RW</td>
<td>11.93</td>
</tr>
</tbody>
</table>

due to Transverse moment

\[ A1 = 47.72 \times 1.16 = 55.355 \]

\[ A1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t} \]

**Summary of loads from superstructure:**

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>A1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
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</tr>
<tr>
<td>SIDL</td>
<td>3.3</td>
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<tr>
<td>LL</td>
<td>7.3807</td>
</tr>
<tr>
<td>70RW</td>
<td>11.93</td>
</tr>
</tbody>
</table>

\[ 37.111 \]

\[ \begin{align*} 
0.3 & \quad \begin{cases} 
0.88 \\ 0.65 \\ 1.500 
\end{cases} \\
& \quad \begin{cases} 
3.00 \\ 2.13 
\end{cases} 
\end{align*} \]

Selfweight of piercap upto section through outerbearing:

\[ = \frac{0.3 + 1.50 \times 3.00 \times l \times 2.4}{2.00} = 6.48 \text{ t} \]

Distance of c.g from section

\[ = 1.1667 \text{ m} \]

**Calculation of Cantilevermoment at bearing section**

Moment at the face due to load from outerbearing

\[ = 37.11 \times 2.13 = 78.860223 \text{ t-m} \]

Due to selfweight of Abutment cap

\[ = 6.48 \times 1.1667 = 7.56 \text{ t-m} \]

Total moment at the face of support

\[ = 86.42 \text{ t-m} \]

Torsion at outerbearing section:

\[ = 37.111 \]

Eccentricity

\[ = 0.225 \]

Moment

\[ = 8.3499 \text{ t-m} \]
**Longitudinal Reinforcement:**

\[
M = 86.42 \text{ t-m} \\
\text{For, } M = 25 \text{ & } Fe = 415
\]

\[
\sigma_{cbc} = 8.33 \text{ N/mm}^2 \\
m = 10 \\
\sigma_{st} = 200 \text{ N/mm}^2 \\
K = 0.294 \\
j = 0.902 \\
R = 1.105
\]

Effective depth required

\[
= 86.42 \times 10^{-7} \\
= 884.22 < 1425 \text{ O.K}
\]

\[
A_{st} \text{ required} \\
= 86.42 \\
= 200 \times 0.902 \times 1425 \\
= 3361.8851 \text{ mm}^2
\]

Provide 6 nos 20 mm dia bars in 2 layers 3769.9 mm²

This reinforcement will provided in full length of Abutment cap.

**Check for Shear:**

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 37.111 t

Downward Load due to self wt of Abutment cap = 6.48 t

Total downward Load = 43.591 t

After shear correction, \( S.F = \frac{V - M \tan \beta}{d} \)

\[
= 43.591 - \frac{86.42 \times 0.4}{1.425} \\
= 19.332385 \text{ t}
\]

Equivalent shear IRC:21-2000,cl:304.2.3.1

\[
b = \frac{1 + 1}{2} = 1 \text{ m}
\]

\[
V_e = V + 1.6 \times \frac{T}{b} \\
= 32.692 \text{ t}
\]

Shear stress

\[
= 32.692 \times 10000 \\
= 0.2294 \text{ N/mm}^2
\]

Area of tension reinforcement

\[
\tau_e = 0.2646 \% \\
\tau_e = 0.2671 \text{ N/mm}^2
\]

Provide minimum shear
EARTH PRESSURE CALCULATION

Formation level = 216.189 m
Founding level = 205.834 m
Low Water Level = 208.272 m
Highest flood level = 214.172 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb’s theory of active earth pressure

\[ K_a = \sin^2 (\alpha + \phi) \left[ \sin \alpha \sin (\alpha - \delta) \right] \]

\[ = \sin^2 \left( \alpha \sin \left( \frac{\phi + \delta}{\sin (\alpha - \delta) \sin (\alpha + \delta)} \right) \right) \]

Here
\- Angle of internal friction, \( \phi = 30^\circ \)
\- Angle of friction between soil and concrete, \( \delta = 20^\circ \)
\- Surcharge angle, \( i = 0^\circ \)
\- Angle of wall face with horizontal, \( \alpha = 90^\circ \)
\- Bulk density of earth, \( \gamma = 1.8 \text{ t/m}^3 \)
\- Submerged density of earth, \( \gamma_{\text{sub}} = 1.0 \text{ t/m}^3 \)
\- Width of abutment, \( a = 2.50 \text{ m} \)

\[ K_a = \frac{\sin^2 2.09}{2.523} \]

\[ = 0.750 = 0.2973 \]

CALCULATION OF EARTH PRESSURE IN DRY CONDITION

\[ P_1 = 0.2973 \times 1.8 \times 10.36 = 5.5 \text{ t/m}^2 \]
\[ P_2 = 0.2973 \times 1.0 \times 0.00 = 0 \text{ t/m}^2 \]
\[ F_1 = 0.5 \times 5.542 \times 10.36 \times 2.5 \times \cos 0.3 \]
\[ = 67.404 \text{ t} \]
\[ = 67.404 \times 0.42 \times 10.36 \]
\[ = 293 \text{ t-m} \]

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

\[ P_4 = 0.2973 \times 1.8 \times 2.02 = 1.1 \text{ t/m}^2 \]
\[ P_5 = 0.2973 \times 1.0 \times 8.34 = 2.5 \text{ t/m}^2 \]
\[ F_4 = 0.5 \times 1.079 \times 2.02 \times 2.5 \times \cos 0.3 \]
\[ = 2.5574 \text{ t} \]
\[ F_5 = 0.5 \times 2.479 \times 8.34 \times 2.5 \times \cos 0.3 \]
\[ = 24.28 \text{ t} \]
\[ F_6 = 1.08 \times 8.34 \times 2.5 \times \cos 0.3 \]
\[ = 21.14 \text{ t} \]

Total force, \( F = 2.5574 + 24.28 + 21.14 \]
\[ = 48 \text{ t} \]

\[ M = 2.5574 \times \left( \frac{0.42 \times 2.02 + 8.34}{2} \right) \]
\[ + 24.28 \times 0.33 \times 8.34 + 21.1 \times 8.34 \]
\[ = 178 \text{ t-m} \]
CALCULATION OF LIVE LOAD SURCHARGE

Dry

\[
P_6 = 0.2973 \times 1.2 \times 1.8 = 0.64 \text{ t/m}^2
\]

\[
F_7 = 0.64 \times 10.36 \times 3 \times \cos 0.3
\]

\[
= 16 \text{ t}
\]

\[
M = 16 \times \frac{9.75}{2} = 76 \text{ t-m}
\]

H.F.L

\[
P_6 = 0.2973 \times 1.2 \times 1.0 = 0.36 \text{ t/m}^2
\]

\[
F_7 = 0.6422 \times 2.02 \times 3 \times \cos 0.3 = 6.988548 \text{ t}
\]

\[
= 0.3568 \times 8.34 \times 3 \times \cos 0.3 = 6.988548 \text{ t}
\]

\[
= 10.03156
\]

\[
M = 3.043 + 6.988548147 \times 5.1775 = 51.9384 \text{ tm}
\]
EARTH PRESSURE CALCULATION

Formation level = 216.189 m
Abutment shaft bottom level = 206.804 m
Low Water Level = 208.272 m
Highest flood level = 214.172 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb's theory of active earth pressure

\[ K_a = \frac{\sin^2 (\alpha + \phi)}{\sin^2 \alpha \sin (\alpha - \delta) \left[ 1 + \frac{\sin (\phi + \delta) \sin (\phi - i)}{\sin (\alpha - \delta) \sin (\alpha + i)} \right]^2} \]

Here
- Angle of internal friction, \( \phi = 30^\circ \)
- Angle of friction between soil and concrete, \( \delta = 20^\circ \)
- Surcharge angle, \( i = 0^\circ \)
- Angle of wall face with horizontal, \( \alpha = 90^\circ \)
- Bulk density of earth, \( \gamma = 1.8 \text{ t/m}^3 \)
- Submerged density of earth, \( \gamma_{\text{sub}} = 1.0 \text{ t/m}^3 \)
- Width of abutment, \( = 2.50 \text{ m} \)

\[ K_a = \frac{\sin^2 2.09}{\sin^2 1.57 \times \sin 1.22 \times \sqrt{1 + \frac{\sin 0.87 \times \sin 0.52}{\sin 1.22 \times \sin 1.57}}} \]

\[ = \frac{0.750}{2.523} = 0.2973 \]

CALCULATION OF EARTH PRESSURE IN DRY CONDITION

\[ P_1 = 0.2973 \times 1.8 \times 9.38 = 5.02 \text{ t/m}^2 \]
\[ P_2 = 0.2973 \times 1.0 \times 0.00 = 0 \text{ t/m}^2 \]
\[ F_1 = 0.5 \times 5.023 \times 9.38 \times 2.5 \times \cos 0.3 \]
\[ = 55.367 \text{ t} \text{ say } 56 \text{ t} \]
\[ M = 55.367 \times 0.42 \times 9.38 \]
\[ = 218 \text{ t-m} \]

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

\[ P_4 = 0.2973 \times 1.8 \times 2.02 = 1.08 \text{ t/m}^2 \]
\[ P_5 = 0.2973 \times 1.0 \times 7.37 = 2.19 \text{ t/m}^2 \]
\[ F_4 = 0.5 \times 1.079 \times 2.02 \times 2.5 \times \cos 0.3 \]
\[ = 2.5574 \text{ t} \]
\[ F_5 = 0.5 \times 2.191 \times 7.37 \times 3 \times \cos 0.3 \]
\[ = 18.96 \text{ t} \]
\[ F_6 = 1.08 \times 7.37 \times 2.5 \times \cos 0.3 \]
\[ = 18.68 \text{ t} \]
Total force, \( F = 2.5574 + 18.96 + 18.68 \)
\[ = 40 \text{ t} \]
\[ M = 2.5574 \times (0.42 \times 2.02 + 7.37) + 18.96 \times 0.33 \times 7.37 + 18.7 \times 7.37 \]
\[ = 136 \text{ t-m} \]
**CALCULATION OF LIVE LOAD SURCHARGE**

**Dry**

\[
P_6 = 0.2973 \times 1.2 \times 1.8 = 0.64 \text{ t/m}^2
\]

\[
F_7 = 0.64 \times 9.38 \times 3 \times \cos 0.3
\]

\[
= 14 \text{ t}
\]

\[
M = 14 \times \frac{8.78}{2} = 62 \text{ t-m}
\]

**H.F.L**

\[
P_6 = 0.2973 \times 1.2 \times 1.0 = 0.36 \text{ t/m}^2
\]

\[
F_7 = 0.6422 \times 2.02 \times 3 \times \cos 0.3 = 3.043011 \text{ t}
\]

\[
= 0.3568 \times 7.37 \times 3 \times \cos 0.3 = 6.175536 \text{ t}
\]

\[
= 9.218547
\]

\[
M = 3.043 + 6.17536429 \times 4.6925 = 43.25803 \text{ tm}
\]
INPUT FILE: 9.2 M SPAN DL.STD
1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 09-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
8. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 3 0.4 0.0 0.0; 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
9. 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0; 18 22 18 0.0 0.0; 23 18 0.0 0.0; 24 18.4 0.0 0.0
10. REPEAT ALL 1 0.0 0.0 1.0
11. REPEAT 3 0.0 0.0 2.0
12. REPEAT 1 0.0 0.0 1.0
13. MEMBER INCIDENCES
14. 1 1 2 23 1
15. REPEAT 5 23 24
16. 139 1 25 143 1
17. REPEAT 23 5 1
18. CONSTANTS
19. E 2.7386E+006
20. POISSON 0.15
21. DENSITY 2.4 MEMB 1 TO 138
22. DENSITY 0.0001 MEMB 139 TO 258
23. MEMBER PROPERTY INDIAN
24. 2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
25. 26. 25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
27. 28. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
29. 30. 123 24 35 46 47 58 69 70 81 92 93 104 115 -
31. 32. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
33. 34. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
35. SUPPORTS
36. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
37. 38. 38 36 TO 104 127 START MZ
39. 40. 12 35 58 81 104 127 END MZ
31. MEMBER RELEASE
32. 37. 1 24 47 93 116 END MZ
33. 38. 23 46 92 115 138 START MZ
34. 39. 12 35 58 81 104 127 START MZ
35. 40. 12 35 58 81 104 127 END MZ
41. LOAD 1 SELFWEIGHT
42. SELF Y -1
43. PERFORM ANALYSIS
44. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70 94 118
<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
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</table>

************** END OF LATEST ANALYSIS RESULT **************

45. FINISH
INPUT FILE: 9.2 M SPAN SIDL.STD
1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 09-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
8. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 3 0.4 0.0 0.0; 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
9. 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0 22180.0 0.0 0.0
10. 24 18.4 0.0 0.0
11. REPEAT ALL 1 0.0 0.0 1.0
12. REPEAT 3 0.0 0.0 2.0
13. REPEAT 1 0.0 0.0 1.0
14. MEMBER INCIDENCES
15. 1 1 23 1 1
16. REPEAT 5 23 24
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29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
31. SUPPORTS
32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
34. 35 59 83 107 PINNED
35. 46 70 94 118 PINNED
36. MEMBER RELEASE
37. 1 24 47 93 116 END MZ
38. 23 46 92 115 138 START MZ
39. 12 35 58 81 104 127 START MZ
40. 12 35 58 81 104 127 END MZ
41. LOAD 1 SIDL
42. *** CRASH BARRIER
43. MEMBER LOAD
44. 25 TO 45 UNI GY -1.0
45. 94 TO 114 UNI GY -1.0
46. 25 TO 45 UMOM GX -1.0
47. 94 TO 114 UMOM GX 1.0
48. LOAD 2 WEARINGCOAT
49. MEMBER LOAD
50. 25 TO 114 UNI GY -0.2464
51. PERFORM ANALYSIS
### SUPPORT REACTION LIST

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
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*************** END OF LATEST ANALYSIS RESULT ***************

53. FINISH
INPUT FILE: 7ORW.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
8. MEMBER INCIDENCES
9. 1 1 2 3
10. MEMBER PROPERTY CANADIAN
11. 1 TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
20. LOAD GENERATION 175
21. TYPE 1 -13.4 0. 0. XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 30
24. PRINT SUPPORT REACTION
SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE
-----------------

JOINT  LOAD  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM Z

2  30  0.00  47.72  0.00  0.00  0.00  0.00
3  30  0.00  20.28  0.00  0.00  0.00  0.00

************** END OF LATEST ANALYSIS RESULT **************

25. FINISH
INPUT FILE: CLASS A.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
   7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
8. MEMBER INCIDENCES
   9. 1 1 2 3
10. MEMBER PROPERTY CANADIAN
11. 1 TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
   17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
20. LOAD GENERATION 100
21. TYPE 1 -18.8 0. 0. XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 74
24. PRINT SUPPORT REACTION
SUPPORT REACTION
-- Houston --
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM Z
2 74 0.00 24.74 0.00 0.00 0.00 0.00
3 74 0.00 11.66 0.00 0.00 0.00 0.00

*************** END OF LATEST ANALYSIS RESULT ***************

25. FINISH
STABILITY CHECK & DESIGN OF SUBSTRUCTURE (PIER)
**Design Data:**
For design purposes, following parameters have been considered.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of concrete</td>
<td>M - 20</td>
</tr>
<tr>
<td>Pier Cap</td>
<td>M - 25</td>
</tr>
<tr>
<td>Grade of reinforcement steel</td>
<td>Fe - 415</td>
</tr>
<tr>
<td>Centre to Centre distance between Expansion joints</td>
<td>9.200 m</td>
</tr>
<tr>
<td>Centre to Centre distance of Bearing</td>
<td>8.800 m</td>
</tr>
<tr>
<td>Depth of superstructure</td>
<td>660 mm</td>
</tr>
<tr>
<td>Thickness of wearing coat</td>
<td>56.00 mm</td>
</tr>
<tr>
<td>Formation level along C of carriage way</td>
<td>216.189 m</td>
</tr>
<tr>
<td>Soffit level</td>
<td>215.423</td>
</tr>
<tr>
<td>Pedestal top level</td>
<td>215.473</td>
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<tr>
<td>Height of bearing and Pedestal</td>
<td>0.050 m</td>
</tr>
<tr>
<td>L.W.L./Bed level</td>
<td>208.272 m</td>
</tr>
<tr>
<td>H.F.L</td>
<td>214.172 m</td>
</tr>
<tr>
<td>M.S.L</td>
<td>205.452 m</td>
</tr>
<tr>
<td>Founding Level</td>
<td>205.834 m</td>
</tr>
<tr>
<td>Pier cap top level</td>
<td>215.423 m</td>
</tr>
<tr>
<td>Live Load</td>
<td>(a) Class A two Lane</td>
</tr>
<tr>
<td></td>
<td>(b) Class 70R wheeled</td>
</tr>
<tr>
<td>Bearing : Neoprene but during raising Tar paper bearing may be kept.</td>
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</tr>
<tr>
<td>Seismic zone</td>
<td>II</td>
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</tbody>
</table>

The following codes are used for the design of substructure:

1. IRC : 6 - 2000
2. IRC : 21 - 2000
3. IRC : 78 - 2000
Longitudinal translation due to creep, shrinkage & temperature = 0.0005
Horizontal movement = 0.0005 x 4.60 x 1000 = 2.300 mm

**Longitudinal Force:**

Size of bearing = 400 x 250 x 50 mm
Strain in bearing = \[
\frac{2.300}{50} = 0.046
\]
Shear modulus = 1.0 Mpa
Shear force per Bearing = 0.046 x 1.0 x 400 x 250 = 4600 N = 0.469 t

Total shear force for 4 bearings (with 5% increase)
\[
= 0.469 \times 4 \times 1.05 = 1.969 \text{ t}
\]

Refer IRC: 6 clause 214.5.1.5:

\[10\%\text{ increase for variation in movement of span}\]

Total shear force = 1.1 x 1.969 = 2.166 t

As per clause 214.2 of IRC:6, horizontal braking force \(F_h\), for each span is:

For Class A 2 lane:
\[
F_h = \left\{ \begin{array}{l}
0.2 \times 59.2 \\
\end{array} \right. = 11.836 \text{ t}
\]

For class 70R wheeled:
\[
F_h = \left\{ \begin{array}{l}
0.2 \times 66.1 \\
\end{array} \right. = 13.210 \text{ t}
\]

**Summary of Longitudinal Forces:**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
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</thead>
<tbody>
<tr>
<td>Class A 2 lane</td>
<td>8.08</td>
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<tr>
<td>70R</td>
<td>8.77</td>
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<tr>
<td>Span dislodged condition</td>
<td>2.17</td>
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</table>
### Dry Condition with L.L.

**a) Vertical load and their moments about C/L of Foundation base.**

<table>
<thead>
<tr>
<th>I</th>
<th>1 D.L. Reaction</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<tbody>
<tr>
<td>a</td>
<td>Left span</td>
<td>58.00</td>
<td>-0.4</td>
<td>-23.2</td>
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<tr>
<td>b</td>
<td>Right span</td>
<td>58</td>
<td>0.4</td>
<td>23.2</td>
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<table>
<thead>
<tr>
<th>2 S.I.D.L</th>
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<tbody>
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<td>b</td>
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**4 L.L. Max Reaction case**

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<td>36</td>
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<td>Class A 3 Lane</td>
<td>33.6</td>
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<table>
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</thead>
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<td>70R Wheeled + Class A</td>
<td>30.05</td>
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<td>Class A 3 Lane</td>
<td>25.58</td>
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**4 L.L. Max Longitudinal Moment case**

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<td>Class A 3 Lane</td>
<td>6.88</td>
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<tbody>
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<td>70R Wheeled + Class A</td>
<td>47.15</td>
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<tr>
<td>Class A 3 Lane</td>
<td>47.74</td>
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**Max Reaction case**

| 209.41 | 1.42 | 72.655 |

**Max Longitudinal Moment**

| 190.51 | 18.86 | 51.865 |

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<th>Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
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<td>0</td>
<td>0.4</td>
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<td>69.653</td>
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**149.08** | **0** | **0**
b) Horizontal Forces and Moments with respect to Base

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<th>$H_T$</th>
<th>$e_L$</th>
<th>$M_L$</th>
<th>$e_T$</th>
<th>$M_T$</th>
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<td>9.639</td>
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Summary

Max Reaction

Max Longitudinal

$$ P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} $$

$$ = \frac{358.49}{20.3} + \frac{85.967}{19.623} + \frac{72.655}{11.842} = 28.1759 \text{ t/m}^2 $$

$$ P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} $$

$$ = \frac{358.49}{20.3} - \frac{85.967}{19.623} - \frac{72.655}{11.842} = 7.1431 \text{ t/m}^2 $$

Max Long

$$ P_{max} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} $$

$$ = \frac{339.59}{20.3} + \frac{103.41}{19.623} + \frac{51.865}{11.842} = 26.3779 \text{ t/m}^2 $$

$$ P_{min} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} $$

$$ = \frac{339.59}{20.3} - \frac{103.41}{19.623} - \frac{51.865}{11.842} = 7.078993 \text{ t/m}^2 $$
Design of Footing in Transverse direction

Gross Pressure Diagram

Net Pressure diagram

Bending moment at the face of pier b-b = 167.27 t-m
Footing is checked for shear at distance of d from face pier
Effective depth d of footing = 0.8125 m
Total depth of section at distance d from pier = 0.6553 m
Effective depth at distance d from face of pier = 0.5643 m

Bending moment at a distance of d from face of pier = 36.54 t-m
Shear force at a distance d from face of pier = 91.623 t
Net shear force at a distance d from face of pier = 72.118 t (after correction)

Design for Flexure:

\[ M = 20 \quad Fe = 415 \]

Permissible compressive stress \( \sigma_{bc} = 6.67 \text{ N/mm}^2 \)
Permissible tensile stress \( \sigma_{st} = 200 \text{ N/mm}^2 \)
\( m = 10 \)
\( k = 0.25 \)
\( j = 0.92 \)
\( Q = 0.7639 \)
width \( b = 3.50 \)

Effective depth required \( d = 790.97 \text{ mm} \)
Effective depth provided \( = 812.5 \text{ mm} \)
\( A_{st} \) Required \( = 3208.4 \text{ mm}^2 \)

Provide 7 nos 25 dia bars /m width

Check for shear

Shear stress at effective depth from pier \( = 36.517 \text{ t/m}^2 \) 0.358005
\% of steel provided \( = 0.3565 \)
Permissible shear stress \( = 25.5 \text{ t/m}^2 \) 0.25

Area of shear reinforcement IRC:21-2000, cl:304.7.1.4

\[
A_{sw} = \frac{V - \left( \tau_c \times b \times d \right) \times s}{\sigma_s \times d} = 403.08 \text{ mm}^2
\]

Minimum Shear Reinforcement :

\[
\frac{A_{sw}}{b \times s} = \frac{0.4}{0.87 \times f_y}
\]

\[
\frac{A_{sw}}{s} = \frac{0.4 \times b}{0.87 \times f_y} = 3.8776 \text{ mm}^2/\text{mm}
\]

Provide 2 Legged 12 @ 200 = 1131

<table>
<thead>
<tr>
<th>Dry Condition with L.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a) Vertical load and their moments about Pier Shaft bottom</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>( V )</th>
<th>( \rho )</th>
<th>( P )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
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<tr>
<td>1 Pedestal</td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>a Left span</td>
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<td>0</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b Right span</td>
<td>0</td>
<td>2.4</td>
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<td>0.4</td>
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<td>2 Pier cap R</td>
<td>2.385</td>
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<td>5.724</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.37</td>
<td>2.4</td>
<td>12.888</td>
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<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>3 Pier shaft</td>
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<td>up to G.L</td>
<td>10.76912</td>
<td>2.4</td>
<td>25.846</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>below G.L</td>
<td>2.8356842</td>
<td>2.4</td>
<td>6.8056</td>
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<tr>
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<td>51.264</td>
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</table>
### b) Horizontal Forces and Moments with respect to Pier Shaft bottom

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>$H_L$</th>
<th>$H_T$</th>
<th>$e_L$</th>
<th>$M_{L}$</th>
<th>$e_T$</th>
<th>$M_{T}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.77</td>
<td>0</td>
<td>8.689</td>
<td>76.214</td>
<td>8.639</td>
<td>0</td>
</tr>
<tr>
<td>8.77</td>
<td>0</td>
<td>76.214</td>
<td>0</td>
<td>8.639</td>
<td>0</td>
</tr>
</tbody>
</table>

**Summary for design of Pier**

Max Reaction case

Max Longitudinal moment

$P = 260.67$ t

$M_L = 77.634$ t-m

$M_T = 72.655$ t-m

### Dry Condition One. Span dislodged

#### a) Vertical load and their moments about C/L of Foundation base.

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>$H_L$</th>
<th>$H_T$</th>
<th>$e_L$</th>
<th>$M_{L}$</th>
<th>$e_T$</th>
<th>$M_{T}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.17</td>
<td>0</td>
<td>9.639</td>
<td>20.882</td>
<td>9.589</td>
<td>0</td>
</tr>
<tr>
<td>2.17</td>
<td>0</td>
<td>20.882</td>
<td>0</td>
<td>9.589</td>
<td>0</td>
</tr>
</tbody>
</table>

**Summary**

$P = 292.44$ t

$M_L = 20.882$ t-m

$M_T = 0$ t-m

**Check for Maximum Allowable Base Pressure:**

$$P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}$$

$$= \frac{292.43789}{20.3} + \frac{20.882}{19.623} + \frac{0}{11.842} = 15.46993 \text{ t/m}^2$$

$$P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}$$

$$= \frac{292.43789}{20.3} - \frac{20.882}{19.623} - \frac{0}{11.842} = 13.34169 \text{ t/m}^2$$

### Dry Condition One. Span dislodged

#### a) Vertical load and their moments about Abutment Shaft bottom.

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>$H_L$</th>
<th>$H_T$</th>
<th>$e_L$</th>
<th>$M_{L}$</th>
<th>$e_T$</th>
<th>$M_{T}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.17</td>
<td>0</td>
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<td>18.824</td>
<td>8.639</td>
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</tr>
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<td>18.824</td>
<td>0</td>
<td>8.639</td>
<td>0</td>
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</tbody>
</table>

**Summary for design of Pier**

$P = 194.62$ t

$M_L = 18.824$ t-m

$M_T = 0$ t-m
**H.F.L Condition with L.L**

*a) Vertical load and their moments about C/L of Foundation base.*

<table>
<thead>
<tr>
<th>1000 D.L Reaction</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>a Left span</td>
<td>58.00</td>
<td>-0.400</td>
<td>-23.200</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>58.00</td>
<td>0.400</td>
<td>23.200</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2000 S.I.D.I.</td>
<td></td>
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</tr>
<tr>
<td>a Left span</td>
<td>13.680</td>
<td>-0.400</td>
<td>-5.472</td>
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</tr>
<tr>
<td>b Right span</td>
<td>13.680</td>
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<td>5.472</td>
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<td></td>
<td>143.360</td>
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<tr>
<td>3000 L.L</td>
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</tr>
<tr>
<td>a Left span</td>
<td>36.000</td>
<td>-0.400</td>
<td>-14.400</td>
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<tr>
<td>Class A 2 Lane</td>
<td>33.600</td>
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<td>-13.440</td>
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<tr>
<td>b Right span</td>
<td>30.050</td>
<td>0.400</td>
<td>12.020</td>
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<td>33.055</td>
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<tr>
<td>Class A 2 Lane</td>
<td>25.580</td>
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<td>10.232</td>
<td>0.625</td>
<td>15.988</td>
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<tr>
<td>4000 L.L</td>
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<tr>
<td>a Left span</td>
<td>47.150</td>
<td>0.400</td>
<td>18.860</td>
<td>1.100</td>
<td>51.865</td>
</tr>
<tr>
<td>Class A 2 Lane</td>
<td>47.740</td>
<td>0.400</td>
<td>19.096</td>
<td>0.625</td>
<td>29.838</td>
</tr>
</tbody>
</table>

**Max Reaction case:** 209.410 1.420 72.655  
**Max Long. moment case:** 190.510 18.860 51.865

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>V</th>
<th>p</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000 Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a Left span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>-0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2000 Pier cap R</td>
<td>2.385</td>
<td>2.400</td>
<td>5.724</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.370</td>
<td>2.400</td>
<td>12.888</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3000 Pier shaft</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>up to H.F.L</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>13.605</td>
<td>2.400</td>
<td>19.047</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4000 Footing</td>
<td>11.734</td>
<td>1.400</td>
<td>16.427</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>5000 Earth above</td>
<td>34.827</td>
<td>1.000</td>
<td>34.827</td>
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<td>0.000</td>
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<tr>
<td>footing</td>
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<tr>
<td>6000 Water current</td>
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<td>16.274</td>
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<td>6.612</td>
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</tbody>
</table>

**b) Horizontal Forces and Moments with respect to Base**

<table>
<thead>
<tr>
<th>1000 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_L</td>
</tr>
<tr>
<td>-----</td>
</tr>
<tr>
<td>8.771</td>
</tr>
</tbody>
</table>

**Summary**

Max Reaction case:

- P = 298.323 t
- M_L = 102.241 t-m
- M_T = 79.267 t-m

Max Long. moment case:

- P = 279.423
- M_L = 119.681
- M_T = 58.477
Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{298.323}{20.300} + \frac{102.241}{19.623} + \frac{79.267}{11.842} = 26.600 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{298.323}{20.300} - \frac{102.241}{19.623} - \frac{79.267}{11.842} = 2.792 \text{ t/m}^2
\]

Max Long

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{279.423}{20.300} + \frac{119.681}{19.623} + \frac{58.477}{11.842} = 24.802 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{279.423}{20.300} - \frac{119.681}{19.623} - \frac{58.477}{11.842} = 2.727 \text{ t/m}^2
\]

H.F.L Condition with L.L

\(a\) Vertical load and their moments about Pier Shaft bottom

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<tr>
<td>1.000 Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>a</td>
<td>Left span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>-0.400</td>
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<td>0.000</td>
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<tr>
<td>b</td>
<td>Right span</td>
<td>0.000</td>
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<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2.000 Pier cap R</td>
<td>2.385</td>
<td>2.400</td>
<td>5.724</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.370</td>
<td>2.400</td>
<td>12.888</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
<td>3.000 Pier shaft up to H.F.L</td>
<td>0.000</td>
<td>1.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>13.605</td>
<td>1.400</td>
<td>19.047</td>
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<td>0.000</td>
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<td>4.000 Water current</td>
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<td></td>
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<td>6.438</td>
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</table>

37.659 6.438 16.236

\(b\) Horizontal Forces and Moments with respect to Pier Shaft bottom

<table>
<thead>
<tr>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
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<tbody>
<tr>
<td>8.771</td>
<td>0.000</td>
<td>8.689</td>
<td>76.214</td>
<td>8.639</td>
<td>0.000</td>
</tr>
<tr>
<td>8.771</td>
<td>0.000</td>
<td></td>
<td>76.214</td>
<td>0.000</td>
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</tbody>
</table>
Gross Pressure Diagram

<table>
<thead>
<tr>
<th>Summary for design of Pier</th>
<th>Max Reaction case</th>
<th>Max Long.moment case</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P )</td>
<td>247.069 t</td>
<td>228.169 t</td>
</tr>
<tr>
<td>( M_L )</td>
<td>84.072 t-m</td>
<td>101.512 t-m</td>
</tr>
<tr>
<td>Net ( M_T )</td>
<td>88.891 t-m</td>
<td>68.101 t-m</td>
</tr>
</tbody>
</table>

H.F.L Condition One: Span dislodged

(a) Vertical load and their moments about C/L of Foundation base.

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>( V )</th>
<th>( \rho )</th>
<th>( P )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
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<tbody>
<tr>
<td>1.000</td>
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<td>0.000</td>
<td>-0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>2.385</td>
<td>2.400</td>
<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>5.370</td>
<td>2.400</td>
<td>12.888</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2.000</td>
<td>Pier cap R</td>
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<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Pier cap Tr</td>
<td>13.605</td>
<td>1.400</td>
<td>19.047</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<td>1.400</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>up to H.F.L</td>
<td></td>
<td>34.827</td>
<td>1.000</td>
<td>34.827</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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</tr>
<tr>
<td>below H.F.L</td>
<td></td>
<td>0.000</td>
<td>0.000</td>
<td>16.274</td>
<td>0.000</td>
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</tr>
<tr>
<td>4.000</td>
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<td>0.000</td>
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<td>0.000</td>
<td>0.000</td>
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</tr>
<tr>
<td>5.000</td>
<td>Earth above</td>
<td>0.000</td>
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<td>0.000</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>6.000</td>
<td>Water current</td>
<td>0.000</td>
<td>0.000</td>
<td>16.274</td>
<td>0.000</td>
<td>0.000</td>
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<td>0.000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Summary</th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( P )</td>
<td>= 232.273 t</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( M_L )</td>
<td>= 16.274 t-m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( M_T )</td>
<td>= 6.612 t-m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P + M_L + M_T}{A + Z_L + Z_T}
\]

\[
= \frac{232.273 + 16.274 + 6.612}{20.300 + 19.623 + 11.842} = 12.830 \ t/m^2
\]

\[
P_{\text{min}} = \frac{P - M_L - M_T}{A - Z_L - Z_T}
\]

\[
= \frac{232.273 - 16.274 - 6.612}{20.300 - 19.623 - 11.842} = 10.054 \ t/m^2
\]
Summary of Loads at Pier Shaft bottom:

1 DRY condition
   With L.L  260.67  77.63  72.66
   Span dislodged  194.62  18.82  0.00

2 H.F.L
   With L.L  247.07  84.07  88.89
   Span dislodged  181.02  6.44  16.24

Reinforcement provided = 13722.477

PLAN: PIER SHAFT
For Outer bearing

\[
a = 2.125 \\
D = 1.5
\]

Available effective depth using 25 mm dia of bars

\[
d = 1.5 - 0.05 - 0.025 = 1.425 \text{ m}
\]

\[
a = \frac{2.125}{1.425} = 1.4912 > 1
\]

= Pier cap designed as Cantilever
Loads on bearing

Bears A1 & B1 are effective.

<table>
<thead>
<tr>
<th></th>
<th>A1</th>
<th>B1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>LL</td>
<td>70RW</td>
<td>7.5125</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>3.38</td>
</tr>
</tbody>
</table>

due to Transverse moment

\[
\begin{align*}
A1 &= 30.05 \times 1.16 = 34.858 \\
B1 &= 36 \times 1.16 = 41.76 \\
A1 &= 34.858 \times 3.38 = 4.6477 \\
B1 &= 41.76 \times 3.38 = 5.568 \\
\end{align*}
\]

Summary of loads from superstructure:

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>Left span</th>
<th>Right span</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>A1</td>
<td></td>
</tr>
<tr>
<td>DL</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>LL</td>
<td>5.568</td>
<td>4.6477</td>
</tr>
<tr>
<td>9</td>
<td>7.5125</td>
<td></td>
</tr>
<tr>
<td><strong>32.368</strong></td>
<td><strong>29.96</strong></td>
<td></td>
</tr>
</tbody>
</table>

Selfweight of piercap up to section through outerbearing:

\[
= \frac{0.3 + 1.50}{2.00} \times 3.00 \times 1 \times 2.4 = 6.48 \ t
\]

Distance of c.g from section = 1.1667 m

Calculation of Cantilever moment at bearing section

Moment at the face due to load from outerbearing

\[
= 62.328 \times 2.125 = 132.4475 \ t-m
\]

Due to selfweight of pier cap

\[
= 6.48 \times 1.1667 = 7.56 \ t-m
\]

Total moment at the face of support

\[
= 140.01 \ t-m
\]

Torsion at outerbearing section:

\[
= 32.368 - 29.96 = 2.4078
\]

Eccentricity

\[
= 0.4
\]

Moment

\[
= 0.9631 \ t-m
\]
Longitudinal Reinforcement:

\[ M = 140.01 + \]  
For, \( M - 25 \) & \( Fe - 415 \)

\[ \sigma_{cbc} = 8.33 \text{ N/mm}^2 \]
\[ m = 10 \]
\[ \sigma_{st} = 200 \text{ N/mm}^2 \]
\[ K = 0.294 \]
\[ j = 0.902 \]
\[ R = 1.105 \]

Effective depth required
\[ = \frac{140.01 \times 10^7}{1.105 \times 1000} \]
\[ = 1125.5 < 1425 \text{ O.K} \]

\[ A_{st} \text{ required} = \frac{140.01}{200 \times 0.902 \times 1425} \]
\[ = 5446.5159 \text{ mm}^2 \]

Provide 6 nos 25 mm dia bars in 2 layers 5890.5 mm$^2$
This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root \( = 1425 \text{ mm} \)
Downward Load from Superstructure \( = 32.368 + 29.96 \text{ t} \)
Downward Load due to self wt of pier cap \( = 6.48 \text{ t} \)
Total downward Load \( = 68.808 \text{ t} \)

After shear correction, \( S.F \)
\[ = V + \frac{M \tan \beta}{d} \]
\[ = 68.808 + \frac{140.01 \times 0.4}{1.425} \]
\[ = 29.507884 \text{ t} \]

Equivalent shear IRC:21-2000, cl:304.2.3.1
\[ b = \frac{1 + 1}{2} = 1 \text{ m} \]
\[ V = 29.508 \text{ t} \]

Shear stress
\[ = \frac{29.508 \times 10000}{1000 \times 1425} = 0.2071 \text{ N/mm}^2 \]

Area of tension reinforcement
\[ \tau_c = 0.2873 \text{ N/mm}^2 \]

Provide minimum shear
For Outer bearing  
\[ a = 2.125 \]
\[ D = 1.5 \]

Available effective depth  
using 25 mm dia of bars  
\[ d = 1.5 - 0.05 - 0.025 = 1.425 \text{ m} \]

\[ \frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1 \]

= Pier cap designed as Cantilever
**Loads on bearing**

Bearings A1 & B1 are effective.

<table>
<thead>
<tr>
<th></th>
<th>A1</th>
<th>B1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>LL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70RW</td>
<td>0</td>
<td>11.93</td>
</tr>
</tbody>
</table>

due to Transverse moment

\[
A1 = 0 \times 1.16 = 0 \\
B1 = 47.72 \times 1.16 = 55.355 \\
\]

\[
A1 = \frac{0}{3.38} = 0 \text{ t} \\
B1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t} \\
\]

**Summary of loads from superstructure:**

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>Left span</th>
<th>Right span</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>A1</td>
<td></td>
</tr>
<tr>
<td>DL</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>LL</td>
<td>11.93</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td><strong>37.111</strong></td>
<td><strong>17.8</strong></td>
</tr>
</tbody>
</table>

Selfweight of piercap upto section through outerbearing:

\[
= \frac{0.3 + 1.50 \times 3.00 \times 1 \times 2.4}{2.00} = 6.48 \text{ t} \\
\text{Distance of c.g from section} = 1.1667 \text{ m}
\]

**Calculation of Cantilever moment at bearing section**

Moment at the face due to load from outerbearing

\[
= 54.911 \times 2.125 = 116.68522 \text{ t-m}
\]

Due to selfweight of pier cap

\[
= 6.48 \times 1.1667 = 7.56 \text{ t-m}
\]

Total moment at the face of support

\[
= 124.25 \text{ t-m}
\]

**Torsion at outerbearing section:**

\[
= 37.111 - 17.8 \\
= 19.311 \\
\text{Eccentricity} = 0.4 \\
\text{Moment} = 7.7243 \text{ t-m}
\]
**Longitudinal Reinforcement:**

\[ M_e = M + M_t \]
\[ M_t = T x \frac{1 + D/b}{1.7} \]
\[ = 6.8155 \text{ t-m} \]
\[ M_e = 124.25 + 6.8155 = 131.06 \text{ t-m} \]

For, \( M - 25 \) & \( Fe - 415 \)

\[ \sigma_{cbe} = \frac{8.33}{N/mm^2} \]
\[ m = 10 \]
\[ \sigma_{st} = 200 \text{ N/mm}^2 \]
\[ K = 0.294 \]
\[ j = 0.902 \]
\[ R = 1.105 \]

Effective depth required

\[ = \frac{131.06 \times 10^7}{1.105 \times 1000} \]
\[ = 1088.9 < 1425 \text{ O.K} \]

\[ A_{st \text{ required}} = \frac{131.06}{200 \times 0.902 \times 1425} \]
\[ = 5098.4736 \text{ mm}^2 \]

Provide 6 nos 25 mm dia bars in 2 layers 5890.5 mm². This reinforcement will provided in full length of pier cap.

**Check for Shear:**

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 37.111 + 17.8 = 54.911 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 61.391 t

After shear correction, \( S.F = \frac{V + M \tan \beta}{d} \)

\[ = 61.391 + \frac{124.25 \times 0.4}{1.425} \]
\[ = 26.514841 \text{ t} \]

Equivalente shear IRC:21-2000,cl:304.2.3.1

\[ b = \frac{1}{2} \]
\[ V_e = \frac{V + 1.6 \times T}{b} \]
\[ = 38.874 \text{ t} \]

Shear stress

\[ = \frac{38.874 \times 10000}{1000 \times 1425} \]
\[ = 0.2728 \text{ N/mm}^2 \]

Area of tension reinforcement

\[ = 0.4134 \% \]
\[ \tau_e = 0.2873 \text{ N/mm}^2 \]
INPUT FILE: 9.2 M SPAN CLASS A 2.STD
 1. STAAD FLOOR
 2. START JOB INFORMATION
 3. ENGINEER DATE 09-JUN-06
 4. END JOB INFORMATION
 5. INPUT WIDTH 79
 6. PAGE LENGTH 1000
 7. UNIT METER MTON
 8. JOINT COORDINATES
 9. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 30.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
 10. 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38
     0.0 0.0
 11. 24 18.4 0.0 0.0
 12. MEMBER INCIDENCES
 13. 1 1 2 23 1 1
 14. CONSTANTS
 15. E 2.7386E+006
 16. POISSON 0.15
 17. DENSITY 2.4 ALL
 18. MEMBER PROPERTY INDIAN
 19. 1 TO 23 PRIS YD 1 ZD 1
 20. SUPPORTS
 21. 3 22 FIXED BUT FX MZ
 22. 11 14 PINNED
 23. MEMBER RELEASE
 24. 1 START FX FY MZ
 25. 12 START FX FY MZ
 26. 23 END FX FY MZ
 27. DEFINE MOVING LOAD
 28. TYPE 1 LOAD 2.7 2.7 11.4 11.4 6.8 6.8 6.8 6.8
 29. DIST 1.1 3.2 1.2 4.3 3 3 3
 30. **TWO LANE OF CLASS A
 31. LOAD GENERATION 82
 32. TYPE 1 -18.8 0 0 XINC 0.5
 33. PERFORM ANALYSIS
 34. **MAX REA
 35. LOAD LIST 46
 36. PRINT SUPPORT REACTION

** SUPPORT REACTION

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM-Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>46</td>
<td>0.00</td>
<td>4.01</td>
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<td>0.00</td>
<td>0.00</td>
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<tr>
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<tr>
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<td>37. ** MAX MOM</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>38. LOAD LIST 49</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>39. PRINT SUPPORT REACTION</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>22</td>
<td>49</td>
<td>0.00</td>
<td>12.53</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
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<tr>
<td>40. FINISH</td>
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</tr>
</tbody>
</table>
INPUT FILE: 9.2 M SPAN CLASSA  70R.STD
1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 09-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
   1 0.0 0.0 0.0;2 0.02 0.0 0.0;3 0.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
   12 9.18 0.0 0.0;13 9.22 0.0 0.0;14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38
   0.0 0.0
   11. 24 18.4 0.0 0.0
12. MEMBER INCIDENCES
13. 1 1 2 23 1 1
14. CONSTANTS
15. E 2.7386E+006
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17. DENSITY 2.4 ALL
18. MEMBER PROPERTY INDIAN
19. 1 TO 23 PRIS YD 1 ZD 1
20. SUPPORTS
21. 3 22 FIXED BUT FX MZ
22. 11 14 PINNED
23. MEMBER RELEASE
24. 1 START FX FY MZ
25. 12 START FX FY MZ
26. 23 END FX FY MZ
27. DEFINE MOVING LOAD
28. TYPE 1 LOAD 8 12 12 17 17 17 17
29. DIST 3.96 1.52 2.13 1.37 3.05 1.37
30. ** CLASS 70 RW
31. LOAD GENERATION 82
32. TYPE 1 -13.4 0 0 XINC 0.5
33. PERFORM ANALYSIS
34. **MAX REA
35. LOAD LIST 29
36. PRINT SUPPORT REACTION
SUPPORT REACTION

-------------------------------------
JOINT  LOAD  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM Z

   3  29      0.00     18.95      0.00      0.00      0.00      0.00
   22 29      0.00     15.00      0.00      0.00      0.00      0.00
   11 29      0.00     30.05      0.00      0.00      0.00      0.00
   14 29      0.00     36.00      0.00      0.00      0.00      0.00
37. ** MAX MOMENT
38. LOAD LIST 19
39. PRINT SUPPORT REACTION
   3  19      0.00     32.85      0.00      0.00      0.00      0.00
   22 19      0.00      0.00      0.00      0.00      0.00      0.00
   11 19      0.00     47.15      0.00      0.00      0.00      0.00
   14 19      0.00      0.00      0.00      0.00      0.00      0.00
40. FINISH
BRIDGE AT CH:27+800
DESIGN OF SUBSTRUCTURE (PIER)
**Design Data:**
For design purposes, following parameters have been considered.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of concrete Pier &amp; Pier Cap</td>
<td>M - 35</td>
</tr>
<tr>
<td>Grade of concrete Pile cap &amp; Pile</td>
<td>M - 35</td>
</tr>
<tr>
<td>Grade of reinforcement steel</td>
<td>Fe - 415</td>
</tr>
<tr>
<td>Centre to Centre distance of A / Expansion joints</td>
<td>32.200 m</td>
</tr>
<tr>
<td>Centre to Centre distance of Bearing</td>
<td>30.000 m</td>
</tr>
<tr>
<td>Depth of superstructure</td>
<td>2100 mm</td>
</tr>
<tr>
<td>Thickness of wearing coat</td>
<td>65.00 mm</td>
</tr>
<tr>
<td>Formation level along C of carriage way</td>
<td>218.139 m</td>
</tr>
<tr>
<td>Height of bearing and Pedestal</td>
<td>0.300 m</td>
</tr>
<tr>
<td>L.W.L./G.L</td>
<td>207.905 m</td>
</tr>
<tr>
<td>H.F.L</td>
<td>214.172 m</td>
</tr>
<tr>
<td>M.S.L</td>
<td>193.355</td>
</tr>
<tr>
<td>Pier cap top level</td>
<td>215.674 m</td>
</tr>
<tr>
<td>Pile cap top level</td>
<td>207.155 m</td>
</tr>
<tr>
<td>Depth of pile cap</td>
<td>1800 mm</td>
</tr>
<tr>
<td>Pile cap bottom level</td>
<td>205.355 m</td>
</tr>
<tr>
<td>Diameter of bored cast-in-situ piles</td>
<td>1200 mm</td>
</tr>
<tr>
<td>Length of piles above rock level</td>
<td>10.000 m</td>
</tr>
<tr>
<td>Length of piles below rock level</td>
<td>2.000 m</td>
</tr>
<tr>
<td>Total length of pile</td>
<td>12.000 m</td>
</tr>
<tr>
<td>Load carrying capacity of each pile</td>
<td>275 Tons</td>
</tr>
<tr>
<td>Live Load (a) Class A 2 Lane</td>
<td></td>
</tr>
<tr>
<td>Live Load (b) Class 70R wheeled</td>
<td></td>
</tr>
<tr>
<td>Maximum surface velocity of water</td>
<td>( \sqrt{2} \times 3.000 \text{ m/sec} = 4.24 \text{ m/sec} ) Say, 4.30 m/sec</td>
</tr>
</tbody>
</table>

The following codes are used for the design of substructure:

1. IRC : 6 - 2000
2. IRC : 21 - 2000
3. IRC : 78 - 2000
Depth of SuperStructure = 12.00 m
Number of Longitudinal Girders = 4

Mid section
1 slab = 12 * 0.25 = 3 m²
2 Top Flange = \(4 \times (0.15 \times 1.25 + 0.5 \times (1.25 + 0.275)) \times 0.16\) = 1.238 m²
3 Web = \(4 \times [(2.1 - (0.15 + 0.16) - (0.2 + 0.24)) \times 0.275]\) = 1.485 m²
4 Bottom Bulb = \(4 \times [(1 \times 0.2) + (0.5 \times (1 + 0.275))] \times 0.24\) = 1.412 m²
Total = 4.135 m²
Depth of SuperStructure = 2.35
Number of Longitudinal Girders = 4

End section
1 slab = 12*0.25 = 3 m²
2 Top Flange = 4*(0.15*1.25)+(0.5*(1.25+0.6))*0.107 = 1.1459 m²
3 Web = 4*[2.1-(0.15+0.107)-(0.2+0.132)]*0.6 = 3.6264 m²
4 Bottom Bulb = 4*[1*0.2]+(0.5*(1+0.6))*0.132 = 1.2224 m²
Total = 5.99 m²
DL per running meter at Intermediate section = 10.3375 t/m
DL per running meter at End section = 14.98675 t/m
**Intermediate Cross Girder**

Thickness of Cross Girder = 0.3
Number of intermediate cross girders = 3
Cross sectional area in elevation = 12.2585 m²
Dl of intermediate cross girder = 27.58163 t

**End Cross Girder**

Thickness of end Cross Girder = 0.45
Number of intermediate cross girders = 2
Cross sectional area in elevation = 18.0277 m²
Dl of intermediate cross girder = 40.5623 t

**S.I.D.I.**

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>I Wearing Coat</td>
<td>@ 1.0725 t/m</td>
<td>1.0725 t/m</td>
</tr>
<tr>
<td>II Crash barrier</td>
<td>@ 1.00 t/m</td>
<td>2.00 t/m</td>
</tr>
<tr>
<td>III R.C.C Post &amp; Railing</td>
<td>@ 0.085 t/m</td>
<td>0.085 t/m</td>
</tr>
</tbody>
</table>

*Note: (provide one side enter 1, both sides enter 2, otherwise enter 0)*

**Diagram:**

- Footpath precast slab
- Diameter: 100 mm
### Wt due to Crashbarrier

<table>
<thead>
<tr>
<th>Component</th>
<th>Area</th>
<th>Wt</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.100</td>
<td>0.24</td>
</tr>
<tr>
<td>2</td>
<td>0.0875</td>
<td>0.21</td>
</tr>
<tr>
<td>3</td>
<td>0.075</td>
<td>0.18</td>
</tr>
<tr>
<td>4</td>
<td>0.075</td>
<td>0.18</td>
</tr>
<tr>
<td>5</td>
<td>0.002</td>
<td>0.15413</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>0.96</strong></td>
</tr>
</tbody>
</table>

**Footpath precast slab**
- 0.1525 t/m
- 0.38125 t/m

**miscellaneous utilities**
- 0.2 t/m

**Footpath kerb**
- 0.05625 t/m
- 0.14063 t/m

**sandfilling**
- 0.20519 t/m
- 0.36934 t/m

### Foot path precast slab
(provide one side enter 1, bothsides enter 2, otherwise enter 0)
- 2 = 0.7625 t/m

### Footpath Kerb
(provide one side enter 1, bothsides enter 2, otherwise enter 0)
- 2 = 0.28125 t/m

### Missillaneous utilities
(provide one side enter 1, bothsides enter 2, otherwise enter 0)
- 2 = 0.4 t/m

### Sandfilling in footpath
(provide one side enter 1, bothsides enter 2, otherwise enter 0)
- 2 = 0.738677 t/m

### Total S.I.D.L Reaction
= 5.25 t/m
= 169.2086 t

### Due to girders:

**Reaction**
- \( R_A = (10.3375 \times 0.5 \times 25.4) + (14.99 \times 2.5) + (0.5 \times 12.66 \times 2.9) + 20.28 + 13.79 \)
- \( R_A = 221.1852 \text{ t} \)

**Due to SLAB**
- 120.75 t

**Due to SIDL**
- 84.6043 t

**Total DL + SIDL**
- 853.079 t
Longitudinal Force:

For Class A Single lane: \( F_h = \left( \begin{array}{c} 0.2 \times 55.4 \end{array} \right) = 11.080 \text{ t} \)

For class 70R wheeled: \( F_h = \left( \begin{array}{c} 0.2 \times 100 \end{array} \right) = 20.000 \text{ t} \)

For class A 2 lane: \( F_h = 0.2 \times 55.4 = 11.08 \text{ t} \)

Fixed Bearing

Free Bearing

\[
\begin{align*}
\text{i} & \quad F_h - \mu(R_g + R_q) & \mu(R_g + R_q) \\
\text{ii} & \quad F_h + \frac{\mu(R_g + R_q)}{2} & \mu(R_g + R_q)
\end{align*}
\]

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
<th>Longitudinal Moment (t-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70R Wheeled</td>
<td>32.03</td>
<td></td>
</tr>
<tr>
<td>Span dislodged</td>
<td>0.00</td>
<td></td>
</tr>
</tbody>
</table>
**Moment in Transverse Direction**

Eccentricity of vertical load in transverse direction,

(a) Class A Single lane  
\[
\frac{11.0 \times 0.40 - \frac{1.80}{2}}{2.00} = 4.2 \frac{\text{m}}{\text{m}}
\]

(b) Class A 2 lane  
\[
\frac{11.0 \times 0.40 - \left(\frac{1.80 \times 2 + 1.7}{2}\right)}{2.00} = 2.45 \frac{\text{m}}{\text{m}}
\]

(c) Class 70R Wheeled  
\[
\frac{11.0 \times 1.63 - \frac{1.93}{2}}{2.00} = 2.91 \frac{\text{m}}{\text{m}}
\]

**Footpath Live load:**

The effective span of girders = 32.200 m > 30.00 m

Therefore, Intensity of footpath live load = \( P = \frac{P' - 260 + \frac{4800}{L} \left(\frac{16.5 - W}{15}\right)}{400 \ \text{Kg/m}^2} \)

Therefore,
\[
= 289.07 \ \text{Kg/m}^2 = 0.289 \ \text{t/m}^2
\]

(a) When one side is loaded;
Load on each span = 0.289 x 1.5 x 32.200 = 13.962 t Say, 14.0 t

(b) When both sides are loaded;
Load on each span = 13.962 x 2 = 27.924 t Say, 28.0 t

Eccentricity of load in transverse direction:

(a) When one side is loaded = 6 - \(\frac{0.5 + 1.5}{2}\) = 4.75 m

(b) When both sides loaded = Nil

**Summary of Footpath Live load:**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Reaction</th>
<th>Longitudinal moment</th>
<th>Transverse moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>When one side is loaded</td>
<td>7.00</td>
<td>0.00</td>
<td>33.25</td>
</tr>
<tr>
<td>When both sides loaded</td>
<td>28.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**LOADING:**

<table>
<thead>
<tr>
<th>CASE</th>
<th>( R_A(t) )</th>
<th>( R_B(t) )</th>
<th>( R_C(t) )</th>
<th>( R_D(t) )</th>
<th>( M_L )</th>
<th>( M_T )</th>
<th>( R_B + R_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 1 Lane</td>
<td>MAX REA</td>
<td>13.17</td>
<td>28.63</td>
<td>19.71</td>
<td>7.49</td>
<td>9.812</td>
<td>203.03</td>
</tr>
<tr>
<td></td>
<td>MAX LONG</td>
<td>17.19</td>
<td>38.21</td>
<td>0</td>
<td>0</td>
<td>42.031</td>
<td>160.48</td>
</tr>
<tr>
<td>Class A 2 Lane</td>
<td>MAX REA</td>
<td>26.34</td>
<td>57.26</td>
<td>39.42</td>
<td>14.98</td>
<td>19.624</td>
<td>236.87</td>
</tr>
<tr>
<td></td>
<td>MAX LONG</td>
<td>34.38</td>
<td>76.42</td>
<td>0</td>
<td>0</td>
<td>84.062</td>
<td>187.23</td>
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<tr>
<td>70R Wheeled</td>
<td>MAX REA</td>
<td>5.3</td>
<td>43.7</td>
<td>46.87</td>
<td>4.13</td>
<td>3.487</td>
<td>263.56</td>
</tr>
<tr>
<td></td>
<td>MAX LONG</td>
<td>16.08</td>
<td>83.92</td>
<td>0</td>
<td>7.86</td>
<td>92.312</td>
<td>244.21</td>
</tr>
</tbody>
</table>

**SUMMARY OF LOAD ON PIER FROM SUPERSTRUCTURE**

<table>
<thead>
<tr>
<th></th>
<th>( P )</th>
<th>( M_L )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 D.L</td>
<td>683.87</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 S.I.D.L</td>
<td>197.21</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 L.L</td>
<td>Max Rea</td>
<td>96.68</td>
<td>19.624</td>
</tr>
<tr>
<td></td>
<td>Max Long</td>
<td>83.92</td>
<td>92.312</td>
</tr>
</tbody>
</table>
Wt of Substructure:

- GL
- H.F.L
- pile cap toplevel

Dimensions:
- 0.7
- 0.75
- 1.2
- 3.6
- 5.500
- 7.74
- 10
- 2.25
- 205.36
- 207.155
- 207.905
- 214.17
- 214.274
- 215.674
- 215.740
- 216.274
- 193.355
- 193.355
- 193.355
Loads from substructure

1 DRY Condition

1 Wt of Pier cap

<table>
<thead>
<tr>
<th>Type</th>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>10 x 0.7 x 3.20 x 2.4</td>
<td>= 53.76 t</td>
<td></td>
</tr>
<tr>
<td>Trapezoidal</td>
<td>7.75 x 0.7 x 3.20 x 2.4</td>
<td>= 41.664 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>95.424 t</td>
</tr>
</tbody>
</table>

2 Wt of Pier Shaft

<table>
<thead>
<tr>
<th>Type</th>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.000 x 7.119 x 1.5 x 2.4</td>
<td>= 102.514</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.7854 x 2.250 x 7.12 x 2.4</td>
<td>= 30.1927 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>132.706 t</td>
</tr>
</tbody>
</table>

3 Wt of Pile cap

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>56.07 x 1.8 x 2.4</td>
<td>= 242.24 t</td>
<td></td>
</tr>
</tbody>
</table>

4 Weight due to Earth

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>56.07 - 7.77</td>
<td>x 1.8 x 0.75</td>
<td>= 65.2144</td>
</tr>
</tbody>
</table>

2 H.F.L

1 Wt of Pier cap

<table>
<thead>
<tr>
<th>Type</th>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>10 x 0.7 x 3.20 x 2.4</td>
<td>= 53.76 t</td>
<td></td>
</tr>
<tr>
<td>Trapezoidal</td>
<td>7.75 x 0.7 x 3.20 x 2.4</td>
<td>= 41.664 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>95.424 t</td>
</tr>
</tbody>
</table>

2 Wt of Pier Shaft

<table>
<thead>
<tr>
<th>Type</th>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.000 x 1.5 x 0.10 x 2.4</td>
<td>= 1.4688 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4 x 1.5 x 7.02 x 1.4</td>
<td>= 58.9428 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.7854 x 2.250 x 0.10 x 2.4</td>
<td>= 0.4326 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.7854 x 2.250 x 7.02 x 1.4</td>
<td>= 17.3601 t</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>78.2043 t</td>
</tr>
</tbody>
</table>

3 Wt of Pile cap

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>56.07 x 1.8 x 1.4</td>
<td>= 141.307 t</td>
<td></td>
</tr>
</tbody>
</table>

4 Weight due to Earth

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Calculations</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>56.07 - 7.77</td>
<td>x 1.0 x 0.75</td>
<td>= 36.2302</td>
</tr>
</tbody>
</table>
H.F.L. Case:
Formation level = 218.139 m
H.F.L. = 214.172 m
Foundation level = 193.355 m

Vertical load on pile foundation:
Vertical load from substructure & Pile cap
\[
95.424 + 78.204 + 141.31 + 36.23 = 351.165 \text{ t Say, } 352.00 \text{ t}
\]
Pier cap Pier shaft pile cap earth Total load = 352.00 t

Water current pressure:
Maximum surface velocity at H.F.L. = 4.30 m/sec
So,
\[
P = 52 kV^2
\]
\[
= 52 \times k \times 4.30^2 = 961.48 \text{ k kg/m}^2
\]
\[
= 0.961 \text{ k t/m}^2
\]

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>k (kN/m²)</th>
<th>H.F.L. (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PIER CAP</td>
<td>0.961</td>
<td>0.897</td>
<td>214.172</td>
</tr>
<tr>
<td>PIER SHAFT</td>
<td>0.568</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PILE CAP</td>
<td>0.485</td>
<td>207.053 m</td>
<td></td>
</tr>
<tr>
<td>PILE</td>
<td>193.355 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Transverse direction;

Pressure per metre depth
(a) Pier cap
\[0.961 \times \cos 20^\circ \times 0.00 \times 1.50 = 0 \text{ t/m depth}\]
\[0.897 \times \cos 20^\circ \times 0.00 \times 1.50 = 0 \text{ t/m depth}\]

(b) Pier shaft
\[0.897 \times \cos 20^\circ \times 0.66 \times 1.50 = 0.834 \text{ t/m depth}\]
\[0.568 \times \cos 20^\circ \times 0.66 \times 1.50 = 0.528 \text{ t/m depth}\]

(c) Pile cap
\[0.568 \times \cos 20^\circ \times 1.50 \times 8.70 = 6.966 \text{ t/m depth}\]
\[0.485 \times \cos 20^\circ \times 1.50 \times 8.70 = 5.946 \text{ t/m depth}\]

Pressure & moment at pile cap bottom:
\[1 \times \left( 0 - 0 \right) \times 1.4 = 0 \text{ t x 9.619 m = 0 t-m}\]
\[1 \times \left( 0.834 - 0.558 \right) \times 7.119 = 3.762 \text{ t x 5.360 m = 20.16 t-m}\]
\[1 \times \left( 6.966 - 5.946 \right) \times 1.800 = 0.918 \text{ t x 1.200 m = 1.10 t-m}\]
Total load = 16.471 t = 38.022 t-m
Say, 17.00 t Say, 39.00 t-m

Longitudinal direction;
Pressure per metre depth
(a) Pier cap
\[0.961 \times \sin 20^\circ \times 0.00 \times 10.00 = 0.000 \text{ t/m depth}\]
\[0.897 \times \sin 20^\circ \times 0.00 \times 10.00 = 0.000 \text{ t/m depth}\]

(a) Pier shaft
\[0.897 \times \sin 20^\circ \times 1.50 \times 5.50 = 2.531 \text{ t/m depth}\]
\[0.568 \times \sin 20^\circ \times 1.50 \times 5.50 = 1.603 \text{ t/m depth}\]

(b) Pile cap
\[0.568 \times \sin 20^\circ \times 1.50 \times 7.74 = 2.254 \text{ t/m depth}\]
\[0.485 \times \sin 20^\circ \times 1.50 \times 7.74 = 1.924 \text{ t/m depth}\]

Pressure & moment at pile cap bottom:
\[1 \times \left( 0.000 - 0.000 \right) \times 1.400 = 0.000 \text{ t x 9.619 m = 0 t-m}\]
\[1 \times \left( 2.531 - 1.603 \right) \times 7.119 = 11.410 \text{ t x 5.360 m = 61.15 t-m}\]
\[1 \times \left( 2.254 - 1.924 \right) \times 1.800 = 0.297 \text{ t x 1.200 m = 0.36 t-m}\]
Total load = 18.473 t = 86.243 t-m
Say, 19.00 t Say, 87.00 t-m

Therefore, Total vertical load = P = 352.00 t
Longitudinal moment = M_L = 87.00 t-m
Transverse moment = M_T = 39.00 t-m
Case-I(1) : DRY. condition with L.L:

L.W.L. = 207.905 m
Pile cap top level = 207.155 m
Pile cap bottom level = 205.355 m

Vertical load on Pile foundation;
Load from superstructure : Max Reaction case
\[ \text{S.I.D.L. + D.L.} + \text{Live load} = 197.21 + 683.87 = 881.08 \text{ t} \]
\[ 977.76 \text{ t} \text{ Say, } 978.00 \text{ t} \]

Load from superstructure : Max Long.Moment case
\[ \text{S.I.D.L. + D.L.} + \text{Live load} = 83.92 \text{ t} \]
\[ 965.00 \text{ t} \text{ Say, } 965.00 \text{ t} \]

Load from substructure & pile cap =
\[ 95.42 + 132.71 + 242.24 + 65.214 = 535.59 \text{ t} \text{ Say, } 536.00 \text{ t} \]

Horizontal longitudinal force at bearing level for pier = 32.027 t
Moment = 32.027 x 10.619 = 340.1
Total = 340.09 Say, 341 t-m

<table>
<thead>
<tr>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>1514.00</td>
<td>360.62</td>
</tr>
<tr>
<td>Max Long</td>
<td>1501.00</td>
<td>433.31</td>
</tr>
</tbody>
</table>

Case-II(1) : H.F.L. case (Live load condition):

Water level = 214.172 m
Pile cap top level = 207.155 m
Pile cap bottom level = 205.355 m

Load from superstructure: Max Reaction case
\[ \text{S.I.D.L. + D.L.} + \text{Live load} = 197.21 + 683.87 = 881.08 \text{ t} \]
\[ 977.76 \text{ t} \text{ Say, } 978.00 \text{ t} \]

Load from superstructure: Max Long.Moment case
\[ \text{S.I.D.L. + D.L.} + \text{Live load} = 83.92 \text{ t} \]
\[ 965.00 \text{ t} \text{ Say, } 965.00 \text{ t} \]

Load from substructure & pile cap = 351.165 t \text{ Say, } 352.00 \text{ t}

<table>
<thead>
<tr>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>1330.00</td>
<td>447.62</td>
</tr>
<tr>
<td>Max Long</td>
<td>1317.00</td>
<td>520.31</td>
</tr>
</tbody>
</table>

Seismic forces do not govern for Zone II & III. Sample calculations have already been submitted.
Case-I(2) : DRY. case (Span dislodged condition) :

Vertical load on Pile foundation;
Load from superstructure : Max Reaction case
\[
\text{S.I.D.L. + D.L.} = 197.21 + 683.87 = 881.08 \text{ t}
\]
\[
\text{Live load} = 0.00 \text{ t}
\]
\[
\text{Total load} = \frac{881.08 \text{ t}}{882.00 \text{ t}} \text{ Say, 882.00 t}
\]

Load from substructure & pile cap = 535.585 t \text{ Say, 536.00 t}

Horizontal longitudinal force at bearing level for pier = 0.00 t
\[
\text{Moment} = 0.00 \times 10.619 = 0.000 \text{ t-m}
\]
\[
\text{Total} = 0.000 \text{ Say, 0 t-m}
\]

Transverse moment = 0.00 Say, 0.0 t-m

<table>
<thead>
<tr>
<th>P</th>
<th>(M_L)</th>
<th>(M_T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>1418.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Max Long</td>
<td>1418.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Case-II(2) : H.F.L. case (Span dislodged condition) :

Vertical load on Pile foundation;
Load from superstructure
\[
\text{S.I.D.L. + D.L.} = 197.21 + 683.87 = 881.08 \text{ t}
\]
\[
\text{Live load} = 0.00 \text{ t}
\]
\[
\text{Total load} = \frac{881.08 \text{ t}}{882.00 \text{ t}} \text{ Say, 882.00 t}
\]

Load from substructure & pile cap = 351.165 t \text{ Say, 352.00 t}

Horizontal longitudinal force at bearing level for pier = 0.00 t
\[
\text{Moment} = 0.00 \times 10.619 = 0.000 \text{ t-m}
\]
\[
\text{Total} = 0.000 \text{ Say, 0 t-m}
\]

<table>
<thead>
<tr>
<th>P</th>
<th>(M_L)</th>
<th>(M_T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>1234.00</td>
<td>87.00</td>
</tr>
<tr>
<td>Max Long</td>
<td>1234.00</td>
<td>87.00</td>
</tr>
</tbody>
</table>
DESIGN OF PILE LAYOUT

Diameter of pile $D = 1.20$ m

c/c distance ratio of pile, $K = 3.00 \times 1.20 = 3.60$ m

Projection of pile cap from the face = 0.15 m

NO. OF PILES : 7

\[
\begin{array}{c|c|c|c}
\text{PILE NO} & X & Z_L & Y \\
\hline
1 & 1.800 & 21.60 & 3.118 & 12.47 \\
2 & 1.800 & 21.60 & 3.118 & 12.47 \\
3 & 3.600 & 10.80 & 0.000 & - \\
4 & 0.000 & - & 0.000 & - \\
5 & 3.600 & 10.80 & 0.000 & - \\
6 & 1.800 & 21.60 & 3.118 & 12.47 \\
7 & 1.800 & 21.60 & 3.118 & 12.47 \\
\end{array}
\]

\[
\begin{align*}
\varepsilon x^2 & = 38.88 \\
\varepsilon y^2 & = 38.88
\end{align*}
\]

Where

\[x = \text{Distance of pile from C.G of pile group in longitudinal direction}\]

\[Z_L = \frac{\varepsilon x^2}{x}\]

\[y = \text{Distance of pile from C.G of pile group in transverse direction}\]

\[Z_T = \frac{\varepsilon y^2}{y}\]
## Pile Cap

Area of Pile cap = $56.07 \text{ m}^2$

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>$M_L$</th>
<th>$M_T$</th>
<th>Pile 1</th>
<th>Pile 2</th>
<th>Pile 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dry Case:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Rea</td>
<td>1514.00</td>
<td>360.62</td>
<td>236.87</td>
<td>251.97</td>
<td>218.58</td>
<td>249.68</td>
</tr>
<tr>
<td>Max Long</td>
<td>1501.00</td>
<td>433.31</td>
<td>244.21</td>
<td>254.07</td>
<td>213.95</td>
<td>254.55</td>
</tr>
<tr>
<td>Span Dislodged</td>
<td>1418.00</td>
<td>0.00</td>
<td>0.00</td>
<td>202.57</td>
<td>202.57</td>
<td>202.57</td>
</tr>
<tr>
<td><strong>H.F.L Case:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Rea</td>
<td>1330.00</td>
<td>447.62</td>
<td>275.87</td>
<td>232.84</td>
<td>191.40</td>
<td>231.45</td>
</tr>
</tbody>
</table>
### Max Long

<table>
<thead>
<tr>
<th>Span Dislodged</th>
<th>1317.00</th>
<th>520.31</th>
<th>283.21</th>
<th>234.94</th>
<th>186.76</th>
<th>236.32</th>
</tr>
</thead>
</table>

**Dry Case:**
- Max Rea: Pile 4
- Max Long: Pile 5
- Span Dislodged: Pile 6, Pile 7

**H.F.I. Case:**
- Max Rea: 190.00
- Max Long: 188.14
- Span Dislodged: 176.29

| Max Long | 216.29 | 182.8946 | 213.99 | 180.60 |
| Span Dislodged | 214.43 | 174.3071 | 214.91 | 174.79 |

### Design of Pile Layout

<table>
<thead>
<tr>
<th>Dry Case:</th>
<th>Pile 1 + Pile 2</th>
<th>Pile 1 + Pile 6</th>
<th>Pile 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>470.56</td>
<td>715.64</td>
<td>249.68</td>
</tr>
<tr>
<td>Max Long</td>
<td>468.02</td>
<td>723.53</td>
<td>254.55</td>
</tr>
<tr>
<td>Span Dislodged</td>
<td>405.14</td>
<td>607.71</td>
<td>202.57</td>
</tr>
</tbody>
</table>

**H.F.I. Case:**
- Max Rea: 424.24
- Max Long: 421.71
- Span Dislodged: 358.83

| Max Rea | 424.24 | 570.00 | 231.45 |
| Max Long | 421.71 | 564.43 | 236.32 |
| Span Dislodged | 358.83 | 528.86 | 184.34 |
### Load Case

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Total Load</th>
<th>Longitudinal Moment</th>
<th>Transverse Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P$</td>
<td>$M_L$</td>
<td>$M_T$</td>
</tr>
<tr>
<td><strong>I DRY Condition</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 With L.L</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Rea</td>
<td>1514.00</td>
<td>360.62</td>
<td>236.87</td>
</tr>
<tr>
<td>Max Long</td>
<td>1501.00</td>
<td>433.31</td>
<td>244.21</td>
</tr>
<tr>
<td>2 Span dislodged</td>
<td>1418.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td><strong>II H.F.L Condition</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 With LL</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Rea</td>
<td>1330.00</td>
<td>447.62</td>
<td>275.87</td>
</tr>
<tr>
<td>Max Long</td>
<td>1317.00</td>
<td>520.31</td>
<td>283.21</td>
</tr>
<tr>
<td>2 Span dislodged</td>
<td>1234.00</td>
<td>87.00</td>
<td>39.00</td>
</tr>
</tbody>
</table>

Number of Piles $N = 7.00$

#### Pile Design:

**Horizontal load per pile**

**Dry Condition**

- Total horizontal force with L.L $= 32.03$ t
- $= 4.5752822$ t per pile
- span dislodged condition $= 0.00$ t
- $= 0$ t per pile

**H.F.L Condition**

**Longitudinal Direction**

- Total horizontal force with L.L $= 32.03 + 19.00$ t
- $= 7.29$ t per pile
- span dislodged condition $= 0.00 + 19.00$ t
- $= 2.71$ t per pile

**Transverse Direction**

- Total horizontal force with L.L $= 17.00$ t
- $= 2.43$ t per pile
- span dislodged condition $= 17.00$ t
- $= 2.43$ t per pile

The pile is socketed into the rock and considered as fixed at the bottom for analysis and the Effective height of pile for analysis is taken from bottom of pile cap to top of rock level.

- Depth of Rock below bottom of pile cap level $= 10.000$ m
- Self weight of pile $= \frac{\pi \times 1.20^2 \times 10.000 \times 1 \ \text{nos.}}{4}$
  $= 11.310$ m$^3$
- Buoyant wt. @ 1.40 t/m$^3 = 15.834$ t
Load at Pile base:
Maximum load on a pile at pile cap bottom = 254.55 t
Minimum load on a pile at pile cap bottom = 139.97 t

Provide 23 nos 16 φ
= 46.2 cm²
= 45.2 cm²
0.4%
## DESIGN OF PILE SECTION

<table>
<thead>
<tr>
<th>INPUT DATA</th>
<th>MAX. LOAD</th>
<th>MIN. LOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td>D = DIA OF PILE (in m)</td>
<td>1.200</td>
<td>1.200</td>
</tr>
<tr>
<td>C = COVER FROM CENTRE OF BAR (in m)</td>
<td>0.075</td>
<td>0.075</td>
</tr>
<tr>
<td>M = MODULAR RATIO (as per IRC:6)</td>
<td>10.000</td>
<td>10.000</td>
</tr>
<tr>
<td>BD = BETA ANGLE FOR DEFINING N.A. (in degree)</td>
<td>82.90</td>
<td>62.14</td>
</tr>
<tr>
<td>AS = AREA OF STEEL (in sq. Cm)</td>
<td>46.244</td>
<td>46.244</td>
</tr>
<tr>
<td>P = VERTICAL LOAD (in tonnes)</td>
<td>254.55</td>
<td>139.97</td>
</tr>
<tr>
<td>BM = BENDING MOMENT (in t-m)</td>
<td>38.42</td>
<td>38.42</td>
</tr>
</tbody>
</table>

### SOLUTION

RO = OUTER RADIUS = D/2
RI = RADIUS OF REINFORCEMENT RING = D/2 - C
T = THICK. REINFORCEMENT RING = AS/2 * 3.142 * RI
B = BD * PI / 180
B1 = COS(B)
A = ALPHA ANGLE = ACOS (RO * B1 / RI)

NUM1 = (PI - B) / 8 + B3/32 + B1 * B2 / 3
NUM2 = 2 * (RO^3) / (1 + B1)
NUM3 = (RI^3) * T / (RO + RI * A1)
NUM4 = (M - 1) * PI + A * A2 / 2
NUM = NUMINATOR = NUM2 * NUM1 + NUM3 * NUM4
DENM1 = 2 * (RO^2) / (1 + B1)
DENM3 = 2 * (RI^3) * T / (RO + RI * A1)
DENM4 = (M - 1) * PI + A1 - A3 + A1
DENM = DENOMINATOR = DENM1 * DENM2 + DENM3 * DENM4

### CHECK FOR ECCENTRICITIES

CALCULATED ECCENTRICITY
EC = NUM / DENM

ACTUAL ECCENTRICITY
EC = M / P

### CHECK FOR STRESSES

NAC = DEPTH OF N.A. BELOW CENT. AXIS = RO * COS(B)
NAD = DEPTH OF N.A. FROM TOP = RO + NAC
DE = EFFECTIVE DEPTH = D - C

TS = PERMISSIBLE TENSILE STRESSES IN CONCRETE (in t/sq.m)
PERMISSIBLE COMPRESSIVE STRESSES IN STEEL (in t/sq.m)

<table>
<thead>
<tr>
<th></th>
<th>MAX. LOAD</th>
<th>MIN. LOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAC</td>
<td>0.074</td>
<td>0.280</td>
</tr>
<tr>
<td>NAD</td>
<td>0.674</td>
<td>0.880</td>
</tr>
<tr>
<td>DE</td>
<td>1.125</td>
<td>1.125</td>
</tr>
<tr>
<td>CC</td>
<td>895</td>
<td>342</td>
</tr>
<tr>
<td>TS</td>
<td>5985</td>
<td>950</td>
</tr>
<tr>
<td>PERMISSIBLE COMPRESSIVE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PERMISSIBLE TENSILE</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
PILE CAP DESIGN:

Total Load on Pile in One row Pile 1 & Pile 6 = 723.53 t
Total Load on Pile in Second row Pile 3 = 254.55 t
Lever arm for bending moment due to Pile 1 & Pile 6 = 1.05 m
Lever arm for bending moment due to Pile 3 = 2.85 m
Bending Moment in Transverse direction Pile 1 & Pile 6 = 759.71 t-m
Bending Moment in Transverse direction Pile 3 = 725.4677 t-m
Total Load on Pile 1 & Pile 2 = 470.56 t
Bending Moment in Longitudinal direction = 213.17 t-m
Wt of Pile cap in Transverse direction

Area of strip 1 = 13.92 m²
Wt due to strip 1 = 60.15 t
Moment due to strip 1 = 54.14 t-m
Area of strip 2 = 2.70 m²
Wt due to strip 2 = 11.66 t
Moment due to strip 2 = 31.49 t-m
Area of strip 3 = 5.61 m²
Wt due to strip 3 = 24.24 t
Moment due to strip 3 = 58.18 t-m

Moment due to pile cap (Pile 1 & Pile 6) = 85.63 t-m
Moment due to pile cap (Pile 3) = 58.18 t-m

Wt of Pile cap in Longitudinal direction

Area of strip 4 = 6.14 m²
Wt due to strip 4 = 26.50 t
Moment due to strip 4 = 15.94 t-m
Area of strip 5 = 0.84 m²
Wt due to strip 5 = 3.61 t
Moment due to strip 6 = 1.45 t-m

Total Bending Moment in Transverse direction Pile 1 & Pile 6 = 674.08 t-m
Total Bending Moment in Transverse direction Pile 3 = 667.28 t-m
Total Bending Moment in Longitudinal direction = 195.78 t-m

Design for Flexure

\[
\begin{align*}
M &= 35 \\
Fe &= 415 \\
\sigma_{cgc} &= 11.6667 \\
\sigma_{st} &= 200 \\
m &= 10 \\
k &= 0.37 \\
j &= 0.88 \\
Q &= 1.89 \\
\end{align*}
\]

Depth of Pile Cap = 1.8 m
Pile shall be embeded by 50 mm into Pile cap & provide cover for Reinforcement 75 mm = 1.6375

d\text{eff} \text{ available}

d\text{eff} \text{ required in Transverse direction pile1 & Pile 6} = 0.673187 m

d\text{eff} \text{ required in Transverse direction Pile 3} = 0.669786 m

d\text{eff} \text{ required in Longitudinal direction} = 0.39611 m

A_{st} \text{ required in Transverse direction due to pile 1 & pile 6} = 23464.05 mm²

A_{st} \text{ required in Transverse direction due to pile 3} = 23227.6 mm²

A_{st} \text{ required in Longitudinal direction} = 6815.029 mm²

A_{st} \text{ required in Transverse direction /m width} = 3033.341 mm²/m

A_{st} \text{ required in Transverse direction /m width} = 3002.773 mm²/m

A_{st} \text{ required in Transverse direction /m width} = 6036.114 mm²/m

Minimum area of steel required = 2160.00 mm²/m

(As per cl. 305.19 of IRC :21-2000)

A_{st} \text{ required in Longitudinal direction /m width} = 1336.28 mm²/m

Minimum area of steel required = 2160.00 mm²/m
Transverse direction on Bottom face:
Total reinforcement in transverse direction on bottom face = 6036.114 mm$^2$
Provide 32 $\phi$ 125 mm c/c = 6433.982 mm$^2$/m

Transverse direction on Top face:
Minimum area of steel required on top face = 1080 mm$^2$/m
Provide 16 $\phi$ 130 mm c/c = 1546.63 mm$^2$/m

Longitudinal direction on Bottom face:
Provide 20 $\phi$ 130 mm c/c = 2416.61 mm$^2$/m
Provide 20 $\phi$ 130 mm c/c = 2416.61 mm$^2$/m

Longitudinal direction on Top face:
Minimum area of steel required on top face = 1080 mm$^2$/m
Provide 16 $\phi$ 130 mm c/c = 1546.63 mm$^2$/m

Check for Shear
Distance of section S-S from face of Pier
Portion of Pile Cap effective for Shear = 1.6375 m
Pile 3 is fully effective for shear & Pile 1, Pile 6 are partially effective = 1.9625
Portion of Pile effective for Shear = 1.20 m
Shear due to Pile 3 = 254.55 t
Shear due to Pile 1 & Pile 6 = 7.54 t
Total Shear = 262.09 t
Wt of Pile cap up to section S-S = 39.15 t
Net Upward Shear = 222.94 t
Shear Stress = 17.60 t/m$^2$
Percentage reinforcement provided = 0.185242 %
Permissible Shear Stress = 19.13129 t/m$^2$

No shear reinforcement required Provide Minimum Shear
Design of Pier shaft:

**Calculation of Forces at Pier base**

(At Top of Pile cap)

Checking the section at R.L. = **207.16** m

Lever arm for Longitudinal forces = 8.819 m i.e (215.97 - 207.16)

**DRY With L.L.**

1. Vertical load -
   - S.I.D.L. = 197.21 t
   - Dead load = 683.87 t
   - Live load = 96.68 t
   - Pier cap = 95.42 t
   - Pier shaft = 132.706 t

   Total load = 1205.89 t

2. Longitudinal moment -
   - Due to Longitudinal force = 32.027 x 8.819 = 282.45 t-m
   - Total moment = 282.45 t-m

**H.F.L. With L.L.**

1. Vertical load -
   - S.I.D.L. = 197.21 t
   - Dead load = 683.87 t
   - Live load = 96.68 t
   - Pier cap = 95.42 t
   - Pier shaft = 78.204 t

   Total load = 1151.39 t

2. Longitudinal moment -
   - Due to earth = 0.000 t-m
   - Due to Longitudinal force = 282.45 t-m

   Total moment = 282.45 t-m

**DRY spandislodged condition**

1. Vertical load -
   - S.I.D.L. = 197.21 t
   - Dead load = 683.87 t
   - Pier cap = 95.42 t
   - Pier shaft = 132.71 t

   Total load = 1109.21 t

2. Longitudinal moment -
   - Due to Longitudinal force = 0.000 t-m

   Total moment = 0.00 t-m

**H.F.L. spandislodged condition**

1. Vertical load -
   - S.I.D.L. = 197.21 t
   - Dead load = 683.87 t
   - Pier cap = 95.42 t
Pier shaft \( = \frac{78.204}{\text{t}} \)

Total load \( = \frac{1054.71}{\text{t}} \) Say, \( 1054.71 \text{ t} \)

2 Longitudinal moment -
- Due to Longitudinal force \( = \frac{0.000 \times 8.819}{\text{t-m}} = 0 \text{ t-m} \)
- Due to earth \( = \frac{0.000}{\text{t-m}} = 0 \text{ t-m} \)

<table>
<thead>
<tr>
<th></th>
<th>( P )</th>
<th>( M_L )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Rea</td>
<td>1054.71</td>
<td>82.77</td>
<td>0.00</td>
</tr>
<tr>
<td>Max Long</td>
<td>1054.71</td>
<td>82.77</td>
<td>0.00</td>
</tr>
</tbody>
</table>

PLAN : PIER SHAFT

% Reinforcement provided \( = \frac{32669.422}{\text{mm}^2} \)

Summary of Loads at Abutment Shaft bottom:

1 DRY condition
- With L.L
- Max Rea 1205.89 302.62 236.87
- Max Long 1193.13 375.31 244.21
- Span dislodged 1109.21 0.00 0.00

2 H.F.L
- With L.L
- Max Rea 1151.39 385.39 264.15
- Max Long 1138.63 458.08 271.50
- Span dislodged 1054.71 82.77 0.00

Check for Cracked/Uncracked Section
- Length of section \( = 5329.3404 \text{ mm} \)
- Width of section \( = 1500 \text{ mm} \)
- Gross Area of section \( A_g = 7994010.6 \text{ mm}^2 \)
- Gross M.O.I of section \( I_{gxx} = 1.499E+12 \text{ mm}^4 \)
- Gross M.O.I of section \( I_{gyy} = 1.892E+13 \text{ mm}^4 \)

Transformed sectional properties of section:
Adopting
Modular ratio \( m \) = 10
Cover
Dia of Bars = 25 25
No of bars in tension face (longer) = 30
No of bars in compression face = 30
No of bars in shorter direction = 8
Total bars in section = 76
Steel Area \( A_s \) = 37306 mm\(^2\)
% of Steel = 0.4667 %

\[
A_{xx} = -14726 \quad \text{mm}^2
\]
\[
A_{xy} = -3927 \quad \text{mm}^2
\]

Area of concrete \( A_c = A_g - A_s \) = 7956704.2 mm\(^2\)
C.G of Steel placed on longer face = 677.5 mm
C.G of Steel placed on shorter face = 2592.2 mm
Transformed Area of Section \( A_{dfm} \) = 8329768.3 mm\(^2\)
Transformed M.I\(_{xx} = I_{gxx} + 2 \left[ m - \frac{1}{2} A_s ax^2 \right] \)
\[
= 1.37721E+12 \quad \text{mm}^4
\]
\[
Z_{xx} = \frac{M.I_{xx}}{d/2} = 1.836E+09 \quad \text{mm}^3
\]

Transformed M.I\(_{yy} = I_{gyy} + 2 \left[ m - \frac{1}{2} A_s ay^2 \right] \)
\[
= 1.84454E+13 \quad \text{mm}^4
\]
\[
Z_{yy} = \frac{M.I_{yy}}{d/2} = 6.922E+09 \quad \text{mm}^3
\]

Permissible stresses
Minimum Gross Moment of inertia \( I_{\min} \) = 1.499E+12 mm\(^4\)
Area of section \( r \) = 7994010.6 mm\(^2\)
Effective length of Pier shaft \( L \) = 7.119 m
Effective length \( L_{eff} \) = 12.458 m
Slenderness ratio = 28.771 < 50
Type of member = Shotr Column
Stress reduction coefficient

(IRC:21-2000 cl: 306.4.2,3)

\[ \beta = 1 \]

Permissible stresses : concrete

\[ \sigma_{\text{cbc}} = 11.67 \text{ N/mm}^2 \]
\[ \sigma_{\text{co}} = 8.75 \text{ N/mm}^2 \]

Tensile stress = 0.67 N/mm²

Permissible stresses : Steel

\[ \sigma_{\text{s}} = 200 \text{ N/mm}^2 \]

### Dry case

<table>
<thead>
<tr>
<th>S.No</th>
<th>Item Lights and Moments</th>
<th>Max Rea</th>
<th>Max Long</th>
<th>Span Dislodg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P</td>
<td>1205.89 t</td>
<td>1193.13 t</td>
<td>1109.21</td>
</tr>
<tr>
<td>2</td>
<td>M_L</td>
<td>302.62 t-m</td>
<td>375.31 t-m</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>236.87 t-m</td>
<td>244.21 t-m</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Actual(calculated) Stresses

<table>
<thead>
<tr>
<th></th>
<th>[ \sigma_{\text{co,cal}} = \frac{P}{A_{\text{fin}}} ]</th>
<th>[ \sigma_{\text{cbc,cal}} = \frac{M_L}{Z_{xx}} ]</th>
<th>[ \sigma_{\text{cbc,cal}} = \frac{M_T}{Z_{yy}} ]</th>
<th>[ \sigma_{\text{cbc,cal}} = 5 + 6 ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1.44769</td>
<td>1.43237</td>
<td>1.33162</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.64803</td>
<td>2.04388</td>
<td>0.00000</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.34218</td>
<td>0.35279</td>
<td>0.00000</td>
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</tr>
<tr>
<td>7</td>
<td>1.99021</td>
<td>2.39666</td>
<td>0.00000</td>
<td></td>
</tr>
</tbody>
</table>

Permissible Stresses

<table>
<thead>
<tr>
<th></th>
<th>[ \sigma_{\text{cbc}} ]</th>
<th>[ \sigma_{\text{co}} ]</th>
<th>[ \sigma_{\text{cbc}} ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>11.66667</td>
<td>11.667</td>
<td>11.66667</td>
</tr>
<tr>
<td>9</td>
<td>8.75</td>
<td>8.75</td>
<td>8.75</td>
</tr>
</tbody>
</table>

Check for Minimum steel area \( \text{mm}^2 \)

<table>
<thead>
<tr>
<th></th>
<th>Conc.Area Required for directstress (1)/(9)</th>
<th>1378159.3</th>
<th>1363576.4</th>
<th>1267667.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Conc.Area Required for directstress (1)/(9)</td>
<td>11025.274</td>
<td>10909</td>
<td>10141.343</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
<td>23982.032</td>
<td>23982</td>
<td>23982.032</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of ( A_g )</td>
<td>23982.032</td>
<td>23982</td>
<td>23982.032</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel ( \text{mm}^2 )</td>
<td>32669.422</td>
<td>32669</td>
<td>32669.422</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area ( \text{mm}^2 )</td>
<td>32669.422</td>
<td>32669</td>
<td>32669.422</td>
</tr>
</tbody>
</table>

Check for safety of section

\[ \frac{\sigma_{\text{co,cal}} + \sigma_{\text{cbc,cal}}}{\sigma_{\text{co}}} \] \[ \sigma_{\text{cbc}} \] < 1 < 1 < 1

Check for Cracked /Uncracked section

\[ \sigma_{\text{co,cal}} - \sigma_{\text{cbc,cal}} \] \[ \sigma_{\text{cbc,cal}} \] \[ \sigma_{\text{cbc,cal}} \] \[ \sigma_{\text{cbc,cal}} \] \[ \sigma_{\text{cbc,cal}} \] \[ \sigma_{\text{cbc,cal}} \]

-0.542526 -0.9643 -1.3316209

-0.67 -0.67 -0.67

Section to be designed as

<table>
<thead>
<tr>
<th>Uncracked</th>
<th>Cracked</th>
<th>Uncracked</th>
</tr>
</thead>
</table>

Permissible Basic tensile stress in concrete

<table>
<thead>
<tr>
<th>Uncracked</th>
<th>Cracked</th>
<th>Uncracked</th>
</tr>
</thead>
</table>

-0.67 -0.67 -0.67
## H.F.L case

<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>Max Rea</th>
<th>Max Long</th>
<th>Span Dislodg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P</td>
<td>1151.39</td>
<td>1138.63</td>
<td>1054.71</td>
</tr>
<tr>
<td>2</td>
<td>M_L</td>
<td>385.39</td>
<td>458.08</td>
<td>82.77</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>264.15</td>
<td>271.50</td>
<td>0.00</td>
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</table>

### Actual(calculated) Stresses

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>( \sigma_{co,cal} )</td>
<td>( P/A_{fm} )</td>
<td>1.38226</td>
</tr>
<tr>
<td>5</td>
<td>( \sigma_{cbc,cal} )</td>
<td>( M_L/Z_{ax} )</td>
<td>2.09877</td>
</tr>
<tr>
<td>6</td>
<td>( \sigma_{cbc,cal} )</td>
<td>( M_T/Z_{yy} )</td>
<td>0.38160</td>
</tr>
<tr>
<td>7</td>
<td>( \sigma_{cbc,cal} = 5 + 6 )</td>
<td></td>
<td>2.48038</td>
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</table>

### Permissible Stresses

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>( \sigma_{cbc} )</td>
<td></td>
<td>11.666667</td>
</tr>
<tr>
<td>9</td>
<td>( \sigma_{co} )</td>
<td></td>
<td>8.75</td>
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### Check for Minimum steel area \( \text{mm}^2 \)

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Conc.Area Required for directstress</td>
<td>((1)/(9))</td>
<td>1315871.2</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
<td></td>
<td>10526.97</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of ( A_g )</td>
<td></td>
<td>23982.032</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel ( \text{mm}^2 )</td>
<td></td>
<td>23982.032</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area ( \text{mm}^2 )</td>
<td></td>
<td>32669.422</td>
</tr>
</tbody>
</table>

### Check for safety of section

\[
\frac{\sigma_{co,cal} + \sigma_{cbc,cal}}{\sigma_{co} - \sigma_{cbc,cal}} < 1
\]

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>( \frac{\sigma_{co,cal} + \sigma_{cbc,cal}}{\sigma_{co} - \sigma_{cbc,cal}} )</td>
<td></td>
</tr>
</tbody>
</table>

### Check for Cracked/Uncracked section

\[
\frac{\sigma_{co,cal} - \sigma_{cbc,cal}}{\sigma_{co,cal}}
\]

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>( \frac{\sigma_{co,cal} - \sigma_{cbc,cal}}{\sigma_{co,cal}} )</td>
<td></td>
</tr>
</tbody>
</table>

Permissible Basic tensile stress in concrete: \( -0.67 \) mm\(^2\)

Section to be designed as: Cracked, Cracked, Uncracked
DESIGN OF PIER CAP

DESIGN OF PIER CAP (along Transverse Direction)

Depth of Pier Cap in Transverse Direction = 1400 mm
considering two layers of 32 φ + in longitudinal 32 φ in transverse direction

\[
a = \text{distance of cg of bearing from face of equivalent square} = 1535 \text{ mm}
\]
\[
\text{Cover} = 40,000 \text{ mm}
\]
\[
d_{\text{eff}} = 1312 \text{ mm}
\]
\[
\frac{a}{d} = 1.170221 > 1
\]

DESIGNED AS A CANTILEVER BEAM

Distance of bearing 1 from face of pier = 1535
Distance of bearing 2 from face of pier = 1535
Distance of bearing 3 from face of pier = 0
Distance of bearing 4 from face of pier = 0

Loads on bearings:

<table>
<thead>
<tr>
<th>Reaction</th>
<th>Ecc</th>
<th>1.2</th>
<th>3.4</th>
<th>5.6</th>
<th>7.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Dead Load</td>
<td>683.8704</td>
<td>0</td>
<td>171.0</td>
<td>171.0</td>
<td>171.0</td>
</tr>
<tr>
<td>2 SIDL</td>
<td>197.2086</td>
<td>49.302</td>
<td>49.302</td>
<td>49.302</td>
<td>49.302</td>
</tr>
<tr>
<td>3 LL, max Long</td>
<td>83.92</td>
<td>2.91</td>
<td>47.145</td>
<td>29.702</td>
<td>12.258</td>
</tr>
</tbody>
</table>

LL reaction with impact factor 50% 1.058901 = 49.92 t
Length of Cantilever at one end = 2.335 m
Length of Cantilever at other end = 2.335 m
Self wt of Cantilever = 19.4206 t
Torsion due to LL Reaction = 54.91412 t-m
Horizantal force due to LL = 8.006744 t
Corresponding moment due horizontal force = 3.736 t-m
Total Torsional Moment = 58.651 t-m

Equivalent bending moment \( T^*(1+D/b)/1.7 \) = 49.137 t-m
Equivalent Shear  \( 1.6 \times T/b \) = 26.62503 kN

Moment at face of pier due to DL = 338.1867 kN-m
Moment at face of pier due to LL = 76.64662 kN-m
Moment at face of pier due to Cantilever load = 20.157 kN-m

Total bending moment = 484.127 kN-m

\[
\begin{align*}
M &= 35 \text{ kN/mm}^2 \\
Fe &= 415 \text{ kN/mm}^2 \\
\sigma_{cs} &= 11.6667 \text{ kN/mm}^2 \\
\sigma_{st} &= 200 \text{ kN/mm}^2 \\
m &= 10 \\
k &= 0.37 \\
j &= 0.88 \\
Q &= 1.89 \text{ N/mm}^2 \\
\end{align*}
\]

Depth required \( \sqrt{M/Q \times b} \) = 0.873 m

Reinforcement required = 21032.97 mm²

Provide two layers of 32 \( f \) 14 nos + 32 \( f \) 14 nos in longitudinal direction = 22518.94 mm²

Shear force

Self wt at effective depth d = 6.98 kN
Total Shear force = 303.80 kN
Shear stress \( \tau_v = (V - M \times \tan(\beta))/bd \) = 48.18 kN/m²
% of reinforcement provided = 0.706 %
Shear Force carried by Concrete \( \tau_s \times b \times d \) = 99.46 kN

Net design S.F = 204.34 kN

Using 14 Legged stirrups at a spacing of 200 mm

Area of steel required /m run = 7634.766 mm²/m

Providing 12 \( f \) 14 legged @ 200 mm

Area of steel provided /m run = 7916.813

CHECK FOR TORSION (one span dislodged condition)

Dead load reaction on bearing = 85.48 kN
Sidl reaction on bearing = 24.65 kN
Horizontal force due to dead load & sidl reaction = 5.51 kN
Torsion \( T_s \) = 163.55 kN-m
\( X \) = 1.4 m
Y = m
x = inches
y = inches
\( \varepsilon X^2 + Y \)
\( \gamma' \)
\( \phi' (0.5^*(\gamma')^{0.5}*\varepsilon X^2 + Y) \)

\[ A/s = T/((\alpha^*x^*y^*f_c)) \]
\[ \alpha_i = 0.66 + 0.33 * y/x \]

DESIGN OF PIER CAP (along Longitudinal Direction)

Depth of pier cap considering two layers of 32 \( \phi \) + in longitudinal 32 \( \phi \) in transverse direction = 1400 mm

a = distance of cg of brg. from face of pier = 350 mm
b\_eff = tacking pedestal size for dispersion as 1mx1m = 1420 mm
d\_eff = 1312 mm

\[ a/d = 0.66 + 0.33 * y/x \]

Hence the pier cap has been designed as a bracket.
The design has been done as per "Design proposal for reinforced concrete corbels" by A.H. Mattock.

Dead load reaction coming on corbel = 341.94 t
Sidl reaction coming on corbel = 98.60 t
Live load reaction = 88.86 t

\( \gamma' \)

( as per load factores given by ACI-4) 

DL+SIDL+LL + self wt on corbel = 559.64 t
Ultimate force \( V_u \) = 810.1583 t

DL+SIDL+LL + self wt on corbel per bearing = 139.91 t
Ultimate force per bearing \( V_u \) = 202.5396 t
For Pot-Ptfe bearings \( \mu' \) = 0.05
Design value of Nuc = 10.13 t
Check for Nominal Shearstrength

\[ \phi = 0.85 \]

\[ \mu = 1.4 \]

\[ V = \frac{V_u}{\phi \mu f_y} \]

\[ < 0.2 F'_c \]

\[ F'_c = 0.8 f'_c \]

O.K

Calculation for Shear Friction Reinforcement

\[ A_{vf} = \frac{V_u}{\phi \mu f_y} \]

\[ = 41.012 \text{ cm}^2 \]

Calculation of flexural reinforcement

\[ M_u = \frac{V_u a + N_u (D - d_{eff})}{f_y} \]

\[ = 71.78 \text{ t-m} \]

\[ A_{f} = \frac{M_u}{\phi_f f_y (d_{eff} x / 2)} \]

\[ = 15.53 \text{ cm}^2 \]

Where \( x = \) depth of rectangular stress block

assume \( x = 1.876458 \text{ cm} \)

Depth of Rectangular Block

\[ x = \frac{A_{f} f_y}{\phi_f f'_c b_{eff}} \]

\[ = 1.90725 \text{ cm} \]

Calculation for Horizontal Tensile Steel

\[ A_t = \frac{N_u (\phi f_y)}{f_y} \text{ or} \]

\[ = 2/3 A_{vf} \]

\[ = A_f \]

\[ = 15.53 \text{ cm}^2 \]

whichever is maximum

\[ = 27.34 \text{ cm}^2 \]

Minimum reinforcement as per clause 11.9.5 of ACI-4

\[ = 0.04 f'_c b_{eff} d_{eff} / f_y \]

\[ = 50.280 \text{ cm}^2 \]

Calculation for Torsional Steel

*In flexural steel, reinforcement due to torsion is to be added also and to compared with minimum steel requirement.*

Torsion due to horizontal force as a result of the live load reaction coming on the bearing.

Horizontal force

\[ = 2.496 \text{ t} \]

Bending moment

\[ = 2.745706 \text{ t-m} \]

Ultimate torsion

\[ = 4.6677 \text{ t-m} \]

\[ X = 1.4 \text{ m} \]

\[ Y = 1.312 \text{ m} \]

\[ x = 55.118 \text{ inches} \]

\[ y = 51.654 \text{ inches} \]

\[ e X^2 + Y \]

\[ = 156923.8 \]

\[ f'_c \]

\[ = 280 \text{ kg/cm}^2 \]

\[ = 3984.4 \text{ psi} \]

\[ \phi \cdot (0.5 \cdot (f'_c)^{0.5} \cdot X^2 + Y) \]

\[ = 4209777.594 \text{ lb inch} \]

\[ = 48.45454 \text{ t-m} \]

\[ > 4.67 \text{ t-m} \]

(Thus reinforcement is provided to cater to the effect of full torsion, neglecting the effect of concrete)

\[ A_{s} / s = \frac{f'_c (\alpha_x x + \alpha_y y \cdot f_y)}{f_y} \]
\[ \alpha = 0.66 + 0.33 \frac{y}{x} \]
\[ A_{s} = \frac{A_{t}}{s} = \text{inch}^2/\text{inch} = 6.56 \text{ cm}^2/\text{m} \]

Longitudinal reinforcement as per ACI:
\[ A_{s} = 2 \cdot A_{t} \cdot (X1+Y1)/s = 33.883 \text{ cm}^2 \]

Total Steel = 27.342 + 33.88 = 61.225 cm² per 1.42
Therefore this is more than minimum steel.

Hence \( A_s \) design = 431.159 cm²

Provide 60 nos 32 \( \phi \)
Area of steel provided = 482.5486 cm²

Horizontal Stirrups = 0.5 \( A_s \) = 214.1441 cm²

Provide 20 nos 2 Legged 16 \( \phi \) in 3 Layers
Area of steel provided = 241.2743 cm²
PIER SHAFT .case 1

Depth of Section = 1.500 m
Width of Section = 5.329 m

along width-compression face- no of bar: 30 tension face- no of bar: 30
Dia (mm) 25 25
Cover (cm) 7.50 7.5

along depth-compression face- no of bar: 8 tension face- no of bar: 8
Dia (mm) 16 16
Cover (cm) 7.50 7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 1225.35 T
Mxx = 335.620 Tm
Myy = 236.870 Tm

Intercept of Neutral axis : X axis : = 27.149 m
: y axis : = 1.616 m

Concrete Stress Governs Design
Stress in Concrete due to Loads = 33.74 Kg/cm^2
Stress in Steel due to Loads = 25.47 Kg/cm^2
Percentage of Steel = .41%

PIER SHAFT .case 2

Depth of Section = 1.500 m
Width of Section = 5.329 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 1212.590 T
Mxx = 408.310 Tm
Myy = 244.210 Tm

Intercept of Neutral axis : X axis : = 26.643 m
: y axis : = 1.428 m

Concrete Stress Governs Design
Stress in Concrete due to Loads = 37.94 Kg/cm^2
Stress in Steel due to Loads = 74.00 Kg/cm^2
Percentage of Steel = .41%
PIER SHAFT .case 1

Depth of Section = 1.500 m
Width of Section = 5.329 m

along width-compression face- no of bar: 30  tension face- no of bar: 30
Dia (mm) 25 25
Cover (cm) 7.50 7.5

along depth-compression face- no of bar: 8  tension face- no of bar: 8
Dia (mm) 16 16
Cover (cm) 7.50 7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 1172.850 T
Mxx = 437.130 Tm
Myy = 270.330 Tm

Intercept of Neutral axis : X axis : = 23.847 m
: y axis : = 1.346 m

Concrete Stress Governs Design
Stress in Concrete due to Loads = 35.66 Kg/cm^2
Stress in Steel due to Loads = 67.03 Kg/cm^2
Percentage of Steel = .41 %

PIER SHAFT .case 2
23-11-06

Depth of Section = 1.500 m
Width of Section = 5.329 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 1160.090 T
Mxx = 509.820 Tm
Myy = 277.670 Tm

Intercept of Neutral axis : X axis : = 23.171 m
: y axis : = 1.160

Concrete Stress Governs Design
Stress in Concrete due to Loads = 46.53 Kg/cm^2
Stress in Steel due to Loads = 211.80 Kg/cm^2
Percentage of Steel = .41 %
INPUT FILE: 70RW.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. JOINT COORDINATES
   6. 1         0.000         .000          .000
   7. 2         0.4           .000          .000
   8. 3         30.4           .000          .000
   9. 4         30.8           .000          .000
  10. 5         30.84          0.0           0.0
  11. 6         31.24          .000          .000
  12. 7         61.24          0.0           0.0
  13. 8         61.64          0             0
14. MEMBER INCIDENCES
   15. 1         1         2
   16. 2         2         3
   17. 3         3         4
   18. 4         4         5
   19. 5         5         6
   20. 6         6         7
   21. 7         7         8
22. MEMBER PROPERTY CANADIAN
   23. 1 TO 7 PRI YD 2. ZD 1.
24. MEMBER RELEASE
   25. 4 END FY MZ
   26. 5 START FY MZ
27. CONSTANT
   28. E CONCRETE ALL
   29. DENSITY CONCRETE ALL
   30. POISSON CONCRETE ALL
31. SUPPORT
   32. 2 3 6 7 PINNED
33. DEFINE MOVING LOAD
   34. *TYPE 1 LOAD 4*17.0 2*12.0 8.0 DIS 1.37 3.05 1.37 2.13 1.52 3.96
   35. TYPE 1 LOAD 8 2*12 4*17 DIS 3.96 1.52 2.13 1.37 3.05 1.37
36. LOAD GENERATION 264
   37. TYPE 1 -100.2 0. 0. XINC .5
   38. TYPE 1 -56.8 0. 0. XINC .5
   39. TYPE 1 -13.4 0. 0. XINC .5
40. PERFORM ANALYSIS
41. ***MAX REA
42. LOAD LIST 159
43. PRINT SUPPORT REACTION
SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
-----------------------
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM-Z
  2 159 0.00  5.30  0.00  0.00  0.00  0.00
  3 159 0.00 43.70  0.00  0.00  0.00  0.00
  6 159 0.00 46.87  0.00  0.00  0.00  0.00
  7 159 0.00  4.13  0.00  0.00  0.00  0.00
44. LOAD LIST 236
45. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE    STRUCTURE TYPE = SPACE
<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>236</td>
<td>0.00</td>
<td>16.08</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>236</td>
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<td>83.92</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
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<td>236</td>
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<td>0.00</td>
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<td>0.00</td>
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<tr>
<td>7</td>
<td>236</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

46. FINISH

*********** END OF THE STAAD.Pro RUN ***********
DESIGN OF SUBSTRUCTURE (ABUTMENT)
DESIGN DATA

Formation Level = 218.139 m
Ground Level = 210.144 m
Lowest Water Level = 210.144 m
Highest Flood Level = 214.172 m
Bottom of pile cap Level = 207.144 m
Thickness of bearing & pedestal = 0.300 m
Length of Span expansion joint to expansion joint = 32.200 m
Clear Span = 30.000 m
Bouyancy factor = 1
Submerged density of earth = 1.0 t/cum
Saturated density of earth = 2.000 t/cum
Unit wt of concrete = 2.400 t/m³
Depth of Superstructure = 2.350 m
Thickness of wearing coat = 0.056 m

PILE DATA
Number of piles = 12
Diameter of pile = 1.20 m
Spacing of pile in longitudinal direction = 3.60 m
No. of spacing in longi. dir. = 3
Specing of pile in trans. dir. = 3.60 m
No. of specing in trans. dir. = 2
SEC MODULUS FOR LONG. MOMENTS, Zₗ outer = 36.00 m³
SEC MODULUS FOR LONG. MOMENTS, Zₗ inner = 108.00 m³
SEC MODULUS FOR TRANS. MOMENTS, Zₜ outer = 28.80 m³
SEC MODULUS FOR TRANS. MOMENTS, Zₜ inner = 0.00 m³
LENGTH OF PILE CAP IN LONG. DIRECTION = 12.30 m
LENGTH OF PILE CAP IN TRANS. DIRECTION = 8.70 m
Live Load - Two Lane of class A
- One Lane of 70R
Grade of Concrete - M 35
Grade of Reinforcement - 415 (HYSD)
Permissible Compressive stress in Concrete - 1190 t/m²
Permissible Tensile stress in Steel - 20400 t/m²
Modular ratio, m = 10
factor, k = 0.368
Lever arm factor, j = 0.877
Moment of Resistance = 192 t/m²
<table>
<thead>
<tr>
<th>Pile No</th>
<th>x</th>
<th>Z_L</th>
<th>y</th>
<th>Z_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.4</td>
<td>36</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
<td>2</td>
<td>1.8</td>
<td>108</td>
<td>0</td>
<td>0</td>
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<tr>
<td>3</td>
<td>1.8</td>
<td>108</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
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<td>5.4</td>
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<tr>
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<td>6</td>
<td>1.8</td>
<td>108</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
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<td>108</td>
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<td>28.8</td>
</tr>
<tr>
<td>8</td>
<td>5.4</td>
<td>36</td>
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<td>0</td>
</tr>
<tr>
<td>9</td>
<td>5.4</td>
<td>36</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
<td>10</td>
<td>1.8</td>
<td>108</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
<td>11</td>
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</tr>
<tr>
<td>12</td>
<td>5.4</td>
<td>36</td>
<td>3.6</td>
<td>28.8</td>
</tr>
<tr>
<td></td>
<td>194.4</td>
<td>103.68</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
No. of column = 3
Width = 0.700
Depth of SuperStructure = 2.35
Number of Longitudinal Girders = 4

Mid section
1 slab = 12*0.25 = 3 m²
2 Top Flange = 4*[0.15*(1.25)+0.5*(1.25+0.275)]*0.16 = 1.238 m²
3 Web = 4*[(2.1-(0.15+0.16)-(0.2+0.24))*0.275] = 1.485 m²
4 Bottom Bulb = 4*[(1*0.2)+(0.5*(1+0.275))*0.24] = 1.412 m²
Total = 4.135 m²
End Section

Depth of SuperStructure = 2.35
Number of Longitudinal Girders = 4

End section

1 slab = 12*0.25 = 3 m²
2 Top Flange = 4*[(0.15*1.25)+0.5*(1.25+0.6)]*0.107 = 1.1459 m²
3 Web = 4*[2.1-(0.15+0.107)-(0.2+0.132)]*0.6 = 3.6264 m²
4 Bottom Bulb = 4*[(1*0.2)+(0.5*(1+0.6))]*0.132 = 1.2224 m²
Total = 5.99 m²

DL per running meter at Intermediate section = 10.338 t/m
DL per running meter at End section = 14.987 t/m
### Intermediate Cross Girder
- Thickness of Cross Girder: 0.3
- Number of intermediate cross girders: 3
- Cross sectional area in elevation: 12.259 m²
- DI of intermediate cross girder: 27.582 t

### End Cross Girder
- Thickness of end Cross Girder: 0.45
- Number of intermediate cross girders: 2
- Cross sectional area in elevation: 18.028 m²
- DI of intermediate cross girder: 40.562 t

### S.I.D.L
- I. Wearing Coat @ 1.0725 t/m = 1.0725 t/m
- II. Crash barrier @ 1.00 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 2.00 t/m
- III. R.C.C Post & Railing @ 0.085 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 0 t/m

![Diagram of bridge design]
Wt due to Crashbarrier

<table>
<thead>
<tr>
<th>Component</th>
<th>Area</th>
<th>Wt</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.100</td>
<td>0.24</td>
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<td>2</td>
<td>0.0875</td>
<td>0.21</td>
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<tr>
<td>3</td>
<td>0.075</td>
<td>0.18</td>
</tr>
<tr>
<td>4</td>
<td>0.075</td>
<td>0.18</td>
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<tr>
<td>5</td>
<td>0.002</td>
<td>0.1541</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.96</td>
</tr>
</tbody>
</table>

Footpath precast slab

<table>
<thead>
<tr>
<th>Utilities</th>
<th>Area</th>
<th>Wt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miscellaneous</td>
<td>0.1525</td>
<td>0.3813 t/m</td>
</tr>
<tr>
<td>Footpath kerb</td>
<td>0.0563</td>
<td>0.1406 t/m</td>
</tr>
<tr>
<td>Sandfilling</td>
<td>0.2052</td>
<td>0.3693 t/m</td>
</tr>
</tbody>
</table>

IV Foot path precast s @ 0.38125 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 0.7625 t/m

V Footpath Kerb @ 0.140625 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 0.2813 t/m

VI Miscellaneous Utilities @ 0.2 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 0.4 t/m

VII Sandfilling in footpath @ 0.3693 t/m (provide one side enter 1, both sides enter 2, otherwise enter 0) = 0.7387 t/m

Total = 5.25 t/m

Total S.I.D.L Reaction = 169.21 t

Due to girders:

\[ R_A = (10.3375 \times 0.5 \times 25.4) + (14.99 \times 2.5) + (0.5 \times 12.66 \times 2.9) + 20.28 + 13.79 \]

\[ R_A = 221.19 \text{ t} \]

Due to SLAB = 120.75 t

Due to SIDL = 84.604 t

Total DL + SIDL = 853.08 t
**Longitudinal Force:**

As per clause 214.2 of IRC:6, horizontal braking force $F_h$ for each span is:

- **For Class A Single lane:** $F_h = 0.2 \times 55.4 = 11.080 \text{ t}$
- **For class 70R wheeled:** $F_h = 0.2 \times 100 = 20.000 \text{ t}$
- **For class A 2 lane:** $F_h = 0.2 \times 55.4 = 11.08 \text{ t}$

**Summary of Longitudinal Forces:**

<table>
<thead>
<tr>
<th>Span dislodged</th>
<th>Longitudinal horizontal force (t)</th>
<th>Longitudinal Moment (t-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70R Wheeled</td>
<td>32.86</td>
<td>0.00</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Moment in Transverse Direction

Eccentricity of vertical load in transverse direction,

(a) Class A Single lane
\[ = \frac{11.0 - 0.40}{200} \left( 1.80 \right) = 4.2 \]

(b) Class A 2 lane
\[ = \frac{11.0 - 0.40}{200} \left( 1.80 \times 2 + 1.7 \right) = 2.45 \]

(c) Class 70R Wheeled
\[ = \frac{11.0 - 1.63}{200} \left( 1.93 \right) = 2.91 \]

Footpath Live load:

The effective span of girders = \( 32.200 \text{ m} \geq 30.00 \text{ m} \)

Therefore, Intensity of footpath live load
\[ = P = P' - \frac{260 + \frac{4800}{L}}{32.2} \left( \frac{16.5 - W}{15} \right) \]

Where,
\[ P' = 400 \text{ Kg/m}^2 \]

Therefore,
\[ = 289.07 \text{ Kg/m}^2 = 0.289 \text{ t/m}^2 \]

(a) When one side is loaded;
Load on each span = \( 0.289 \times 1.5 \times 32.200 = 13.962 \) t Say, 14.0 t

(b) When both sides are loaded;
Load on each span = \( 13.962 \times 2 = 27.924 \) t Say, 28.0 t

Eccentricity of load in transverse direction:

(a) When one side is loaded
\[ = 6 - \left( \frac{0.45 + 1.5}{2} \right) = 4.80 \text{ m} \]

(b) When both sides loaded = Nil

<table>
<thead>
<tr>
<th>LOADING:</th>
<th>R_A(t)</th>
<th>R_B(t)</th>
<th>M_T</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 1 Lane</td>
<td>13.62</td>
<td>41.78</td>
<td>175.48</td>
<td>41.78</td>
</tr>
<tr>
<td>Class A 2 Lane</td>
<td>27.24</td>
<td>83.56</td>
<td><strong>204.72</strong></td>
<td><strong>83.56</strong></td>
</tr>
<tr>
<td>70R Wheeled</td>
<td>16.75</td>
<td>83.25</td>
<td>242.26</td>
<td>83.25</td>
</tr>
</tbody>
</table>

SUMMARY OF LOAD ON PIER FROM SUPERSTRUCTURE

<table>
<thead>
<tr>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 D.L.</td>
<td>341.9351813</td>
<td>0</td>
</tr>
<tr>
<td>2 S.I.D.L</td>
<td>98.60432467</td>
<td>0</td>
</tr>
<tr>
<td>3 L.L.</td>
<td>83.25</td>
<td>0</td>
</tr>
</tbody>
</table>
**COEFFICIENT OF ACTIVE EARTH PRESSURE**

As per Coulomb's theory, coefficient of active earth pressure is

$$K_a = \frac{\sin^2(\alpha - \delta)}{\sin(\alpha - \delta) \cdot \sin(\alpha + \phi) - \sin^2(\phi) \cdot \sin(\phi + \delta) \cdot \sin(\phi - \delta)}$$

Where

- $\phi =$ Angle of internal friction of earth
- $\alpha =$ Angle of inclination of back of wall
- $\delta =$ Angle of internal friction between wall & earth
- $\iota =$ Angle of inclination of backfill

Here

- $\phi = 30^\circ = 0.524$ Radian
- $\alpha = 90^\circ = 1.571$ Radian
- $\delta = 20^\circ = 0.349$ Radian
- $\iota = 0^\circ = 0$ Radian

Therefore, horizontal coefficient of active pressure $K_a \cos \phi = K_{ha} = 0.2794$

**CALCULATION OF ACTIVE EARTH PRESSURE**

![Diagram of active earth pressure calculation]

**L.W.L. Condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m$^2$)</th>
<th>Actual Width</th>
<th>Effective Width*</th>
<th>Force (t)</th>
<th>L.A. from bottom of pile cap</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge (height=1.2)</td>
<td>1.0</td>
<td>3.906</td>
<td>0.67</td>
<td>12.000</td>
<td>12.000</td>
<td>31.429</td>
<td>9.042</td>
<td>284.18</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>1.0</td>
<td>5.089</td>
<td>0.67</td>
<td>2.100</td>
<td>2.100</td>
<td>7.166</td>
<td>4.545</td>
<td>32.56</td>
</tr>
<tr>
<td>3</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>3.906</td>
<td>2.18</td>
<td>12.000</td>
<td>12.000</td>
<td>51.150</td>
<td>8.730</td>
<td>446.52</td>
</tr>
<tr>
<td>3a</td>
<td></td>
<td>1.0</td>
<td>5.089</td>
<td>2.84</td>
<td>2.100</td>
<td>4.200</td>
<td>60.778</td>
<td>4.545</td>
<td>276.20</td>
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<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>5.089</td>
<td>2.84</td>
<td>2.100</td>
<td>4.200</td>
<td>30.389</td>
<td>4.137</td>
<td>125.73</td>
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<tr>
<td>TOTAL</td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>180.91</td>
<td>1165.19</td>
<td></td>
</tr>
</tbody>
</table>

**b) Span Dislodge Condition**

| Net force = 180.91 t | 38.59 | = 142.32 t |
| Net moment = 1165.19 tm | 316.74 | = 848.45 tm |

* As per clause 710.4.3 of IRC : 78,
H.F.L. Condition

Water Level

8.995
3.587
2.2
5.028
3.967
3.906
6a
6b

Active Earth Pressure

0.559
2.1
8.70
8.95
8.995

0.061
4a
4

12
2.1
8.70

a) Service Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Actual Width</th>
<th>Effective Width</th>
<th>Force (t)</th>
<th>L.A. from bottom of pile cap</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1+2</td>
<td>Surcharge</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>38.594</td>
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<tr>
<td>4</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>3.906</td>
<td>2.183</td>
<td>12.000</td>
<td>7.800</td>
<td>33.248</td>
<td>9.012</td>
<td>163.848</td>
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<tr>
<td>4a</td>
<td></td>
<td>0.5</td>
<td>3.906</td>
<td>2.183</td>
<td>2.100</td>
<td>4.200</td>
<td>18.182</td>
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<tr>
<td>6a</td>
<td>Submgr</td>
<td>1.0</td>
<td>5.028</td>
<td>2.183</td>
<td>2.100</td>
<td>4.200</td>
<td>46.090</td>
<td>4.112</td>
<td>60.987</td>
</tr>
<tr>
<td>6b</td>
<td>Earth</td>
<td>0.5</td>
<td>5.028</td>
<td>1.405</td>
<td>2.10</td>
<td>4.200</td>
<td>14.832</td>
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</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>150.95</td>
</tr>
</tbody>
</table>

Design Force 150.95 t
Design Moment 1039.86 tm

b) Span Dislodge Condition

Net force = 150.95 - 38.59 = 112.35 t
Net moment = 1039.86 - 316.74 = 723.12 tm

Weight of Abutment

L.W.L. Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>L.A. from pile cap B</th>
<th>Moment about B</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Pile cap</td>
<td>1.0</td>
<td>12.300</td>
<td>8.70</td>
<td>2.000</td>
<td>2.40</td>
<td>513.65</td>
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<td>3158.94</td>
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<td>2</td>
<td>Counterfort</td>
<td>0.5</td>
<td>1.250</td>
<td>2.10</td>
<td>5.289</td>
<td>2.40</td>
<td>16.66</td>
<td>7.567</td>
<td>126.06</td>
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<tr>
<td>3</td>
<td></td>
<td>1.0</td>
<td>1.750</td>
<td>2.10</td>
<td>5.289</td>
<td>2.40</td>
<td>46.65</td>
<td>6.275</td>
<td>292.72</td>
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<tr>
<td>4</td>
<td>Cap</td>
<td>0.5</td>
<td>1.250</td>
<td>2.10</td>
<td>5.289</td>
<td>2.40</td>
<td>16.66</td>
<td>4.983</td>
<td>83.02</td>
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<tr>
<td>5</td>
<td>Dirt Wall</td>
<td>1.0</td>
<td>1.750</td>
<td>12.00</td>
<td>2.406</td>
<td>2.40</td>
<td>20.79</td>
<td>7.000</td>
<td>145.51</td>
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<tr>
<td>6</td>
<td>Return Wall</td>
<td>1.0</td>
<td>3.984</td>
<td>0.90</td>
<td>0.500</td>
<td>2.40</td>
<td>4.30</td>
<td>9.142</td>
<td>39.34</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>0.5</td>
<td>3.984</td>
<td>0.90</td>
<td>2.656</td>
<td>2.40</td>
<td>11.43</td>
<td>8.478</td>
<td>96.89</td>
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<td>TOTAL</td>
<td></td>
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<td>680.54</td>
<td>4258.7</td>
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</table>
Weight of Earth and LL surcharge

L.W.L. Condition

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>L.A.from pile cap B</th>
<th>Moment about B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front Earth below cap</td>
<td>1</td>
<td>7.15</td>
<td>8.7</td>
<td>5.289</td>
<td>2.000</td>
<td>658.0045</td>
<td>3.575</td>
<td>2352.366</td>
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</tr>
<tr>
<td>9</td>
<td>VOLUME DEDUCTED</td>
<td>-0.5</td>
<td>5.400</td>
<td>8.70</td>
<td>3.600</td>
<td>2.0</td>
<td>-169.13</td>
<td>1.800</td>
<td>-304.43</td>
</tr>
<tr>
<td>4</td>
<td>VOLUME DEDUCTED</td>
<td>-0.5</td>
<td>5.400</td>
<td>2.10</td>
<td>5.289</td>
<td>2.0</td>
<td>-59.98</td>
<td>3.600</td>
<td>-215.92</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>-1.0</td>
<td>1.750</td>
<td>2.10</td>
<td>5.289</td>
<td>2.0</td>
<td>-38.87</td>
<td>6.275</td>
<td>-243.94</td>
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<tr>
<td>Backfill Earth</td>
<td>1.0</td>
<td>5.150</td>
<td>8.70</td>
<td>8.995</td>
<td>2.0</td>
<td>806.04</td>
<td>9.725</td>
<td>7838.76</td>
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<td>2</td>
<td>VOLUME DEDUCTED</td>
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<td>1.250</td>
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<td>-13.88</td>
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<td>3.984</td>
<td>0.90</td>
<td>0.500</td>
<td>2.0</td>
<td>-3.59</td>
<td>9.142</td>
<td>-32.78</td>
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<td>8</td>
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<td>-0.5</td>
<td>3.984</td>
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<td>2.0</td>
<td>-9.52</td>
<td>8.478</td>
<td>-80.74</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1169.07</td>
<td>9208.3</td>
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</table>

LLSURCHARGE

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length</th>
<th>Width</th>
<th>Height</th>
<th>Density</th>
<th>Weight</th>
<th>L.A.from pile cap B</th>
<th>Moment about B</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY EARTH</td>
<td>1.00</td>
<td>5.150</td>
<td>8.70</td>
<td>1.20</td>
<td>2.000</td>
<td>107.53</td>
<td>9.725</td>
<td>1045.75</td>
<td></td>
</tr>
</tbody>
</table>

BOUYANCY FACTOR = 1

BOUYANT FORCE (H.F.L. Case) = 12.30 X 8.70 X 7.028 = 752.0663

MOMENT DUE TO BOUYANT FORCE @, TOE = 4625.208

SUMMARY OF FORCES AND MOMENTS:

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>Case. L.W.L.</th>
<th>Case. H.F.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class-70R</td>
<td>Service Cond.</td>
<td>Span dislodged</td>
</tr>
<tr>
<td>83.25</td>
<td>83.25</td>
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</tr>
<tr>
<td>Foot Path L.L.</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>D.L. of superstructure</td>
<td>341.94</td>
<td>341.94</td>
</tr>
<tr>
<td>S.I.D.L.</td>
<td>83.25</td>
<td>83.25</td>
</tr>
<tr>
<td>Total Vertical Load from substructure (a)</td>
<td>508.44</td>
<td>0.00</td>
</tr>
<tr>
<td>Vertical load from substructure (b)</td>
<td>1957.14</td>
<td>1849.61</td>
</tr>
<tr>
<td>Total Vertical Load V = (a) + (b)</td>
<td>2465.58</td>
<td>2465.58</td>
</tr>
<tr>
<td>Horizontal force at bearing</td>
<td>32.86</td>
<td>32.86</td>
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<tr>
<td>Horiz force due to active Earth Pressure</td>
<td>180.91</td>
<td>142.32</td>
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<td>Total Horizontal Force H =</td>
<td>213.78</td>
<td>213.78</td>
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<tr>
<td>Moment, due to Horiz. force at bearing</td>
<td>292.13</td>
<td>292.13</td>
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<tr>
<td>'eccen' due to C.G.of pile &amp; CG of bearing</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$M_a$ @ C.G. of pile group due to (a)</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>'eccen' due to C.G.of pile &amp; CG of (b)</td>
<td>1.265</td>
<td>1.131</td>
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<tr>
<td>$M_a$ @ C.G. of pile group due to (b)</td>
<td>2476.33</td>
<td>2476.33</td>
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<tr>
<td>Moment due to active earth pressure</td>
<td>1165.19</td>
<td>848.45</td>
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<tr>
<td>Net Longitudinal moment</td>
<td>-1019.006</td>
<td>-1243.454</td>
</tr>
<tr>
<td>Transverse moment</td>
<td>242.258</td>
<td>242.258</td>
</tr>
</tbody>
</table>
### Seismic Calculation

Seismic forces in longitudinal & transverse direction shall be generated only on superstructure and do not govern for Zone II & III, the design hence ignored.

### CALCULATION OF LOAD ON PILES

Total no. of Piles (N) = 12

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
<th>Z_L</th>
<th>Z_T</th>
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<tbody>
<tr>
<td>(outer)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(inner)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2465.58</td>
<td>-1019.006</td>
<td>242.258</td>
<td>36.00</td>
<td>108.00</td>
</tr>
<tr>
<td></td>
<td>28.80</td>
<td>0.00</td>
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</table>

#### L.W.L. Case

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>205.46</td>
<td>-28.31</td>
<td>-9.44</td>
<td>8.41</td>
<td>0.00</td>
<td>154.13</td>
<td>-34.54</td>
<td>-11.51</td>
<td>0.00</td>
<td>0.00</td>
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</tbody>
</table>

#### H.F.L. Case

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
<td>142.79</td>
<td>-31.79</td>
<td>-10.60</td>
<td>8.41</td>
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<td>91.46</td>
<td>-38.02</td>
<td>-12.67</td>
<td>0.00</td>
<td>0.00</td>
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</tbody>
</table>
DESIGN OF PILE CAP

NORMAL CASE (L.W.L.)

<table>
<thead>
<tr>
<th>P/N</th>
<th>M/Z L (outer)</th>
<th>M/Z L (inner)</th>
<th>M/Z T (outer)</th>
<th>M/Z T (inner)</th>
</tr>
</thead>
<tbody>
<tr>
<td>205.46</td>
<td>-28.31</td>
<td>-9.44</td>
<td>8.41</td>
<td>0.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PILE NO.</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>225.36</td>
<td>206.49</td>
<td>187.62</td>
<td>168.75</td>
<td>788.21</td>
</tr>
<tr>
<td>2</td>
<td>233.77</td>
<td>214.90</td>
<td>196.03</td>
<td>177.16</td>
<td>821.86</td>
</tr>
<tr>
<td>3</td>
<td>242.18</td>
<td>223.31</td>
<td>204.44</td>
<td>185.57</td>
<td>855.51</td>
</tr>
<tr>
<td>Total</td>
<td>701.31</td>
<td>644.70</td>
<td>588.09</td>
<td>531.48</td>
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</table>

DESIGNING THE SECTION PARALLEL TO THE TRAFFIC DIRECTION

CALCULATION OF BENDING MOMENT

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force/m</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads</td>
<td>SELF WEIGHT OF CONCRETE</td>
<td>1a</td>
<td>1.0</td>
<td>173.304</td>
<td>2.075</td>
</tr>
<tr>
<td></td>
<td>SELF WEIGHT OF EARTH</td>
<td>4</td>
<td>1.0</td>
<td>381.9</td>
<td>2.075</td>
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<tr>
<td></td>
<td>SELF WEIGHT OF EARTH</td>
<td>9</td>
<td>-1.0</td>
<td>-229.869</td>
<td>2.767</td>
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<tr>
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<td>SELF WEIGHT OF EARTH</td>
<td>4</td>
<td>1.0</td>
<td>0.000</td>
<td>2.075</td>
</tr>
<tr>
<td>Upward Load of Piles</td>
<td>Pile no. D1,D2,&amp; D3</td>
<td>PileD1-3</td>
<td>-531.477</td>
<td>3.400</td>
<td>-1807.023</td>
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<tr>
<td></td>
<td>Pile no. C1,C2,&amp; C3</td>
<td>PileC1-3</td>
<td>0.000</td>
<td>-0.200</td>
<td>0.000</td>
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<tr>
<td>TOTAL</td>
<td>-206.123</td>
<td>-1290.905</td>
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</tr>
</tbody>
</table>
Bending Moment at face of stem 1290.905 tm

Effective depth required 0.878 m

Effective depth provided 1.910 m

Area of Reinforcement required 43.4 cm²/m

Minimum area of steel 24.0 cm²/m

Provide 32 ø 42 nos

Provide minimum reinforcement at top face

Area of steel provided

Provide

Check for Shear

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force/m</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads</td>
<td>SELF WEIGHT OF CONCRETE</td>
<td>1a</td>
<td>1.0</td>
<td>93.542</td>
<td>1.120</td>
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<tr>
<td></td>
<td>SELF WEIGHT OF EARTH</td>
<td>4</td>
<td>1.0</td>
<td>206.1</td>
<td>1.120</td>
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<tr>
<td></td>
<td>SELF WEIGHT OF EARTH blank</td>
<td>9</td>
<td>-1.0</td>
<td>-99.259</td>
<td>1.120</td>
</tr>
<tr>
<td></td>
<td>SELF WEIGHT OF SOIL triang</td>
<td>4</td>
<td>1.0</td>
<td>0.000</td>
<td>1.120</td>
</tr>
<tr>
<td>Upward Load of piles</td>
<td>Pile no. D1,D2,D3</td>
<td>Pile1</td>
<td>-531.477</td>
<td>1.490</td>
<td>-791.901</td>
</tr>
<tr>
<td></td>
<td>Pile no. C1,C2,C3</td>
<td>Pile2</td>
<td>0.000</td>
<td>-2.110</td>
<td>0.000</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>-331.050</td>
<td></td>
<td>-567.422</td>
</tr>
</tbody>
</table>

Effective depth (d') at distance d 1.910 m

Shear force at critical section 331.05 t

Bending Moment at critical section 567.42 tm

\[ \tan \beta = 0.00 \]

Net shear force S 331.05 t

Hence, Shear stress 19.92 t/m²

% of reinforcement 0.257

Permissible shear stress 23.68 t/m²

Hence O.K., No need of shear reinforcement

Providing 2 legged 12 ø stirrups @ 250 c/c
DESIGN OF PILE CAP ON HEEL SIDE

BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads</td>
<td>SELF WEIGHT OF CONCRETE 1a</td>
<td>1.0</td>
<td>162.864</td>
<td>1.950</td>
<td>317.585</td>
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<tr>
<td></td>
<td>SELF WEIGHT OF EARTH 2a</td>
<td>1.0</td>
<td>610.401</td>
<td>1.950</td>
<td>1190.281</td>
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<tr>
<td></td>
<td>SELF WT OF EARTH tring 0.5</td>
<td>0.5</td>
<td>0.000</td>
<td>1.950</td>
<td>0.000</td>
</tr>
<tr>
<td>Surcharge</td>
<td></td>
<td></td>
<td>81.432</td>
<td>1.950</td>
<td>158.792</td>
</tr>
<tr>
<td>Upward Load of piles</td>
<td>Pile no. B1,B2,B3 0.0</td>
<td>0.0</td>
<td>0.000</td>
<td>-0.450</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Pile no. A1,A2,A3 0.0</td>
<td>0.0</td>
<td>-701.312</td>
<td>3.150</td>
<td>-2209.132</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>153.385</td>
<td>-542.473</td>
<td></td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 542.473 tm
Effective depth required 0.569 m
Effective depth provided at face of stem 1.910 m
Area of Reinforcement required 18.24 cm²/m
Hence, Provide
20 ø , @ 125 C/C
0 ø , @ 125 C/C
Area of steel provided 25.13 cm²/m % OF STEEL EXTRA 37.77

Check for Shear (Critical section at face of stem)
Shear force at face of stem 153.39 t
\(\tan \beta = 0.000\)
Bending moment at face of stem 542.473 tm
Net shear force \(S \cdot M \cdot \tan \beta / d\) 153.39 t
Hence, Shear stress at face of stem 9.23 t/m²
Shear force at distance 1.910
%
% of reinforcement 0.13
Permissible shear stress 20.40 t/m² Hence O.K.
**DESIGNING THE SECTION PERPENDICULAR TO THE TRAFFIC DIRECTION**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum design moment</td>
<td>256.7</td>
</tr>
<tr>
<td>Width taken for design purpose</td>
<td>4.250</td>
</tr>
<tr>
<td>Depth required (effective)</td>
<td>0.560</td>
</tr>
<tr>
<td>Depth provided (effective)</td>
<td>1.910</td>
</tr>
<tr>
<td>Area of steel required per meter of slab cm²</td>
<td>17.668</td>
</tr>
<tr>
<td>Minimum steel required cm²</td>
<td>24.06</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>0.00</td>
</tr>
<tr>
<td>Maximum shear stress</td>
<td>0.00</td>
</tr>
<tr>
<td>% of reinforcement</td>
<td>0.126</td>
</tr>
<tr>
<td>Permissible shear stress</td>
<td>20.400</td>
</tr>
</tbody>
</table>

Hence O.K., No need of shear reinforcement

Provide 20 φ, @ 125 C/C

Area of steel provided 25.13 cm²
DESIGN OF COLUMN

Case : L.W.L. service condition.

(c) CHECKING THE INTERMEDIATE SECTION, AT R.L. = 209.14 m

CALCULATION OF EARTH PRESSURE

Horizontal coefficient of Active earth pressure = \( K_a \cos \phi = K_{ha} = 0.2794 \)

Active Earth Pressure

Surcharge

Actual Width

R.L. of Section = 209.14 m

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Actual Width</th>
<th>Effective Width</th>
<th>Force (t)</th>
<th>L.A. (m)</th>
<th>Moment (tm)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>Surchage</td>
<td>1.00</td>
<td>3.906</td>
<td>0.67</td>
<td>12.00</td>
<td>12.00</td>
<td>31.43</td>
<td>7.04</td>
<td>221.3</td>
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<tr>
<td>2</td>
<td>Dry Earth</td>
<td>1.00</td>
<td>5.089</td>
<td>0.67</td>
<td>2.10</td>
<td>2.10</td>
<td>7.17</td>
<td>6.45</td>
<td>46.2</td>
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<tr>
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<td>Dry Earth</td>
<td>0.50</td>
<td>3.91</td>
<td>2.18</td>
<td>12.00</td>
<td>7.80</td>
<td>33.25</td>
<td>6.73</td>
<td>223.7</td>
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<tr>
<td>4</td>
<td>Dry Earth</td>
<td>0.50</td>
<td>9.00</td>
<td>5.03</td>
<td>2.10</td>
<td>4.20</td>
<td>94.94</td>
<td>3.78</td>
<td>358.7</td>
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<tr>
<td>Total</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>166.78</td>
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<td>850.0</td>
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</tbody>
</table>

Horizontal force at bearing due to superstructure = 32.86 t

Total longitudinal force & moment on all counterfort = 199.65 t & 1066.5 t

Longitudinal force on each column = 66.55 t

Longitudinal moment on each column = 355.50 tm
**TRANSVERSE MOMENT**

Traffic direction

Transverse moment at the section = 16.94 tm

**CALCULATION OF VERTICAL LOAD**

Vertical Load (refer STAAD output) = 290.24 t

The design has been done on BIAx1 software
Input data of forces & moment required to run the BIAx1

Cross-section = 4.25 m x 0.70 m²
Vertical load = 290.24 t
Transverse moment = 16.94 tm
Longitudinal moment = 355.50 tm

Section A-A
CHECK FOR SHEAR

Checking the section at R.L. = 209.14 m

Shear Force = 66.55 t

Area of section = 4.25 x 0.70 m²

Shear stress = 22.37 t/m²

% of tensile reinforcement = 0.30%

Vertical load = 290.24 t

Multiplication factor as per IRC: 21 clause no. 304.7.1.3.3. = 1.14

Permissible shear stress = 28.39 t/m²

Design shear force = (22.37 - 28.39) x (4.25 x 0.70) = 17.92 t

Area of reinforcement required for shear = 17.92 x 10000.00 / 20400.00 (4.25 - 0.15) = 2.14 cm²/m
**DESIGN OF ABUTMENT CAP**

Reaction due to (DL+SIDL) = 441 t
Reaction due to live load = 83 t
Assuming impact factor as 0.12
Additional load due to impact = 10.0
Total reaction = 440.5395059 + 83.25 + 10.0 = 533.8 t
Transverse moment due to live load = 242.3 t-m
Total no. of bearings = 4
c/c distance between bearings = 2.8 m
Section modulus = 1.4 \(2 \times 2 + 4.2 \times 2\) = 39.2 m²
Load due to transverse moment in outer bearing = \(242.3 \times 4.2\) = 25.96 t
Load due to transverse moment in inner bearing = \(242.3 \times 1.4\) = 8.652 t

**MAX LOAD**
Total load on outer bearing = \(\frac{533.8 + 25.96}{4}\) = 159 t & \(\frac{533.8 - 25.96}{4}\) = 107 t
Total load on third outer bearing = \(\frac{533.8 + 8.652}{4}\) = 142 t & \(\frac{533.8 - 8.652}{4}\) = 125 t

**MIN LOAD**
Size of pedestal = 0.8 x 0.8 mm
Width of cap = 1.75 m
Thickness of cap = 1.00 m
Thickness of dirt wall = 0.30 m
Distance between face of dirt wall & c.l. bearing = 0.700 m
Distance between edge of cap & c.l. bearing = 1.75 - 0.700 - 0.30 = 0.750 m
From STAAD analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum negative B.M.</td>
<td>135.00</td>
<td>t-m</td>
</tr>
<tr>
<td>Maximum positive B.M.</td>
<td>34.90</td>
<td>*</td>
</tr>
<tr>
<td>Maximum S.F.</td>
<td>159.00</td>
<td>t</td>
</tr>
</tbody>
</table>

at d from face

- Effective depth required: \( \sqrt{\frac{135.00}{192.3 \times 1.75}} = 0.633 \text{ m} \)
- Effective depth provided: \( 1.00 - 0.05 = 0.95 \text{ m} \)

<table>
<thead>
<tr>
<th>Area of steel (at top)</th>
<th>135.0 x 1E+04</th>
<th>79.41 cm²</th>
<th>98.17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of steel (at bottom)</td>
<td>34.9 x 1E+04</td>
<td>20.53 cm²</td>
<td></td>
</tr>
</tbody>
</table>

**CHECK FOR SHEAR**

**Checking the section at a distance ‘d’ from support towards inner side**

- Shear stress: \( \frac{159}{1.75 \times 0.95} = 95.64 \text{ t/m}^2 \)
- Design shear: \( 95.63909774 \times 30.89 \times 1.75 \times 0.95 = 107.6 \text{ t} \)
- Permissible shear stress: \( 0.30 \text{ MPa} = 30.89 \text{ t/m}^2 \)

<table>
<thead>
<tr>
<th>Ast</th>
<th>107.6 x 1E+04</th>
<th>55.54 cm²/m</th>
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</thead>
<tbody>
<tr>
<td>sv</td>
<td>20400 x 0.95</td>
<td>20.53</td>
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Provide 6 Legged 12 φ `@` 122.2 c/c say 120 c/c

**Checking the section under the load**

- Shear stress: \( \frac{74.3}{1.75 \times 0.95} = 44.69 \text{ t/m}^2 \)
- Design shear: \( 44.7 \times 30.89 \times 1.75 \times 0.95 = 22.94 \text{ t} \)

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<th>Asv</th>
<th>22.94 x 1E+04</th>
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Provide 6 Legged 10 φ `@` 398 c/c say 200 c/c
DESIGN OF DIRT WALL

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<th>ELEMENT NO.</th>
<th>AREA FACTOR</th>
<th>HEIGHT / PRESSURE</th>
<th>WIDTH</th>
<th>FORCE / L.A.</th>
<th>MOMENT</th>
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<td>1</td>
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<td>2.71 / 0.67</td>
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<td>21.77 / 1.353</td>
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<td>46.32 / 57.36</td>
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<td>TOTAL</td>
<td></td>
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<td>13.93 / 3.55</td>
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</table>

BENDING MOMENT AT THE EDGE OF CAP
For dirt wall A: 57.36
For dirt wall B: 3.6

EFFECTIVE DEPTH REQUIRED
For dirt wall A: 0.16
For dirt wall B: 0.04

EFFECTIVE DEPTH PROVIDED
For dirt wall A: 0.25
For dirt wall B: 0.25

AREA OF STEEL REQUIRED PER METER WIDTH
For dirt wall A: 10.68 cm²/m
For dirt wall B: 0.66 cm²/m

CHECK FOR SHEAR

SHEAR FORCE AT THE EDGE OF CAP
For dirt wall A: 46.32
For dirt wall B: 13.9

SHEAR STRESS
For dirt wall A: 15.44 t/m²
For dirt wall B: 4.6 t/m²

% OF STEEL
For dirt wall A: 0.43
For dirt wall B: 0.03

PERMISSIBLE SHEAR STRESS
For dirt wall A: 29.2 t/m²
For dirt wall B: 20.4 t/m²
**DESIGN OF RETURN WALL**

Horizontal coefficient of Active earth pressure = $K_a \cos \phi = K_{ha} = 0.2794$

**CALCULATION OF BENDING MOMENT**

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<tr>
<th>Height from top h (m)</th>
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<th>1.000</th>
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<th>3.000</th>
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<td>Active Earth pressure ($K_a \gamma h$)</td>
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<td>0.56</td>
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<td>Total</td>
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<td>Length of cantilever (m)</td>
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<td>Cantilever Moment</td>
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<td>Required eff. Depth (m)</td>
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<td>Dia of bar (mm)</td>
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<td>20</td>
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<td>Spacing of bar (mm)</td>
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<td>175</td>
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**Check for Shear**

| shear stress at the section | 10.86 | 11.99 | 10.71 | 7.13 |
| % of steel                 | 0.35  | 0.33  | 0.17  | 0.06 |
| Permissible shear stress   | 26.60 | 28.59 | 26.26 | 24.73 |
| Hence                      | O.K.  | O.K.  | O.K.  | O.K. |
Abutment SHAFT .case 1

Depth of Section = 4.250 m
Width of Section = 0.700 m

along width-compression face- no of bar: 6  tension face- no of bar: 18
Dia (mm) 16  25
Cover (cm) 7.50  7.5

along depth-compression face- no of bar: 16  tension face- no of bar: 16
Dia (mm) 12  12
Cover (cm) 7.50  7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 102.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 289.910 T
Mxx = 338.500 Tm
Myy = 17.050 Tm

Intercept of Neutral axis : X axis : = 1.812 m
: y axis : = 3.805 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 33.15 Kg/cm^2
Stress in Steel due to Loads = 146.55 Kg/cm^2
Percentage of Steel = .46 %
INPUT FILE: spill rea.STD
1. STAAD PLANE
2. START JOB INFORMATION
3. ENGINEER DATE 02-NOV-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
   1 0 0 0; 2 1.8 0 0; 3 2.65 0 0; 4 4.6 0 0; 5 6 0 0; 6 7.4 0 0; 7 9.35 0 0
   8 10.2 0 0; 9 12 0 0; 10 2.65 -5.064 0; 11 6 -5.064 0; 12 9.35 -5.064 0
   13 1.65 -5.064 0; 14 10.95 -5.064 0
8. MEMBER INCIDENCES
   1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 3 10; 10 5 11
   11 7 12; 12 13 10; 13 10 11; 14 11 12; 15 12 14
9. DEFINE MATERIAL START
10. ISOTROPIC MATERIAL1
11. E 2.95E+006
12. POISSON 0.15
13. DENSITY 2.4
14. MEMBER PROPERTY INDIAN
15. 1 TO 8 PRIS YD 1 ZD 1.75
16. 9 TO 11 PRIS YD 0.7 ZD 3
17. 12 TO 15 PRIS YD 2 ZD 12.3
18. CONSTANTS
19. MATERIAL MATERIAL1 MEMB 1 TO 15
20. SUPPORTS
21. 10 TO 12 FIXED BUT FX MZ
22. LOAD 1 SELFWEIGHT
23. SELFWEIGHT Y -1
24. LOAD 2
25. JOINT LOAD
26. 2 8 FY -159
27. 4 6 FY -142
28. LOAD 3
29. MEMBER LOAD
30. 1 TO 8 UMOM GX 5.875
31. PERFORM ANALYSIS
32. PRINT MEMBER FORCES

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<th>FORCE-Y</th>
<th>FORCE-Z</th>
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************** END OF LATEST ANALYSIS RESULT **************

40. FINISH
INPUT FILE: CLA.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. JOINT COORDINATES
   1. 0.00 0.000 0.000
   2. 0.4 0.000 0.000
   3. 30.4 0.000 0.000
   4. 30.8 0.000 0.000
6. MEMBER INCIDENCES
   1 1 2
   2 2 3
   3 3 4
7. MEMBER PROPERTY CANADIAN
   1 TO 3 PRI YD 2. ZD 1.
   CONSTANT
8. E CONCRETE ALL
9. DENSITY CONCRETE ALL
10. POISSON CONCRETE ALL
11. SUPPORT
   2 3 PINNED
12. DEFINE MOVING LOAD
   25. TYPE 2 LOAD 2.7 2.7 2*11.4 4*6.8 DIS 3*3 4.3 1.2 3.2 1.1
   26. LOAD GENERATION 180
   27. TYPE 2 -57.6 0.0 0.0 XINC .5
   28. TYPE 2 -18.8 0.0 0.0 XINC .5
   29. PERFORM ANALYSIS
   30. LOAD LIST 140
   31. PRINT SUPPORT REACTION
   SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METER STRUCTURE TYPE = SPACE
   -------------------------------
   JOINT  LOAD  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM-Z
   2 140  0.00  13.62  0.00  0.00  0.00  0.00
   3 140  0.00  41.78  0.00  0.00  0.00  0.00
   ****************** END OF LATEST ANALYSIS RESULT ******************
32. FINISH
   ****************** END OF THE STAAD.Pro RUN ******************
ADDITIONAL SHEETS
(SEISMIC CHECK)
Seismic case

Calculation of seismic coefficient

**Transverse direction**

As per modified clause for interim measures for seismic provisions of IRC:6-2000

Horizontal seismic coefficient \( A_h = \frac{Z/2 \times S_a/g}{R/I} \)

Where

- Zone factor \( Z = 0.10 \)
- Importance factor \( I = 1.50 \)
- Response reduction factor \( R = 2.50 \)

\( S_a/g \) is average response acceleration coefficient for 5% damping depending upon fundamental period of vibration \( T \)

\[ T = 2.00 \left( \frac{D}{1000 \times F} \right) \]

Where

- \( D \) is appropriate dead load of superstructure and Live load
- \( DL = 341.94 \) t
- \( SIDL = 84.60 \) t
- \( LL = 83.56 \) t

\( DL+SIDL+50\% LL = 4683.20 \) kN

\( F \) is horizontal force required to be applied at centre of mass of superstructure for 1mm Horizontal deflection at top of abutment along considered direction of motion

Relation between deflection at \( x \) from free end of cantilever, subjected to force \( F \) at free end is given by

\[ F = \frac{E \times I \times y}{(L^2 \times x/2)-(x^3/6)-(L^3/3)} \]

Modulus of elasticity of concrete \( E = 29580.40 \) GPa

\( I = 2.32 \) m

Deflection at distance \( x \) from free end of cantilever

\( y = 0.001 \) m

Distance from fixed end of cantilever to point of application of load

\( L = 6.49 \) m

Distance of CG of super structure from deck slab top

\( 1.40 \) m

Distance of CG of super structure from free end of cantilever

\( x = 1.01 \) m

\( F = 997.07 \) kN

Corresponding time period

\( T = 0.1371 \) sec

\( S_a/g = 2.50 \)

Therefore

\( A_h = 0.075 \)
**Longitudinal direction**

As per modified clause for interim measures for seismic provisions of IRC:6-2000

Horizontal seismic coefficient

\[ A_h = \frac{Z/2 \times S_a/g}{R/I} \]

Where Zone factor \( Z = 0.10 \)
Importance factor \( I = 1.50 \)
Response reduction factor \( R = 2.50 \)

\( S_a/g \) is average response acceleration coefficient for 5% damping depending upon fundamental period of vibration \( T = 2.00 \) sec

\[ D = \frac{1000 \times F}{D} \]

Where

\( D \) is appropriate dead load of superstructure and Live load
For longitudinal direction, live load is not to be considered.

\[ DL + SIDL = 4265.40 \text{ kN} \]

\( F \) is horizontal force required to be applied at centre of mass of superstructure for 1mm horizontal deflection at top of abutment along considered direction of motion

Relation between deflection at \( x \) from free end of cantilever, subjected to force \( F \) at free end is given by

\[ F = \frac{E \times I \times y}{(L^2/2)-(x^3/6)-(L^3/3)} \]

Modulus of elasticity of concrete \( E = 29580.40 \text{ GPa} \)

\( I = 4.73 \text{ m}^4 \)

Deflection at distance \( x \) from free end of cantilever

\[ y = 0.001 \text{ m} \]

Distance from fixed end of cantilever to point of application of load

\[ L = 6.49 \text{ m} \]

Top of abutment from free end of cantilever

\[ x = 1.01 \text{ m} \]

\[ F = 2034.84 \text{ kN} \]

Corresponding time period

\[ T = 0.09 \text{ sec} \]

\[ S_a/g = 2.50 \]

Therefore

\[ A_h = 0.075 \]

Ground level 90.145

**Weight due to Concrete components**

<table>
<thead>
<tr>
<th>Component</th>
<th>Area factor</th>
<th>Length</th>
<th>Width</th>
<th>Height</th>
<th>Density</th>
<th>Force</th>
<th>L.A</th>
<th>Moment</th>
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<tbody>
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<td>1 Dirt wall</td>
<td>1</td>
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<td>0.3</td>
<td>2.706</td>
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<td>1.753</td>
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<td>2 Abut cap</td>
<td>1</td>
<td>12.0</td>
<td>1.75</td>
<td>1</td>
<td>2.4</td>
<td>3.780</td>
<td>7.789</td>
<td>29.442</td>
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<td>3 Abut shaft</td>
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<td>2.1</td>
<td>3</td>
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<td>4.133</td>
<td>4.875</td>
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</table>

Lever arm for the loads from super structure

| Formation level | 218.139 m |
| Wearing coat    | 56 mm     |
| Top of deck slab| 218.083 m |
| Distance of CG of super structure below deck slab top | 1.400 m |
| Pile cap bottom | 207.144 m |
| Lever arm for DL | 9.539 m |
Distance of CG of SIDL above deck slab top 0.261 m
Level arm for SIDL 11.200 m

Distance of CG of live load above formation level 1.2
Lever arm for live load 12.195

Load from Superstructure

### Longitudinal direction

<table>
<thead>
<tr>
<th>Weight</th>
<th>Force</th>
<th>L.A</th>
<th>moment</th>
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<td>9.539</td>
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<tr>
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<td>84.60</td>
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**Total** 31.99 315.70

### Transverse direction

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<th>moment</th>
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<tr>
<td>SIDL</td>
<td>84.60</td>
<td>6.345</td>
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<tr>
<td>LL 50%</td>
<td>41.70</td>
<td>3.134</td>
<td>12.20</td>
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**Total** 35.124 353.92

### Summary

**seismic longitudinal case**

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<th>M_T</th>
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<tbody>
<tr>
<td>Total load on piles for normal case</td>
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<td>-1019.01</td>
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<tr>
<td>Additional horizontal force and moment due to seismic</td>
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<td>-382.197</td>
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<tr>
<td>(Live load not to be considered for longitudinal moment)</td>
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<tr>
<td>Reduction in longitudinal moment due to braking force</td>
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<td>292.13</td>
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<tr>
<td>Reduction in transverse moment due to live load</td>
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<tr>
<td><strong>Total</strong></td>
<td>2465.58</td>
<td>-1109.070</td>
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**Seismic Transverse case**

<table>
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<tr>
<td>Total load on piles for normal case</td>
<td>2465.58</td>
<td>-1019.01</td>
</tr>
<tr>
<td>Additional horizontal force and moment due to seismic</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(Only 50% live load is to be considered for transverse moment)</td>
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</tr>
<tr>
<td>Reduction in longitudinal moment due to braking force</td>
<td>-</td>
<td>146.066</td>
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<tr>
<td>Reduction in transverse moment due to live load</td>
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<td><strong>Total</strong></td>
<td>2465.58</td>
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### SEISMIC ANALYSIS

Total no. of Piles (N) 12

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<thead>
<tr>
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<th>Z_L (inner)</th>
<th>Z_T (outer)</th>
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<th>M_L/Z_L (inner)</th>
<th>M_T/Z_T (outer)</th>
<th>M_T/Z_T (inner)</th>
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### TRAFFIC DIRECTION

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<td>1</td>
<td>236.27</td>
<td>215.73</td>
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<td>174.66</td>
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<td>215.73</td>
<td>195.20</td>
<td>174.66</td>
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<td>236.27</td>
<td>215.73</td>
<td>195.20</td>
<td>174.66</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
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<tr>
<td>210.91</td>
<td>194.74</td>
<td>178.58</td>
<td>162.41</td>
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<td>229.71</td>
<td>213.55</td>
<td>197.38</td>
<td>181.22</td>
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<tr>
<td>248.52</td>
<td>232.35</td>
<td>216.19</td>
<td>200.02</td>
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</table>

Permissible capacity of pile in normal case = 275 ton
Permissible capacity of pile in seismic case = 275 x 1.25 = 343.75 ton > 248 ton
**Horizontal load per pile for Normal case**

Horizontal force due to weight of earth = 0.00 \ t
Horizontal force due to braking = 32.86 \ t
Horizontal seismic force due to abutment above G.L. = 32.86 \ t

Number of Piles = 12.00
Number of Piles = 2.74 \ t per pile

Horizantal force in Normal condition due to earth = 180.91 \ t
Horizantal force in Normal condition due to earth = 15.08 \ t per pile

Therefore Total Horizontal force per pile = 17.81 \ t per pile

Maximum Load on the Pile = 242.18
Minimum Load on the Pile = 168.75

**Determination of Lateral Deflection of pile:**

\[ T = \frac{E \times I}{5 K_1} \]
\[ R = \frac{E \times I}{4 K_2} \]

- \( E \) = Young's modulus \( \text{kg/m}^2 \)
- \( K_1 \) = Constant \( \text{kg/cm}^3 \)
- \( K_2 \) = \( \text{kg/cm}^2 \)
- \( I \) = moment of inertia of pile cross section \( \text{cm}^4 \)

Dia of pile = 1.20 \ m

\[ E = 5000 \times \sqrt{35} = 30153 \text{ kg/cm}^2 \]

\[ I = \frac{3.14 \times 120^4}{64} = 1017860 \text{ cm}^4 \]

\[ K_1 = 0.775 \]
\[ K_2 = 0 \]

\[ T = 330.78 \]

\[ R = 0 \]

\( L_e \) = Embedded Length of pile
\( L_e = 207.19 - 199.5 = 769.4 \text{ cm} \)

\( L_t \) = Exposed Length
\( L_t = 207.09 - 206.99 = 10.4 \text{ cm} \)

\[ \frac{L_t}{T} = 0.0314 \]

\[ \frac{L_t}{T} = 2.15 \]

\[ L_t = 711.19 \text{ cm} \]
Pile Head deflection

\[ y = \frac{Q (L_1 + L_4)}{12EI} = \frac{17.815 \times 711.19}{12 \times 301533 \times 10178760} \]

= 0.174 cm
= 1.74 mm

In seismic case as permissible increase in deflection is 25%, hence
Permissible deflection = 5 x 1.25 = 6.25 mm > 1.7399 mm

Maximum Moment

\[ M_F = \frac{Q (L_1 + L_4)}{2} = 63.348 \text{ t-m} \]

\[ M = m \times M_F = 0.83 \times 63.348 = 52.58 \text{ t-m} \]

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm²

Provide 10 mm Rings at 190 mm c/c
### DESIGN OF PILE SECTION

<table>
<thead>
<tr>
<th>INPUT DATA</th>
<th>Normal Case</th>
<th>MAX. LOAD</th>
<th>MIN. LOAD</th>
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<tbody>
<tr>
<td>D=DIA OF PILE (in m)</td>
<td>1.200</td>
<td>1.200</td>
<td></td>
</tr>
<tr>
<td>C=COVER FROM CENTRE OF BAR (in m)</td>
<td>0.075</td>
<td>0.075</td>
<td></td>
</tr>
<tr>
<td>M=MODULAR RATIO (as per IRC:6)</td>
<td>10.000</td>
<td>10.000</td>
<td></td>
</tr>
<tr>
<td>BD=BETA ANGLE FOR DEFINING N.A. (in degree)</td>
<td>49.14</td>
<td>74.21</td>
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</tr>
<tr>
<td>AS=AREA OF STEEL (in sq. Cm)</td>
<td>60.319</td>
<td>60.319</td>
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<tr>
<td>P=VERTICAL LOAD (in tonnes)</td>
<td>242.18</td>
<td>168.75</td>
<td></td>
</tr>
<tr>
<td>BM=BENDING MOMENT (in t-m)</td>
<td>52.58</td>
<td>52.58</td>
<td></td>
</tr>
</tbody>
</table>

### SOLUTION

| RO=OUTER RADIUS=D/2 | 0.600 | 0.600 |
| RI=RADIUS OF REINFORCEMENT RING=D/2-C | 0.525 | 0.525 |
| T=THICK.REINFORCEMENT RING=AS/2*3.142*R1 | 0.002 | 0.002 |
| B=BD*PI/180 | 0.858 | 1.295 |
| B1=COS(B) | 0.654 | 0.272 |
| A=ALPHA ANGLE=ACOS (RO*B1/RI) | 0.726 | 1.255 |
| B2=COS(B)^3 | 0.433 | 0.891 |
| B3=SIN(B) | -0.285 | -0.892 |
| B4=SIN(B) | 0.990 | 0.524 |
| A=COS(A) | 0.748 | 0.311 |
| A2=SIN(2*A) | 0.993 | 0.591 |
| A3=SIN(A) | 0.664 | 0.950 |
| NUM1=(PI-B)/8+B3/3+B1*B2/3 | 0.371 | 0.284 |
| NUM2=2*(RO^3)/(1+B1) | 0.261 | 0.340 |
| NUM3=(RI^3)*T/(RO+RI*A1) | 0.000 | 0.000 |
| NUM4=(M-1)*PI+A-A2/2 | 28.50 | 29.23 |
| NUM = NUMINATOR = NUM2*NUM1+NUM3*NUM4 | 0.104 | 0.106 |
| DENM1=2*(RO^2)/(1+B1) | 0.435 | 0.566 |
| DENM2=2*B3+(PI-B)*B1/2+B1*B4/4 | 1.053 | 0.584 |
| DENM3=2*(RI^2) * T/(RO+RI*A1) | 0.001 | 0.001 |
| DENM=DENOMINATOR=DENM1+DENM2+DENM3+DENM4 | 0.480 | 0.341 |

### CHECK FOR ECCENTRICITIES

| CALCULATED ECCENTRICITY | 0.218 | 0.312 |
| ACTUAL ECCENTRICITY | 0.217 | 0.312 |

### CHECK FOR STRESSES

| NAC=DEPTH OF N.A.BELOW CENT.AXIS=RO*COS(B) | 0.393 | 0.163 |
| NAD=DEPTH OF N.A FROM TOP=RO+NAC | 0.993 | 0.763 |
| DE=EFFECTIVE DEPTH=D-C | 1.125 | 1.125 |
| CC=COMP. STRESS IN Conc. IN T/SQM=P/DENM | 505 | 494 |
| TS=TENS. STRESS IN STEEL (T/SQM) | 674 | 2343 |
| PERMISSIBLE COMP. STRESS IN Concrete 50% increase (in t/sq.m) | 1190 | 1190 |
| PERMISSIBLE TENSILE STRESS IN Steel 50% increase(in t/sq.m) | 20400 | 20400 |

Hence stresses in conc and steel are within permissible limits, hence safe.
Horizontal load per pile due to normal & seismic

Seismic Longitudinal direction:
- Horizontal seismic force due superstructure (DL+SIDL) = 31.99 t
  (Live load not to be considered for longitudinal direction)
- Horizontal seismic force due to abutment above G.L = 9.67 t
  = 41.66 t

Number of Piles = 12.00
= 3.47 t per pile

Horizontal force in Normal condition due to earth = 180.91 t
= 15.08 t per pile

Therefore Total Horizontal force per pile = 18.55 t per pile
Maximum Load on the Pile = 236.27
Minimum Load on the Pile = 174.66

Determination of Lateral Deflection of pile:

\[
T = \sqrt{\frac{E \times I}{5K_1}}
\]
\[
R = \sqrt[4]{\frac{E \times I}{K_2}}
\]

E = Young's modulus kg/m²
K₁ = Constant kg/cm³
I = moment of inertia of pile cross section cm⁴

Dia of pile = 1.20 m

\[
E = 5000 \times \sqrt{35} = 301533 \text{ kg/cm}^2
\]

\[
I = \frac{3.14 \times 120^4}{64} = 10178760 \text{ cm}^4
\]

K₁ = 0.775
K₂ = 0

\[
T = 330.78
\]
\[
R = 0
\]

\[
L_e = \text{Embedded Length of pile}
\]
\[
L_i = \text{Exposed Length}
\]

\[
L_e = 207.19 - 199.5 = 769.4 \text{ cm}
\]
\[
L_i = 207.09 - 206.99 = 10.4 \text{ cm}
\]

\[
\frac{L_i}{T} = 0.0314
\]
\[
\frac{L_e}{T} = 2.15
\]
\[
L_e = 711.19 \text{ cm}
\]
Pile Head deflection

\[ y = \frac{Q \left( L_1 + L_d \right)}{12EI} \]

\[ = \frac{18.547 \times 711.19}{12 \times 301533 \times 10178760} \]

\[ = 0.181 \text{ cm} \]

\[ = 1.81 \text{ mm} \]

In seismic case as permissible increase in deflection is 33%, hence

Permissible deflection = 5 x 1.33 = 6.65 mm > 1.8114 mm

Maximum Moment

\[ M_F = Q \times \frac{L_1 + L_d}{2} \]

\[ = 65.953 \text{ t-m} \]

\[ M = m \times M_F \]

\[ = 0.83 \times 65.953 = 54.74 \text{ t-m} \]

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm²

Provide 10 mm Rings at 190 mm c/c
### DESIGN OF PILE SECTION

<table>
<thead>
<tr>
<th>INPUT DATA</th>
<th>Seismic long. Case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MAX. LOAD</td>
</tr>
<tr>
<td>D=DIA OF PILE (in m)</td>
<td>1.200</td>
</tr>
<tr>
<td>C=COVER FROM CENTRE OF BAR (in m)</td>
<td>0.075</td>
</tr>
<tr>
<td>M=MODULAR RATIO (as per IRC:6)</td>
<td>10.000</td>
</tr>
<tr>
<td>BD=BETA ANGLE FOR DEFINING N.A. (in degree)</td>
<td>54.11</td>
</tr>
<tr>
<td>AS=AREA OF STEEL (in sq. cm)</td>
<td>60.319</td>
</tr>
<tr>
<td>P=VERTICAL LOAD (in tonnes)</td>
<td>236.27</td>
</tr>
<tr>
<td>BM=BENDING MOMENT (in t-m)</td>
<td>54.74</td>
</tr>
</tbody>
</table>

**SOLUTION**

RO=OUTER RADIUS=D/2
RI=INNER RADIUS=RO-C
T=THICKNESS
B=BD/PI/180
B1=COS(B)
A=ALPHA ANGLE=ACOS (RO*B1/RI)

### CHECK FOR ECCENTRICITIES

CALCULATED ECCENTRICITY EC = NUM/DENM
ACTUAL ECCENTRICITY EC = M/P

### CHECK FOR STRESSES

NAC=DEPTH OF N.A BELOW CENT.AXIS=RO*COS(B)
NAD=DEPTH OF N.A FROM TOP=RO+NAC
DE=EFFECTIVE DEPTH=D-C
CC=COMPRESSIVE STRESS IN CONC. IN T/SQM=P/DENM
TS=TENSILE STRESS IN STEEL (T/SQM)

PERMISSIBLE COMPRESSIVE STRESS IN Concrete 50% increase (in t/sq.m) 1785
PERMISSIBLE TENSILE STRESS IN Steel 50% increase (in t/sq.m) 30600

Hence stresses in conc and steel are within permissible limits, hence safe.
Horizantal load per pile due to normal & seismic

Seismic Transverse direction:

- Total horizontal force due to deck movement = 0.000 t
- Horizontal force due to earth = 0.000 t
- Horizantal seismic force due superstructure (DL+SIDL) = 31.990 t
- Horizantal seismic force due superstructure (LL) = 3.134 t
- Horizantal force due to abutment above G.L. = 9.666 t
- Horizantal force due to braking = 16.432 t
- Number of Piles = 44.79 t
- Horizontal force in Normal condition due to earth = 180.91 t
- Therefore Total Horizontal force per pile = 16.86 t
- Maximum Load on the Pile = 248.52 t
- Minimum Load on the Pile = 162.41 t

Determination of Lateral Deflection of pile:

\[ T = \frac{E \times I}{K_1} \]

\[ E = \text{Youngs modulus} \quad \text{kg/m}^2 \]

\[ K_1 = \text{Constant} \quad \text{kg/cm}^3 \]

\[ R = \frac{E \times I}{K_2} \]

\[ K_2 = \text{kg/cm}^2 \]

\[ I = \text{moment of inertia of pile cross section} \quad \text{cm}^4 \]

Dia of pile = 1.20

\[ E = 5000 \times \sqrt{35} = 30153 \quad \text{kg/cm}^2 \]

\[ I = \frac{3.14 \times 120}{64} = 101786 \quad \text{cm}^4 \]

\[ K_1 = 0.775 \]

\[ K_2 = 0 \]

\[ T = 330.78 \]

\[ R = 0 \]

L_e = Embeded Length of pile

\[ = 87.35 - 73.845 = 1350 \quad \text{cm} \]

L_i = Exposed Length

\[ = 87.35 - 87.35 = 0 \quad \text{cm} \]

\[ \frac{L_i}{T} = 0 \]

\[ \frac{L_e}{T} = 2.15 \]

\[ L_e = 711.19 \quad \text{cm} \]
Pile Head deflection

\[ y = \frac{Q (L_1 + L_4)}{12EI} \]

\[ = \frac{16.864 \times 711.19}{12 \times 301533 \times 10178760} \]

\[ = 0.165 \text{ cm} \]

\[ = 1.65 \text{ mm} \]

In seismic transverse case permissible increase in deflection = 1.25 x 5 = 6.65 mm > 1.65 mm

Maximum Moment

\[ M_F = Q x \left( \frac{L_1 + L_4}{2} \right) = 59.966 \text{ t-m} \]

\[ M = m x M_F \]

\[ = 0.83 \times 59.966 \]

\[ = 49.77 \text{ t-m} \]

Reinforcement in pile:

Provide 16 mm dia 30 nos 60.319 cm²

Provide 10 mm Rings at 190 mm c/c
### DESIGN OF PILE SECTION

<table>
<thead>
<tr>
<th>INPUT DATA</th>
<th>Seismic trans. Case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MAX. LOAD</td>
</tr>
<tr>
<td>D= DIA OF PILE (in m)</td>
<td>1.200</td>
</tr>
<tr>
<td>C= COVER FROM CENTRE OF BAR (in m)</td>
<td>0.075</td>
</tr>
<tr>
<td>M= MODULAR RATIO (as per IRC:6)</td>
<td>10.000</td>
</tr>
<tr>
<td>BD= BETA ANGLE FOR DEFINING N.A. (in degree)</td>
<td>42.28</td>
</tr>
<tr>
<td>AS= AREA OF STEEL (in sq. Cm)</td>
<td>60.319</td>
</tr>
<tr>
<td>P= VERTICAL LOAD (in tonnes)</td>
<td>248.52</td>
</tr>
<tr>
<td>BM= BENDING MOMENT (in t-m)</td>
<td>49.77</td>
</tr>
</tbody>
</table>

#### SOLUTION

- **RO** = OUTER RADIUS = D/2
- **RI** = RADIUS OF REINFORCEMENT RING = D/2 - C
- **T** = THICK REINFORCEMENT RING = AS/2 * 3.142 * RI
- **B1** = COS(B)
- **A1** = COS(A)
- **NUM1** = (PI - B)/8 + B3/32 + B1 * B2/3
- **NUM2** = 2 * (RO^3)/(1 + B1)
- **NUM3** = (RI^3) * T / (RO + RI * A1)
- **NUM4** = (M - 1) * PI * A - A2/2
- **NUM** = NUMINATOR = NUM2 * NUM1 + NUM3 * NUM4
- **DENM1** = 2 * (RO^2)/(1 + B1)
- **DENM2** = B2/3 + (PI-B) * B1/2 + B1 * B4/4
- **DENM3** = 2 * (RI^2) * T / (RO + RI * A1)
- **DENM4** = (M - 1) * PI * A - A3 + A * A1
- **DENM** = DENOMINATOR = DENM1 * DENM2 + DENM3 * DENM4

**CALCULATED ECCENTRICITY**: EC = NUM/DENM

**ACTUAL ECCENTRICITY**: EC = M/P

#### CHECK FOR STRESSES

- **NAC** = DEPT OF N.A.BELOW CENT.AXIS = RO*COS(B)
- **NAD** = DEPT OF N.A FROM TOP = RO + NAC
- **DE** = EFFECTIVE DEPTH = D - C
- **CC** = COMP. STRESS IN CONC. IN T/SQM = P/DENM
- **TS** = TENS. STRESS IN STEEL (T/SQM)
- **PERMISSIBLE COMP. STRESS IN Concrete 50% increase (in t/sq.m)**

Hence stresses in conc and steel are within permissible limits, hence safe.
DESIGN OF SUPERSTRUCTURE
### Bridge Data

#### Dimensional Data

<table>
<thead>
<tr>
<th></th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Total span c/c of Expansion Joint</td>
<td>32.2 m</td>
</tr>
<tr>
<td>2</td>
<td>Length of gap slab at both ends</td>
<td>0.7 m</td>
</tr>
<tr>
<td>3</td>
<td>Distance of centre of bearing from end of girder</td>
<td>0.4 m</td>
</tr>
<tr>
<td>4</td>
<td>span c/c of bearing I.e. effective span</td>
<td>30 m</td>
</tr>
<tr>
<td>5</td>
<td>skew angle of the bridge</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>width of clear carriageway</td>
<td>7.5 m</td>
</tr>
<tr>
<td>7</td>
<td>Extra widening in carriageway, if any</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>Width of crash barrier footpath side</td>
<td>0.5 m</td>
</tr>
<tr>
<td>9</td>
<td>Width of footpath</td>
<td>1.525 m</td>
</tr>
<tr>
<td>10</td>
<td>Width of railing curb</td>
<td>0.225 m</td>
</tr>
<tr>
<td>11</td>
<td>Width of median side curb</td>
<td>0.5 m</td>
</tr>
<tr>
<td>12</td>
<td>Half width of median including curb</td>
<td>2.25 m</td>
</tr>
<tr>
<td>13</td>
<td>Total width of bridge</td>
<td>12 m</td>
</tr>
<tr>
<td>14</td>
<td>Spacing of main Girders c/c</td>
<td>2.8 m</td>
</tr>
<tr>
<td>15</td>
<td>Number of main girders</td>
<td>4</td>
</tr>
<tr>
<td>16</td>
<td>Spacing of cross girders c/c</td>
<td>7.500 m</td>
</tr>
<tr>
<td>17</td>
<td>Thickness of deck slab</td>
<td>0.25 m</td>
</tr>
<tr>
<td>18</td>
<td>Thickness of wearing coat</td>
<td>0.056 m</td>
</tr>
<tr>
<td>19</td>
<td>Thickness of cantilever slab at fixed end</td>
<td>0.25 m</td>
</tr>
<tr>
<td>20</td>
<td>Thickness of cantilever slab at free end</td>
<td>0.25 m</td>
</tr>
<tr>
<td>21</td>
<td>Depth of precast girder including flange</td>
<td>2.1 m</td>
</tr>
<tr>
<td>22</td>
<td>Web thickness of precast girder at mid section</td>
<td>0.275 m</td>
</tr>
<tr>
<td>23</td>
<td>web thickness of precast girder at support section</td>
<td>0.6 m</td>
</tr>
<tr>
<td>24</td>
<td>Length of side flaring section in main girder</td>
<td>2.5 m</td>
</tr>
<tr>
<td>25</td>
<td>Length of widened section in main girder from end</td>
<td>2.9 m</td>
</tr>
<tr>
<td>26</td>
<td>Distance of end of main girder from centre of bearing</td>
<td>0.4 m</td>
</tr>
<tr>
<td>27</td>
<td>Length of widened section in main girder from bearing</td>
<td>2.5 m</td>
</tr>
<tr>
<td>28</td>
<td>Width of flange of precast girder at top</td>
<td>1.25 m</td>
</tr>
<tr>
<td>29</td>
<td>Width of flange of precast girder at bottom</td>
<td>1 m</td>
</tr>
<tr>
<td>30</td>
<td>Thickness of flange of precast girder at top</td>
<td>0.15 m</td>
</tr>
<tr>
<td>31</td>
<td>Size of top haunch</td>
<td>0.4875 m</td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>0.16 m</td>
</tr>
<tr>
<td>32</td>
<td>Size of bottom haunch</td>
<td>0.3625 m</td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>0.24 m</td>
</tr>
<tr>
<td></td>
<td>Depth</td>
<td>0.3 m</td>
</tr>
<tr>
<td>33</td>
<td>Size of bottom bulb</td>
<td>Width</td>
</tr>
<tr>
<td></td>
<td>Depth</td>
<td>Depth</td>
</tr>
<tr>
<td>34</td>
<td>Depth of Intermediate cross girder excluding deck slab</td>
<td>1.8 m</td>
</tr>
<tr>
<td>35</td>
<td>Depth of end cross girder excluding deck slab</td>
<td>1.8 m</td>
</tr>
<tr>
<td>36</td>
<td>Web thickness of intermediate cross girder</td>
<td>0.3 m</td>
</tr>
<tr>
<td>37</td>
<td>Web thickness of end cross girder</td>
<td>0.45 m</td>
</tr>
</tbody>
</table>

#### Properties

- **Grade of concrete for superstructure**: M 40 N/mm²
- **Grade of reinforcement for superstructure**: FY 415 N/mm²
- **Clear cover to reinforcement for slabs**: 0.04 m
- **Clear cover to reinforcement for beams**: 0.05 m
- **Unit weight of RCC (as per IRC:6-2000, CI 205)**: 2.4 t/m³
- **Unit weight of prestressed concrete (as per IRC:6-2000, CI 205)**: 2.5 t/m³
- **Unit weight of wearing coat**: 2.2 t/m³
- **Weight of crash barrier**: 1 t/m
- **Weight of railing, if any**: no
- **Weight of utilities under footpath slab**: 0.1 t/m
- **Allowable stress in concrete in bending compression**: 13.33 N/mm²
- **Allowable stress in steel in bending compression**: 200 N/mm²
- **Modular ratio**: 10
- **Poisson ratio for concrete**: 0.15
- **Modulus of elasticity**: 32500 N/mm²
- **Coefficient of thermal expansion**: 1.17E-05 per deg.C
General Features

**Girder Length**

<table>
<thead>
<tr>
<th>Feature</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span c/c of exp. Joint</td>
<td>32.200 m</td>
</tr>
<tr>
<td>Gap slab</td>
<td>0.700 m</td>
</tr>
<tr>
<td>Overall length of girder</td>
<td>30.800 m</td>
</tr>
<tr>
<td>Distance of centre of bearing from end of girder</td>
<td>0.400 m</td>
</tr>
<tr>
<td>Effective length of girder from c/c of bearing</td>
<td>30.000 m</td>
</tr>
</tbody>
</table>

**Sequence of deck construction**

1. Cast PSC beams at site
2. Stress Stage -I tendons in the beam
3. Cast the in-situ portion of cross girders and deck slab at top
4. Stress Stage -II tendons in the beam
5. Cast the gap slab, Crash barrier, Kerb etc.
6. Lay the wearing coat

**Design Sections**

For flexural and shear stress
1. mid span sections
2. 3/8 th section
3. 1/4 th sections
4. 1/8th span section
5. effective depth d

**Concrete Strength**

For analysis it has been assumed that the precast beams as well as the deck slab of the top shall have a minimum of M-40 grade concrete. However, the concrete shall be design mix and have a minimum 28 days characteristics strength of 40Mpa on 150 mm cubes for all elements of super structure.

**Codes of References**

(I) IRC: 5-1998
(II) IRC:6-2000
(iii) IRC:18-2000
(iv) IRC:21-2000
(v) IRC:22-1986

**Reinforcement**

Reinforcing steel shall be of HYSD bars (grade designation S:415) confirming to IS:1786

**Prestressing steel and Accessories**

Cable consisting of 12 No. of 12.7 mm dia 7 ply class 2 strand as per IS:6006-1983 shall be used for main stressing.

**Sheathing**

Sheathing shall be of “Drassbatch” type 75 mm ID manufactured from minimum 0.3 mm thick bright metal strip. It shall be as per IRC:18-2000, Appendix 1.
### SECTION PROPERTIES

**GIRDER (Outer)**

**AT MID SPAN**

<table>
<thead>
<tr>
<th>Self</th>
<th>Composite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td>2100</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>1106250</td>
</tr>
<tr>
<td>( I_z ) (mm⁴)</td>
<td>6.2283E+11</td>
</tr>
<tr>
<td>( Y_y ) (mm)</td>
<td>1008.95</td>
</tr>
<tr>
<td>( Y_t ) (mm)</td>
<td>1250</td>
</tr>
<tr>
<td>( Z_b ) (mm³)</td>
<td>6.173E+08</td>
</tr>
<tr>
<td>( Z_t ) (mm³)</td>
<td>5.709E+08</td>
</tr>
</tbody>
</table>

**Concrete** M- 40

<table>
<thead>
<tr>
<th>Element</th>
<th>Self = bd²/12</th>
<th>Composite</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.441E+10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1.030E+09</td>
<td>7.019E+09</td>
</tr>
<tr>
<td>3</td>
<td>2.860E+09</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>6.351E+08</td>
<td>5.805E+09</td>
</tr>
<tr>
<td>5</td>
<td>2.500E+10</td>
<td>0</td>
</tr>
</tbody>
</table>

\[ I_y = \frac{6.676E+10 \text{ mm}^4}{z_L = 1716.1} \]

\[ I_y = \frac{6.387E+11 \text{ mm}^4}{z_L = 1716.1} \]

\[ I_x = \frac{4.037E+10 \text{ mm}^4}{1000} \]

for deck slab \( I_x = b \times d^3 / 6 \)
**AT SUPPORT**

**Iz**

<table>
<thead>
<tr>
<th>Element</th>
<th>Self = (bd^2/12) (mm²)</th>
<th>(A_z^2) (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.44E+10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2.03E+08</td>
<td>5.78E+09</td>
</tr>
<tr>
<td>3</td>
<td>2.97E+10</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>5.88E+07</td>
<td>3.56E+09</td>
</tr>
<tr>
<td>5</td>
<td>2.50E+10</td>
<td>0</td>
</tr>
</tbody>
</table>

\[I_y(self) = 8.872E+10 \text{ mm}^4\]

\[z_L = 1731.6\]

\[I_y(slab) = 6.827E+11\]

\[I_y(comp) = 7.9244E+11\]

**Ix**

\[\text{Tav} = 203.3 \text{ mm}\]

\[\text{Bav} = 366.2 \text{ mm}\]

<table>
<thead>
<tr>
<th>Portion no</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>(3\cdot b \cdot d^3 / 10(b^2+d^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3200</td>
<td>250</td>
<td>8.33E-09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>203</td>
<td>3.07E-09</td>
</tr>
<tr>
<td>3</td>
<td>600</td>
<td>1530</td>
<td>8.60E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>366.2</td>
<td>1.30E+10</td>
</tr>
</tbody>
</table>

\[I_x (\text{mm}^3) = 1.10E+11\]

for deck slab \(I_x = b \cdot d^3 / 6\)
# SECTION PROPERTIES

## GIRDER (INNER)
### AT MID SPAN

<table>
<thead>
<tr>
<th>Self</th>
<th>2100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td>2100</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>1106250</td>
</tr>
<tr>
<td>Iy (mm⁴)</td>
<td>6.23E+11</td>
</tr>
<tr>
<td>Yy (mm)</td>
<td>1008.95</td>
</tr>
<tr>
<td>Zx (mm²)</td>
<td>6.173E+08</td>
</tr>
<tr>
<td>Zt (mm²)</td>
<td>5.709E+08</td>
</tr>
<tr>
<td>Concrete</td>
<td>M-40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composite</th>
<th>2350</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td>2350</td>
</tr>
<tr>
<td>Deck slab</td>
<td>250</td>
</tr>
<tr>
<td>Iy (mm⁴)</td>
<td>1.26046E+12</td>
</tr>
<tr>
<td>+ mm</td>
<td>1480.2</td>
</tr>
<tr>
<td>thick and</td>
<td>869.8</td>
</tr>
<tr>
<td>2800</td>
<td>1.449E+09</td>
</tr>
<tr>
<td>mm wide.</td>
<td>Zz (mm²)</td>
</tr>
<tr>
<td>Concrete</td>
<td>M-40</td>
</tr>
</tbody>
</table>

### Iy

<table>
<thead>
<tr>
<th>Element</th>
<th>Self = bd²/12 mm⁴</th>
<th>Aiy² mm⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.441E+10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>1.030E+09</td>
<td>7.019E+09</td>
</tr>
<tr>
<td>3</td>
<td>2.860E+09</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>6.351E+08</td>
<td>5.805E+09</td>
</tr>
<tr>
<td>5</td>
<td>2.500E+10</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>5.394E+10</td>
<td>1.282E+10</td>
</tr>
</tbody>
</table>

\[ Iy(\text{self}) = 6.676E+10 \text{ mm}^4 \]

\[ zL = 1400.0 \]

\[ Iy(\text{slab}) = 4.573E+11 \]

\[ Iy(\text{comp}) = 5.241E+11 \]

### Ix

\[ Tav = 230.0 \text{ mm} \]

\[ Bav = 420.0 \text{ mm} \]

<table>
<thead>
<tr>
<th>Portion no.</th>
<th>b</th>
<th>d</th>
<th>3.5 b² d²</th>
<th>10(b²+d²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2800</td>
<td>250</td>
<td>7.29E+09</td>
<td>4.19E+09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>230</td>
<td>4.41E+09</td>
<td>8.73E+09</td>
</tr>
<tr>
<td>3</td>
<td>275</td>
<td>1450</td>
<td>8.73E+09</td>
<td>1.89E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>420</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ Ix (\text{mm}^4) = 3.93E+10 \]

for deck slab \( Ix = b \times d^3 / 6 \)
**AT SUPPORT**

### Iz

<table>
<thead>
<tr>
<th>Element</th>
<th>Self = (bd^3/12)</th>
<th>(A_z^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.441E+10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2.034E+08</td>
<td>5.780E+09</td>
</tr>
<tr>
<td>3</td>
<td>2.970E+10</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>5.885E+07</td>
<td>3.560E+09</td>
</tr>
<tr>
<td>5</td>
<td>2.500E+10</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>7.938E+10</td>
<td>9.340E+09</td>
</tr>
</tbody>
</table>

\(I_z^{(self)} = 8.872E+10\) mm\(^4\)

\(z_L = 1400.0\)

\(I_z^{(slab)} = 4.573E+11\)

\(I_z^{(comp)} = 5.4605E+11\)

### Ly

\(Tav = 203.3\) mm

\(Bav = 366.2\) mm

<table>
<thead>
<tr>
<th>Portion no</th>
<th>b</th>
<th>d</th>
<th>(3.\cdot b^2d^3/10(b^2+d^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2800</td>
<td>250</td>
<td>7.29E+09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>203</td>
<td>3.07E+09</td>
</tr>
<tr>
<td>3</td>
<td>600</td>
<td>1530</td>
<td>8.60E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>366.206897</td>
<td>1.30E+10</td>
</tr>
</tbody>
</table>

\(I_y^{(mm^4)} = 1.09E+11\)

for deck slab \(I_x = b \cdot d^3 / 6\)
Cross member At End Supp.

Area (Ax) = 2955 x 250 + 1800 x 450
= 1548750 mm$^2$

$y_b = 1389$ mm
$y_t = 661$ mm

\[
\begin{array}{c|c|c}
\text{Element} & b & d \\
\hline
1 & 2955 & 250 \\
2 & 450 & 1800 \\
\end{array}
\]

\[
\begin{array}{c|c|c}
\text{Element} & b & d \\
\hline
1 & 2955 & 250 \\
2 & 450 & 1800 \\
\end{array}
\]

$I_x = 5.401E+10$ mm$^4$

$I_y = 6.123E+11$ mm$^4$

$I_z = 6.285E+11$ mm$^4$

$y_l = 1270$ mm

$y_r = 1685$ mm
Cross member At Intermediate Supp.

Area (Ax) = 3750 x 250 + 1800 x 300
= 1477500 mm²

\[ y_b = \frac{1550 \text{ mm}}{1875 \text{ mm}} \]

\[ y_t = \frac{500 \text{ mm}}{1875 \text{ mm}} \]

<table>
<thead>
<tr>
<th>Element</th>
<th>I self = bd^3/12 mm⁴</th>
<th>A x mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.883E+09</td>
<td>1.3157E+11</td>
</tr>
<tr>
<td>2</td>
<td>1.458E+11</td>
<td>2.2842E+11</td>
</tr>
<tr>
<td></td>
<td>1.507E+11 3.5999E+11</td>
<td></td>
</tr>
</tbody>
</table>

\[ I_z = 5.107E+11 \text{ mm}^4 \]

\[ y_i = 1875 \text{ mm} \]

\[ y_r = 1875 \text{ mm} \]

<table>
<thead>
<tr>
<th>Element</th>
<th>I self = bd^3/12 mm⁴</th>
<th>A y mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.099E+12</td>
<td>0.00E+00</td>
</tr>
<tr>
<td>2</td>
<td>4.050E+09</td>
<td>0.00E+00</td>
</tr>
<tr>
<td></td>
<td>1.103E+12 0.00E+00</td>
<td></td>
</tr>
</tbody>
</table>

\[ I_y = 1.103E+12 \text{ mm}^4 \]

<table>
<thead>
<tr>
<th>Element</th>
<th>b</th>
<th>d</th>
<th>3b²d²</th>
<th>10*(b²+d²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3750</td>
<td>250</td>
<td>9.766E+09</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>300</td>
<td>1800</td>
<td>1.419E+10</td>
<td></td>
</tr>
</tbody>
</table>

\[ I_x = 2.395E+10 \text{ mm}^4 \]
**GIRDER (inner)**

**AT 1/8 TH SPAN**

<table>
<thead>
<tr>
<th>Self</th>
<th>Depth (mm)</th>
<th>Area (mm$^2$)</th>
<th>$I_y$ (mm$^4$)</th>
<th>$Y_y$ (mm)</th>
<th>$Z_y$ (mm$^3$)</th>
<th>$Z_t$ (mm$^3$)</th>
<th>Concrete M-40</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2100</td>
<td>1315912.36</td>
<td>6.559E+11</td>
<td>1032.73</td>
<td>6.351E+08</td>
<td>6.145E+08</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Composite</th>
<th>Depth (mm)</th>
<th>Deck slab Area (mm$^2$)</th>
<th>$I_y$ (mm$^4$)</th>
<th>$Y_y$ (mm)</th>
<th>$Z_y$ (mm$^3$)</th>
<th>mm thick and $Y_y$ (mm)</th>
<th>mm wide, $Z_y$ (mm$^3$)</th>
<th>Concrete M-40</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2350</td>
<td>2015912</td>
<td>1.30906E+12</td>
<td>1446.7</td>
<td>903.3</td>
<td>1.449E+09</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$I_y = \frac{bd^3}{12}$

<table>
<thead>
<tr>
<th>Element</th>
<th>$I_y$ (mm$^4$)</th>
<th>$A_z$ (mm$^2$)</th>
<th>$z_L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.441E+10</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>4.966E+08</td>
<td>6.794E+09</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.151E+10</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2.301E+08</td>
<td>5.114E+09</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2.500E+10</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

$\frac{A_z}{I_y} = \frac{1.191E+10}{7.356E+10} = 0.161$ (not drawn)

$I_y$ (slab) = 4.573E+11

$I_y$ (comp) = 5.309E+11

$Tav = 216.7$ mm

$Bav = 393.1$ mm

<table>
<thead>
<tr>
<th>Portion no.</th>
<th>b</th>
<th>d</th>
<th>$3b^2d^3$</th>
<th>$10(b^3d^3)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2800</td>
<td>250</td>
<td>7.29E+09</td>
<td>4.46E+10</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>217</td>
<td>3.70E+09</td>
<td>2.21E+10</td>
</tr>
<tr>
<td>3</td>
<td>438</td>
<td>1490</td>
<td>3.45E+10</td>
<td>2.11E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>393.1</td>
<td>1.58E+10</td>
<td>9.46E+09</td>
</tr>
</tbody>
</table>

$5.395E+10$ for Girder only

$I_y = b \times d^3 / 6$
GIRDER (Outer)
AT 1/8 TH SPAN

<table>
<thead>
<tr>
<th>Self</th>
<th></th>
<th>Composite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td>2100</td>
<td>2350</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>1315912.36</td>
<td>Deck slab Area (mm²)</td>
</tr>
<tr>
<td>I₂ (mm⁴)</td>
<td>6.56E+11</td>
<td>2115912</td>
</tr>
<tr>
<td>Y₂ (mm)</td>
<td>1032.73</td>
<td>I₂ (mm⁴)</td>
</tr>
<tr>
<td>Y₁ (mm)</td>
<td>1067.27</td>
<td>1.36729E+12</td>
</tr>
<tr>
<td>Z₀ (mm³)</td>
<td>6.351E+08</td>
<td>thickness and Y₁(mm)</td>
</tr>
<tr>
<td>Z₁ (mm³)</td>
<td>6.145E+08</td>
<td>866.5</td>
</tr>
<tr>
<td>Concrete</td>
<td>M- 40</td>
<td>mm wide. Z₀(mm³)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9.217E+08</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete M- 40</td>
</tr>
</tbody>
</table>

Iᵧ

<table>
<thead>
<tr>
<th>Element</th>
<th>Iᵧ(self) = bd³/12</th>
<th>A₂ ³</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.441E+10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4.966E+08</td>
<td>6.794E+09</td>
</tr>
<tr>
<td>3</td>
<td>1.151E+10</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>2.301E+08</td>
<td>5.114E+09</td>
</tr>
<tr>
<td>5</td>
<td>2.500E+10</td>
<td>0</td>
</tr>
</tbody>
</table>

6.166E+10 1.191E+10

Iᵧ(self) = 7.356E+10 mm⁴

zL = 1724.4

Iᵧ(slab) = 6.827E+11

Iᵧ(comp) = 7.7613E+11

Ix

Tav = 216.7 mm

Bav = 393.1 mm

<table>
<thead>
<tr>
<th>Portion no</th>
<th>b</th>
<th>d</th>
<th>3b²d² / 10(b²+d²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3200</td>
<td>250</td>
<td>8.33E+09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>217</td>
<td>3.70E+09</td>
</tr>
<tr>
<td>3</td>
<td>438</td>
<td>1490</td>
<td>3.45E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>393.103448</td>
<td>1.58E+10</td>
</tr>
</tbody>
</table>

Iₓ (mm⁴) = 6.23E+10

for deck slab Iₓ = b x d³ / 6
GIRDER (Inner)

at effective depth d

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Area (mm$^2$)</th>
<th>$I_y$ (mm$^4$)</th>
<th>$Y_y$ (mm)</th>
<th>$Z_y$ (mm$^3$)</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>2100</td>
<td>1538649.43</td>
<td>6.932E+11</td>
<td>1048.92</td>
<td>6.608E+08</td>
<td>M-40</td>
</tr>
<tr>
<td>2350</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$I_y$ (Self) = 8.872E+10 mm$^4$

$z_y = 1400.0$

$I_y$ (slab) = 4.573E+11

$I_y$ (comp) = 5.4605E+11

<table>
<thead>
<tr>
<th>Portion no.</th>
<th>b (mm)</th>
<th>d (mm)</th>
<th>$3b^3d^3 / 10(b^2-d^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2800</td>
<td>250</td>
<td>7.29E+09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>203</td>
<td>3.07E+09</td>
</tr>
<tr>
<td>3</td>
<td>600</td>
<td>1530</td>
<td>8.60E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>366.206897</td>
<td>1.30E+10</td>
</tr>
</tbody>
</table>

$Tav = 203.3$ mm

$Bav = 366.2$ mm

$I_x$ (girder only) = $1.020$E+11

$I_x$ (deck slab) = $1.093$E+11

for deck slab $I_x = b \times d^3 / 6$
GIRDER (Outer)
AT effective depth d

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>2100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (mm²)</td>
<td>1538649.43</td>
</tr>
<tr>
<td>Iₓ (mm⁴)</td>
<td>6.93E+11</td>
</tr>
<tr>
<td>Yᵧ (mm)</td>
<td>1048.92</td>
</tr>
<tr>
<td>Zᵧ (mm³)</td>
<td>6.608E+08</td>
</tr>
<tr>
<td>Z (mm³)</td>
<td>6.595E+08</td>
</tr>
<tr>
<td>Concrete</td>
<td>M· 40</td>
</tr>
</tbody>
</table>

Composite

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>2350</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck slab</td>
<td>2338649</td>
</tr>
<tr>
<td>Area (mm²)</td>
<td>1.42534E+12</td>
</tr>
<tr>
<td>+ mm</td>
<td>Yᵧ (mm)</td>
</tr>
<tr>
<td>250</td>
<td>1451.2</td>
</tr>
<tr>
<td>mm thick and Yᵧ (mm)</td>
<td>898.8</td>
</tr>
<tr>
<td>3200</td>
<td>Zᵧ (mm³)</td>
</tr>
<tr>
<td>mm wide.</td>
<td>Z (mm³)</td>
</tr>
<tr>
<td>1.586E+09</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>M· 40</td>
</tr>
</tbody>
</table>

| Iᵧ (mm⁴) | 8.872E+10 |
| zL | 1731.6 |
| Iᵧ (slab) | 6.827E+11 |
| Iᵧ (comp) | 7.9244E+11 |

| Tav | 203.3 mm |
| Bav | 366.2 mm |

<table>
<thead>
<tr>
<th>Portion no.</th>
<th>b</th>
<th>d</th>
<th>3(b²d²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm</td>
<td>mm</td>
<td>10(b²d²)</td>
</tr>
<tr>
<td>1</td>
<td>3200</td>
<td>250</td>
<td>8.33E+09</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>203</td>
<td>3.07E+09</td>
</tr>
<tr>
<td>3</td>
<td>600</td>
<td>1530</td>
<td>8.60E+10</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>366.206897</td>
<td>1.30E+10</td>
</tr>
</tbody>
</table>

Iₓ (mm⁴) = 1.10E+11

for deck slab Iₓ = b x d³ / 6
### Bridge Design Report

**FOR MEMBERS 107 TO 112 116 TO 122 126 TO 132 136 TO 141**

<table>
<thead>
<tr>
<th>Area $A_x$</th>
<th>$I_x$</th>
<th>$I_y$</th>
<th>$I_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$3.75 \times 0.25 = 0.9375 \text{ m}^2$</td>
<td>$3.75 \times 0.25^3 / 6 = 0.009766 \text{ m}^4$</td>
<td>$0.25 \times 3.75^3 / 12 = 1.098633 \text{ m}^4$</td>
<td>$3.75 \times 0.25^3 / 12 = 0.004883 \text{ m}^4$</td>
</tr>
</tbody>
</table>

**FOR MEMBERS 102 106 142 146**

<table>
<thead>
<tr>
<th>Area $A_x$</th>
<th>$I_x$</th>
<th>$I_y$</th>
<th>$I_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$2.955 \times 0.25 = 0.73875 \text{ m}^2$</td>
<td>$2.955 \times 0.25^3 / 6 = 0.007695 \text{ m}^4$</td>
<td>$0.25 \times 2.955^3 / 12 = 0.537565 \text{ m}^4$</td>
<td>$2.955 \times 0.25^3 / 12 = 0.003848 \text{ m}^4$</td>
</tr>
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</table>

### Summary of member Properties

<table>
<thead>
<tr>
<th>Member No.</th>
<th>Due to Composite</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 to 16, 81 to 96</td>
<td>0.001 0.0001 0.0001</td>
</tr>
<tr>
<td>22 to 27 70 to 75</td>
<td>1.906 0.0404 1.3135</td>
</tr>
<tr>
<td>38 to 42 54 to 59</td>
<td>1.806 0.0393 1.2605</td>
</tr>
<tr>
<td>17 31 65 79</td>
<td>2.339 0.1104 1.4253</td>
</tr>
<tr>
<td>18 32 66 80</td>
<td>2.339 0.1104 1.4253</td>
</tr>
<tr>
<td>33 34 47 to 50 63 64</td>
<td>2.239 0.1093 1.3623</td>
</tr>
<tr>
<td>19 TO 21 28 TO 30 67 TO 69 76 TO 78</td>
<td>2.122 0.0754 1.3694</td>
</tr>
<tr>
<td>35 TO 37 44 TO 46 51 TO 53 60 TO 62</td>
<td>2.022 0.0743 1.3114</td>
</tr>
<tr>
<td>103 to 105 143 to 145</td>
<td>1.549 0.0540 0.6285</td>
</tr>
<tr>
<td>123 to 125 113 to 115 133 to 135</td>
<td>1.478 0.0240 0.5107</td>
</tr>
<tr>
<td>107 to 112 116 to 122 126 to 132 136 to 141</td>
<td>0.938 0.0098 0.0049</td>
</tr>
<tr>
<td>97 to 101 147 to 151</td>
<td>0.001 0.0001 0.0001</td>
</tr>
<tr>
<td>102 106 142 146</td>
<td>0.739 0.0077 0.0038</td>
</tr>
</tbody>
</table>
### SUMMARY OF BENDING MOMENT AT MID SPAN

<table>
<thead>
<tr>
<th>Loads</th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
<th>G4</th>
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</thead>
<tbody>
<tr>
<td>Self Weight of girder</td>
<td>316.27</td>
<td>316.27</td>
<td>316.27</td>
<td>316.27</td>
</tr>
<tr>
<td>Weight of deck slab</td>
<td>223.83</td>
<td>223.83</td>
<td>223.83</td>
<td>223.83</td>
</tr>
<tr>
<td>Weight of the shuttering</td>
<td>55.96</td>
<td>55.96</td>
<td>55.96</td>
<td>55.96</td>
</tr>
<tr>
<td>Weight of Diaphragm</td>
<td>44.63</td>
<td>44.63</td>
<td>44.63</td>
<td>44.63</td>
</tr>
<tr>
<td>SIDL</td>
<td>163.3</td>
<td>164.96</td>
<td>164.96</td>
<td>163.3</td>
</tr>
<tr>
<td>LIVE LOAD (Max)</td>
<td>238.92</td>
<td>177.7</td>
<td>146.88</td>
<td>123.46</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>1042.91</strong></td>
<td><strong>983.35</strong></td>
<td><strong>952.53</strong></td>
<td><strong>927.45</strong></td>
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</tbody>
</table>

Hence Girder G-1 shall be designed and rest of the girders will be kept same as that of G-1

### SUMMARY OF BENDING MOMENT & SHEAR FORCES AT DIFFERENT SECTIONS OF G-1

<table>
<thead>
<tr>
<th>Section</th>
<th>DL OF GIRDER</th>
<th>DL OF DECK SLAB</th>
<th>SHUTTERING</th>
<th>WT. OF DIAPHRAGM</th>
<th>SIDL</th>
<th>LIVE LOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BM (t.m.)</td>
<td>SF (t)</td>
<td>BM (t.m.)</td>
<td>SF (t)</td>
<td>BM (t.m.)</td>
<td>SF (t)</td>
</tr>
<tr>
<td></td>
<td>SF (t)</td>
<td>BM (t.m.)</td>
<td>SF (t)</td>
<td>BM (t.m.)</td>
<td>SF (t)</td>
<td>BM (t.m.)</td>
</tr>
<tr>
<td>1 - 1</td>
<td>316.27</td>
<td>0.00</td>
<td>223.83</td>
<td>0</td>
<td>55.96</td>
<td>44.63</td>
</tr>
<tr>
<td>2 - 2</td>
<td>296.83</td>
<td>10.37</td>
<td>209.77</td>
<td>7.5</td>
<td>52.44</td>
<td>39.05</td>
</tr>
<tr>
<td>3 - 3</td>
<td>238.50</td>
<td>20.74</td>
<td>167.58</td>
<td>15</td>
<td>41.9</td>
<td>33.47</td>
</tr>
<tr>
<td>4 - 4</td>
<td>141.29</td>
<td>31.11</td>
<td>97.27</td>
<td>22.5</td>
<td>24.32</td>
<td>16.74</td>
</tr>
<tr>
<td>5 - 5</td>
<td>92.26</td>
<td>36.06</td>
<td>62.24</td>
<td>25.42</td>
<td>15.56</td>
<td>10.21</td>
</tr>
</tbody>
</table>

**Summary**

- **Self Weight of girder**: The self weight of girder ranges from 316.27 to 316.27 kN for different sections.
- **Weight of deck slab**: The weight of the deck slab ranges from 223.83 to 223.83 kN for different sections.
- **Weight of the shuttering**: The weight of the shuttering ranges from 55.96 to 55.96 kN for different sections.
- **Weight of Diaphragm**: The weight of the diaphragm ranges from 44.63 to 44.63 kN for different sections.
- **SIDL**: The SIDL value ranges from 163.3 to 164.96 kN for different sections.
- **LIVE LOAD (Max)**: The maximum live load ranges from 123.46 to 238.92 kN for different sections.

**Total**

The total bending moment ranges from 1042.91 to 927.45 kN-t.m. for different sections.
CALCULATION FOR FRICCTION AND SLIP LOSSES

The cable is assumed to have a straight portion near ends followed by a parabolic profile. The parabolic profile is followed by plan bending. Cable profile for plan bending is assumed to be made up of two parabola. Near midspan it has a small horizontal.

No of Prestressing Cables = 7
Type of cable = 12-T-13
Type of Sheathing = Bright Metal
X-sectional area of each cable, \( A_s = 11.844 \text{ cm}^2 \)
Modulus of elasticity of prestressing steel, \( E_s = 1.988 \times 10^7 \text{ t/m}^2 \)
Span length bearing to bearing = 30.000 m
Wobble coefficient \( k = 0.0046 \)
Friction coefficient \( \mu = 0.2500 \)
Expected slip, \( s = 6.0 \text{ mm} \)
Half slip area (= 0.5 x \( A_s \) x \( E_s \) x \( s \)) = 70.64 \text{ tm}
Grip length inside the jack = 0.350 m
Extra length of cable required for jack attachment = 0.750 m
UTS of Cable = 224.71 t
Dia of Duct = 75 mm

CABLE PROFILE

<table>
<thead>
<tr>
<th>CABLE</th>
<th>a = Hor. Length of inclined Portion in m.</th>
<th>b = Hor. Length of parabolic Portion in m.</th>
<th>c = Hor. Length of plan bend. Portion in m.</th>
<th>d = Hor. Length of straight Portion in m.</th>
<th>( H = Y ) co-ordinate At jacking end</th>
<th>( h_3 = Y ) co-ordinate At mid span</th>
<th>( m = ) plan bending in m.</th>
<th>Jacking force (t)</th>
<th>Stressing stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.000</td>
<td>14.250</td>
<td>0.000</td>
<td>0.000</td>
<td>0.300</td>
<td>0.115</td>
<td>0.000</td>
<td>157.30</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>1.000</td>
<td>14.250</td>
<td>0.000</td>
<td>0.000</td>
<td>0.300</td>
<td>0.115</td>
<td>0.000</td>
<td>157.30</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>1.004</td>
<td>8.250</td>
<td>6.000</td>
<td>6.000</td>
<td>0.900</td>
<td>0.265</td>
<td>0.000</td>
<td>157.30</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>1.000</td>
<td>8.250</td>
<td>0.000</td>
<td>6.000</td>
<td>0.900</td>
<td>0.265</td>
<td>0.000</td>
<td>157.30</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>1.004</td>
<td>14.250</td>
<td>0.000</td>
<td>0.000</td>
<td>1.200</td>
<td>0.415</td>
<td>0.000</td>
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<td>2</td>
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<tr>
<td>6</td>
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<td>14.250</td>
<td>0.000</td>
<td>0.000</td>
<td>1.500</td>
<td>0.565</td>
<td>0.000</td>
<td>157.30</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>1.000</td>
<td>14.250</td>
<td>0.000</td>
<td>0.000</td>
<td>1.800</td>
<td>0.715</td>
<td>0.000</td>
<td>157.30</td>
<td>1</td>
</tr>
</tbody>
</table>

CALCULATION FOR LENGTHS AND CUMULATIVE ANGLES

<table>
<thead>
<tr>
<th>CABLE</th>
<th>A to B</th>
<th>A to C</th>
<th>A to D</th>
<th>A to E</th>
<th>A to F</th>
<th>Length* of cable</th>
<th>( \vartheta_c )</th>
<th>( \vartheta_a )</th>
<th>( \vartheta_a+\vartheta_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(rad)</td>
<td>(rad)</td>
<td>(rad)</td>
<td>(rad)</td>
</tr>
<tr>
<td>1</td>
<td>1.000</td>
<td>15.251</td>
<td>15.251</td>
<td>15.251</td>
<td>15.251</td>
<td>32.003</td>
<td>0.0228</td>
<td>0.0228</td>
<td>0.0228</td>
</tr>
<tr>
<td>2</td>
<td>1.000</td>
<td>15.251</td>
<td>15.251</td>
<td>15.251</td>
<td>15.251</td>
<td>32.003</td>
<td>0.0228</td>
<td>0.0228</td>
<td>0.0228</td>
</tr>
<tr>
<td>3</td>
<td>1.004</td>
<td>9.267</td>
<td>9.267</td>
<td>9.267</td>
<td>9.267</td>
<td>32.034</td>
<td>0.0944</td>
<td>0.0944</td>
<td>0.0944</td>
</tr>
<tr>
<td>4</td>
<td>1.008</td>
<td>9.279</td>
<td>9.279</td>
<td>9.279</td>
<td>9.279</td>
<td>32.058</td>
<td>0.1233</td>
<td>0.1233</td>
<td>0.1233</td>
</tr>
<tr>
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<td>1.005</td>
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<td>15.277</td>
<td>15.277</td>
<td>15.277</td>
<td>32.054</td>
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<td>0.0963</td>
<td>0.0963</td>
</tr>
<tr>
<td>6</td>
<td>1.007</td>
<td>15.288</td>
<td>15.288</td>
<td>15.288</td>
<td>15.288</td>
<td>32.076</td>
<td>0.1146</td>
<td>0.1146</td>
<td>0.1146</td>
</tr>
<tr>
<td>7</td>
<td>1.009</td>
<td>15.301</td>
<td>15.301</td>
<td>15.301</td>
<td>15.301</td>
<td>32.102</td>
<td>0.1328</td>
<td>0.1328</td>
<td>0.1328</td>
</tr>
</tbody>
</table>

* Total length of cable includes 750mm extra gripping length required for jack attachment.
\( \vartheta \) is total angular change w.r.t. node A.
### FORCES IN CABLES AFTER FRICTION ONLY

<table>
<thead>
<tr>
<th>CABLE</th>
<th>k</th>
<th>$F_A$ (t)</th>
<th>$F_B$ (t)</th>
<th>$F_C$ (t)</th>
<th>$F_D$ (t)</th>
<th>$F_E$ (t)</th>
<th>$F_F$ (t)</th>
<th>Average Force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.99E-04</td>
<td>157.30</td>
<td>156.57</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>151.57</td>
</tr>
<tr>
<td>2</td>
<td>7.99E-04</td>
<td>157.30</td>
<td>156.57</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>151.57</td>
</tr>
<tr>
<td>3</td>
<td>5.74E-03</td>
<td>157.30</td>
<td>156.57</td>
<td>147.22</td>
<td>147.22</td>
<td>147.22</td>
<td>147.22</td>
<td>149.60</td>
</tr>
<tr>
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<td>7.51E-03</td>
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<td>156.57</td>
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<td>146.15</td>
<td>146.15</td>
<td>146.15</td>
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</tr>
<tr>
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<td>3.99E-03</td>
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<td>156.57</td>
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<td>143.13</td>
<td>143.13</td>
<td>143.13</td>
<td>150.32</td>
</tr>
<tr>
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<td>4.04E-03</td>
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<td>156.57</td>
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<td>141.82</td>
<td>141.82</td>
<td>149.70</td>
</tr>
</tbody>
</table>

$k$ is coefficient of parabolic equation $y = k x^2$ (in vertical plane)

### SLIP DISTANCE CALCULATION

<table>
<thead>
<tr>
<th>CABLE</th>
<th>$d$ (t)</th>
<th>$F_A$ (t)</th>
<th>$F_B$ (t)</th>
<th>$F_C$ (t)</th>
<th>$F_D$ (t)</th>
<th>$F_E$ (t)</th>
<th>$F_F$ (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-70.28</td>
<td>17.22</td>
<td>17.22</td>
<td>17.22</td>
<td>156.57</td>
<td>145.81</td>
<td>14.25</td>
</tr>
<tr>
<td>2</td>
<td>-70.28</td>
<td>17.22</td>
<td>17.22</td>
<td>17.22</td>
<td>156.57</td>
<td>145.81</td>
<td>14.25</td>
</tr>
<tr>
<td>3</td>
<td>-70.27</td>
<td>-22.24</td>
<td>-22.24</td>
<td>-22.24</td>
<td>26.92</td>
<td>147.22</td>
<td>14.27</td>
</tr>
<tr>
<td>4</td>
<td>-70.27</td>
<td>-16.68</td>
<td>-16.68</td>
<td>-16.68</td>
<td>32.17</td>
<td>146.15</td>
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<td>39.12</td>
<td>39.12</td>
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</tr>
<tr>
<td>6</td>
<td>-70.27</td>
<td>44.57</td>
<td>44.57</td>
<td>44.57</td>
<td>156.57</td>
<td>142.47</td>
<td>14.27</td>
</tr>
<tr>
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<td>50.00</td>
<td>50.00</td>
<td>156.57</td>
<td>141.82</td>
<td>14.27</td>
</tr>
</tbody>
</table>

### FORCES IN CABLES AFTER SLIP

<table>
<thead>
<tr>
<th>Cable</th>
<th>Slip distance</th>
<th>$F_A$ (t)</th>
<th>$F_B$ (t)</th>
<th>$F_C$ (t)</th>
<th>$F_D$ (t)</th>
<th>$F_E$ (t)</th>
<th>$F_F$ (t)</th>
<th>Avg. Force (t)</th>
<th>Elongation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13.676</td>
<td>136.70</td>
<td>137.42</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>141.32</td>
<td>100.51</td>
</tr>
<tr>
<td>2</td>
<td>13.676</td>
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<td>137.42</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>141.32</td>
<td>100.51</td>
</tr>
<tr>
<td>3</td>
<td>12.348</td>
<td>133.75</td>
<td>133.75</td>
<td>143.10</td>
<td>143.10</td>
<td>143.10</td>
<td>143.10</td>
<td>139.95</td>
<td>99.34</td>
</tr>
<tr>
<td>4</td>
<td>11.680</td>
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<td>132.55</td>
<td>142.97</td>
<td>142.97</td>
<td>142.97</td>
<td>142.97</td>
<td>139.28</td>
<td>98.96</td>
</tr>
<tr>
<td>5</td>
<td>12.259</td>
<td>134.65</td>
<td>135.38</td>
<td>143.13</td>
<td>143.13</td>
<td>143.13</td>
<td>143.13</td>
<td>138.98</td>
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</tr>
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<td>142.47</td>
<td>142.47</td>
<td>142.47</td>
<td>142.47</td>
<td>138.42</td>
<td>99.74</td>
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<tr>
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<td>11.714</td>
<td>133.75</td>
<td>134.48</td>
<td>141.82</td>
<td>141.82</td>
<td>141.82</td>
<td>141.82</td>
<td>137.88</td>
<td>99.62</td>
</tr>
</tbody>
</table>

### FORCES IN CABLES AT SECTION 1-1 (MIDSPAN) AFTER SLIP

<table>
<thead>
<tr>
<th>Cable no</th>
<th>Stage</th>
<th>Length of cable from midspan</th>
<th>$\phi$ (rad)</th>
<th>$\varphi$ (rad)</th>
<th>Co-ord from bottom</th>
<th>$P$ (t)</th>
<th>$P_{cos}\phi$ (t)</th>
<th>$P_{sin}\varphi$ (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.115</td>
<td>145.81</td>
<td>145.81</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.115</td>
<td>145.81</td>
<td>145.81</td>
<td>0.00</td>
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<td>0.115</td>
<td>143.21</td>
<td>143.21</td>
<td>0.00</td>
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<tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.265</td>
<td>142.17</td>
<td>142.17</td>
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<td>0.00</td>
<td>0.565</td>
<td>142.47</td>
<td>142.47</td>
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<td>0.00</td>
<td>0.715</td>
<td>141.82</td>
<td>141.82</td>
<td>0.00</td>
</tr>
</tbody>
</table>

$\phi$ is angle of cable in vertical plane w.r.t. soffit slab

$\varphi$ is angle of cable in horizontal plane

Stage 1
- Total horizontal force = 718.8 t
- Ecc wrt bottom = 0.263 m
- Total vertical force = 0.00 t

Stage 2
- Total horizontal force = 285.6 t
- Ecc wrt bottom = 0.490 m
- Total vertical force = 0.00 t
### FORCES IN CABLES AT SECTION 2-2 (THREE EIGHTH SECTION) AFTER SLIP
3.750 m FROM MIDSPAN

| Cable no | Stage | Length from midspan | $\phi$ (rad) | $\theta$ (rad) | Co-ord from bottom | P (t) | Pcos$\phi$ (t) | Psin$\theta$ (t) |
|----------|-------|---------------------|--------------|--------------|-------------------|-------|--------------|----------------|-------|
| 1        | 1     | 3.750               | 0.006        | 0.000        | 0.126             | 145.43| 145.43       | 0.87            |       |
| 2        | 1     | 3.750               | 0.006        | 0.000        | 0.126             | 145.43| 145.43       | 0.87            |       |
| 3        | 1     | 3.750               | 0.000        | 0.000        | 0.115             | 144.62| 144.62       | 0.00            |       |
| 4        | 1     | 3.750               | 0.000        | 0.000        | 0.265             | 144.47| 144.47       | 0.00            |       |
| 5        | 2     | 3.750               | 0.025        | 0.000        | 0.463             | 145.40| 145.35       | 3.70            |       |
| 6        | 2     | 3.751               | 0.030        | 0.000        | 0.622             | 145.43| 145.37       | 4.40            |       |
| 7        | 1     | 3.751               | 0.035        | 0.000        | 0.781             | 145.48| 145.39       | 5.11            |       |

Stage 1
- Total horizontal force = 725.3 t
- Ecc wrt bottom = 0.283 m
- Total vertical force = 6.854 t

Stage 2
- Total horizontal force = 290.72 t
- Ecc wrt bottom = 0.542 m
- Total vertical force = 8.100 t

Stage 3
- Total horizontal force = 0.000 t
- Ecc wrt bottom = 0.000 m
- Total vertical force = 0.000 t

### FORCES IN CABLES AT SECTION 3-3 (ONE FOURTH SECTION) AFTER SLIP
7.500 m FROM MIDSPAN

| Cable | Stage | Length from midspan | $\phi$ (rad) | $\theta$ (rad) | Co-ord from bottom | P (t) | Pcos$\phi$ (t) | Psin$\theta$ (t) |
|-------|-------|---------------------|--------------|--------------|-------------------|-------|--------------|----------------|-------|
| 1     | 1     | 7.500               | 0.012        | 0.000        | 0.160             | 142.62| 142.61       | 1.71            |       |
| 2     | 1     | 7.500               | 0.012        | 0.000        | 0.160             | 142.62| 142.61       | 1.71            |       |
| 3     | 1     | 7.500               | 0.017        | 0.000        | 0.128             | 141.44| 141.42       | 2.43            |       |
| 4     | 1     | 7.500               | 0.023        | 0.000        | 0.282             | 141.12| 141.09       | 3.18            |       |
| 5     | 2     | 7.503               | 0.051        | 0.000        | 0.606             | 141.89| 141.71       | 7.21            |       |
| 6     | 2     | 7.505               | 0.061        | 0.000        | 0.792             | 141.76| 141.50       | 8.58            |       |
| 7     | 1     | 7.506               | 0.070        | 0.000        | 0.979             | 141.64| 141.29       | 9.95            |       |

Stage 1
- Total horizontal force = 709.0 t
- Ecc wrt bottom = 0.341 m
- Total vertical force = 18.977 t

Stage 2
- Total horizontal force = 283.2 t
- Ecc wrt bottom = 0.699 m
- Total vertical force = 15.79 t
**FORCES IN CABLES AT SECTION 4-4 (ONE EIGHTH SECTION) AFTER SLIP**

11.250 m FROM MIDSPAN

<table>
<thead>
<tr>
<th>Cable</th>
<th>Stage</th>
<th>Length from midspan</th>
<th>( \phi ) (rad)</th>
<th>( \varphi ) (rad)</th>
<th>Co-ord from bottom</th>
<th>P (t)</th>
<th>( P \cos \phi ) (t)</th>
<th>( P \sin \varphi ) (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>11.251</td>
<td>0.0180</td>
<td>0.000</td>
<td>0.216</td>
<td>139.75</td>
<td>139.729</td>
<td>2.512</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>11.251</td>
<td>0.0180</td>
<td>0.000</td>
<td>0.216</td>
<td>139.75</td>
<td>139.729</td>
<td>2.512</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>11.253</td>
<td>0.0602</td>
<td>0.000</td>
<td>0.273</td>
<td>137.21</td>
<td>136.965</td>
<td>8.258</td>
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<tr>
<td>4</td>
<td>1</td>
<td>11.255</td>
<td>0.0788</td>
<td>0.000</td>
<td>0.472</td>
<td>136.41</td>
<td>135.985</td>
<td>10.744</td>
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<td>0.0763</td>
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<td>0.844</td>
<td>138.30</td>
<td>137.902</td>
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<td>0.000</td>
<td>1.076</td>
<td>137.99</td>
<td>137.419</td>
<td>12.519</td>
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<td>11.271</td>
<td>0.1054</td>
<td>0.000</td>
<td>1.308</td>
<td>137.69</td>
<td>136.923</td>
<td>14.489</td>
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</tbody>
</table>

Stage 1  Total horizontal force = 689.3 t  
         ecc wrt bottom = 0.495 m  
         Total vertical force = 38.515 t

Stage 2  Total horizontal force = 275.32 t  
         ecc wrt bottom = 0.960 m  
         Total vertical force = 23.058 t

**FORCES IN CABLES AT SECTION 5-5 (Effective depth d) AFTER SLIP**

12.713 m FROM MIDSPAN

<table>
<thead>
<tr>
<th>Cable no</th>
<th>Stage</th>
<th>Length from midspan</th>
<th>( \phi ) (rad)</th>
<th>( \varphi ) (rad)</th>
<th>Co-ord from bottom</th>
<th>P (t)</th>
<th>( P \cos \phi ) (t)</th>
<th>( P \sin \varphi ) (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>12.713</td>
<td>0.020</td>
<td>0.000</td>
<td>0.244</td>
<td>138.62</td>
<td>138.59</td>
<td>2.82</td>
</tr>
<tr>
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<td>1</td>
<td>12.713</td>
<td>0.020</td>
<td>0.000</td>
<td>0.244</td>
<td>138.62</td>
<td>138.59</td>
<td>2.82</td>
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<tr>
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<td>12.719</td>
<td>0.077</td>
<td>0.000</td>
<td>0.373</td>
<td>135.53</td>
<td>135.13</td>
<td>10.43</td>
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<td>0.603</td>
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<td>133.84</td>
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<td>0.086</td>
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<td>0.963</td>
<td>136.88</td>
<td>136.37</td>
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<td>0.000</td>
<td>1.218</td>
<td>136.49</td>
<td>135.77</td>
<td>13.99</td>
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<td>1</td>
<td>12.743</td>
<td>0.119</td>
<td>0.000</td>
<td>1.472</td>
<td>136.12</td>
<td>135.15</td>
<td>16.18</td>
</tr>
</tbody>
</table>

Stage 1  Total horizontal force = 681.3 t  
         ecc wrt bottom = 0.584 m  
         Total vertical force = 45.772 t

Stage 2  Total horizontal force = 272.1 t  
         ecc wrt bottom = 1.090 m  
         Total vertical force = 25.77 t
STRESS CHECK AT VARIOUS SECTIONS

Type of cable = 12-T-13
Area of each cable = 1184.4 sq.mm
UTS of one cable = 224.7 t
Jacking stress = 70.0% of UTS
Jacking force = 157.29 t
Modulus of Elasticity of HTS, Es = 1.95E+05 MPa = 1.99E+07 t/m²
Grade of concrete = M - 40
\( f_{ck} \) = 40 N/mm²
Modulus of Elasticity of Concrete, Ec = 5700x(\( f_{ck} \))⁰.⁵ = 3.22E+04 MPa = 3.29E+06 t/m² (At 4th Day)
 = 3.60E+04 MPa = 3.68E+06 t/m² (At 28th Day)

The girders shall be cast on the ground near bridge site. The stressing shall be done when cube strength of concrete reached 32 MPa. Thereafter the girders will be launched / shifted to the bridge location and placed on permanent bearings and deck slab & diaphragm shall be casted.

\( f_{ij} \) at release = 32 MPa
Maturity of concrete at the time of release as % of \( f_{ck} \) = 80.00 %
\( E_c \) = 5700x(\( f_{ck} \))⁰.⁵ = 3.22E+04 MPa = 3.29E+06 t/m²

<table>
<thead>
<tr>
<th>Creep strain (per 10MPa) as per Table 2 of IRC:18 - 2000</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>From 14th day (80% maturity) to 28th day (100% maturity)</td>
<td></td>
</tr>
<tr>
<td>Creep strain for 80% maturity</td>
<td>0.000510</td>
</tr>
<tr>
<td>Creep strain for 100% maturity</td>
<td>0.000400</td>
</tr>
<tr>
<td>Differential Creep strain between 14th and 28th day</td>
<td>0.000110</td>
</tr>
<tr>
<td>Residual Creep strain 28th day onward</td>
<td>0.000400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Residual shrinkage strain as per Table 3 of IRC:18 - 2000</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Shrinkage strain (( \varepsilon_s )) at 14th day</td>
<td>0.00025</td>
</tr>
<tr>
<td>Residual Shrinkage strain (( \varepsilon_s )) at 28th day</td>
<td>0.00019</td>
</tr>
<tr>
<td>Therefore, Shrinkage strain (( \varepsilon_s )) between 14th &amp; 28th day</td>
<td>0.00006</td>
</tr>
<tr>
<td>Residual Shrinkage strain 28th day onward</td>
<td>0.00019</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loss due to Relaxation for low Relaxation steel as per Table 4A of IRC:18 - 2000</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>at average initial force of ( 0.5 ) ( f_p ) is</td>
<td>0 MPa</td>
</tr>
<tr>
<td>at average initial force of ( 0.7 ) ( f_p ) is</td>
<td>97 MPa</td>
</tr>
<tr>
<td>at average initial force of ( 0.8 ) ( f_p ) is</td>
<td>200 MPa</td>
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</table>
STRESS CHECK AT MID SPAN (SEC 1-1)

At first stage 4 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, \(eb\) = 0.263 m
Eccentricity of cables from c.g., \(e\) = 0.746 m
Sectional Modulus at C.G. of cables, \(Z_c\) = 0.835 \(m^3\)
(for precast girder only)

B.M. due to S.Wt. of the girder \(M_d\) = 316.27 tm
B.M. due to Cast in Situ slab \(M_c\) = 324.42 tm
Prestressing force (after friction and slip losses), \(P\) = 718.82 t
Moment due to \(P\), \(M_p\) = \(P \times e\) = 536.17 tm

Stresses due to self weight of the girder
\[\sigma_t = \frac{316.27}{0.571} = 554.02 \, t/m^2\]
\[\sigma_c = \frac{-378.77}{0.835} = -454.13 \, t/m^2\]
\[\sigma_b = \frac{-512.34}{0.617} = -833.12 \, t/m^2\]

Stresses due to prestress at transfer stage
\[\sigma_t = \frac{718.82}{1.106} - \frac{536.17}{0.571} = -289.5 \, t/m^2\]
\[\sigma_c = \frac{-378.8}{1.106} + \frac{1291.9}{0.835} = 1291.9 \, t/m^2\]
\[\sigma_b = \frac{-512.3}{1.106} + \frac{1518.3}{0.617} = 1518.3 \, t/m^2\]

Stresses due to initial prestress & self weight of the girder
\[\sigma_t = \frac{554.0}{1.106} + \frac{-289.5}{0.835} = 264.6 \, t/m^2\]
\[\sigma_c = \frac{-378.8}{1.106} + \frac{1291.9}{1.106} = 913.1 \, t/m^2\]
\[\sigma_b = \frac{-512.3}{1.106} + \frac{1518.3}{0.835} = 1006.0 \, t/m^2\]

Losses due to Elastic Shortening
\[\text{Loss due to Elastic Shortening} = 0.5 \times \sigma \times E_s / E_c\]

Therefore, initial loss = 2759 \(t/m^2\)
(i.e. Elastic Shortening loss) = 16.34 \(t\) = 2.08 \% of Jacking Force

Prestressing force after initial losses \(P_t\) = 702.48 t
Moment due to \(P_t\), \(M_{pt}\) = \(P_t \times e\) = 523.98 tm

Loss due to Shrinkage of Concrete
\[\text{Loss due to Shrinkage of Concrete} = \varepsilon_s \times E_s\]
\[= 1192.7 \, t/m^2\]
\[= 7.06 \, t\] = 0.90 \% of Jacking Force
Loss due to relaxation of steel
The loss due to relaxation of steel depends on the stress in HTS after initial losses.
Ratio of average force to UTS = 0.685
Loss due to friction & slip = 786.4653 - 718.8 = 67.6 t
% friction and slip loss = 67.6 x 100 = 6.021 %
Ratio of average force to UTS = 0.685 - 0.060 = 0.625
Corresponding Relaxation Loss = 6195.8 t/m²
= 36.69 t
4.67 % of Jacking Force
Loss due to Relaxation & Shrinkage = 43.75 t
Moment due to Relaxation & Shrinkage loss = 32.637 tm

Stresses due to Prestress after initial losses
\[ \sigma_i = \frac{702.48}{1.106} - \frac{523.98}{0.571} = -282.88 \text{ t/m}^2 \]
\[ \sigma_c = \frac{702.48}{1.106} + \frac{523.98}{0.835} = 1262.5 \text{ t/m}^2 \]
\[ \sigma_b = \frac{702.48}{1.106} + \frac{523.98}{0.617} = 1483.8 \text{ t/m}^2 \]

Stresses due to prestress after initial losses & self weight of the girder
\[ \sigma_i = \frac{554.0}{1.106} + \frac{-282.9}{0.571} = 271.1 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-378.8}{1.106} + \frac{1262.5}{0.835} = 883.8 \text{ t/m}^2 \]
\[ \sigma_b = \frac{-512.3}{1.106} + \frac{1483.8}{0.617} = 971.5 \text{ t/m}^2 \]

Stresses after Shrinkage and Relaxation losses
\[ \sigma_i = \frac{271.15}{1.106} - \frac{43.75}{0.571} + \frac{32.637}{0.835} = 288.77 \text{ t/m}^2 \]
\[ \sigma_c = \frac{883.8}{1.106} - \frac{43.75}{0.835} - \frac{32.637}{0.617} = 805.13 \text{ t/m}^2 \]
\[ \sigma_b = \frac{971.49}{1.106} - \frac{43.75}{0.617} = 879.07 \text{ t/m}^2 \]

Loss due to Creep of concrete
Arithmetic mean of initial and final stress between 14th and 28 days at C.G. of cables
Creep loss = \( \sigma / 10 \times \varepsilon_c \times E_s = 1810.2 \text{ t/m}^2 \)
= 10.72 t = 1.363 % of Jacking Force
Total Initial & Intermediate losses = 70.82 t = 9.00 % of Jacking Force (i.e. losses upto 28th day)
Force after intermediate losses, \( P_{int} = 648.0 \text{ t} \)
Moment due to \( P_{int} = \left( M_{pint} = P_{int} \times e = 483.35 \text{ tm} \)

Effect of prestress after Intermediate loss
\[ \sigma_i = \frac{648.0}{1.106} - \frac{483.35}{0.571} = -260.9 \text{ t/m}^2 \]
\[ \sigma_c = \frac{648.0}{1.106} + \frac{483.35}{0.835} = 1164.6 \text{ t/m}^2 \]
\[ \sigma_b = \frac{648.0}{1.106} + \frac{483.35}{0.617} = 1368.8 \text{ t/m}^2 \]

Stresses Due to cast-in-situ slab
\[ \sigma_i = \frac{324.4}{0.571} = 568.30 \text{ t/m}^2 \]
\[ \sigma_c = \frac{324.4}{0.835} = -388.53 \text{ t/m}^2 \]
\[ \sigma_b = \frac{324.4}{0.617} = -525.54 \text{ t/m}^2 \]
RESULTANT STRESSES

a) Transfer stage

\[
\begin{array}{ccc}
554.0 & -289.5 & 264.6 \\
-512.3 & 1518.3 & 1006.0 \\
\end{array}
\]

Self weight Jacking Stress

b) Just after initial loss

\[
\begin{array}{ccc}
554.0 & -282.9 & 271.1 \\
-512.3 & 1483.8 & 971.5 \\
\end{array}
\]

Self wt. prestress after initial loss

Refer cl. 7.1. Of IRC : 18 - 2000

c) After intermediate loss

\[
\begin{array}{ccc}
554.0 & -260.9 & 293.1 \\
-512.3 & 1368.8 & 856.4 \\
\end{array}
\]

Self weight prestress after Net effect intermediate loss

d) After casting of Deck Slab

\[
\begin{array}{ccc}
293.1 & 568.3 & 861.4 \\
856.4 & -525.5 & 330.9 \\
\end{array}
\]

Section Modulus at top of composite section = 1.5812 $m^3$

Section Modulus at junction of deck slab & girder $Z_j = 2.2620 m^3$

Section Modulus at C.G. of cables $Z_c = 1.1067 m^3$

Section Modulus at bottom of composite section = 0.8646 $m^3$

Composite area of deck slab + outer girder = 1.9063 $m^2$

**STAGE-II (28 TH DAY ONWARD)**

Prestressing force ( after 1st stage ) , $P_1 = 648.00 t$

Eccentricity of cables from bottom, $eb$ (1st stage) = 0.263 m

Eccentricity of cables from c.g., $e$ (1st stage) = 0.746 m

Prestressing force ( after friction and slip losses ) , $P_2 = 285.61 t$

Eccentricity of cables from bottom, $eb$ (2nd stage) = 0.490 m

Eccentricity of cables from c.g., $e$ (2nd stage) = 1.029 m

Net eccentricity of cables of 1st and 2nd stages, $eb' = 0.332 m$ (from bottom)

Net eccentricity of cables of 1st and 2nd stages, $e' = 1.187 m$ (from c.g.)
B.M. due to S.Wt. of the girder $M_g = 316.27$ tm
B.M. due to Cast in Situ slab $M_c$ (without shuttering) = 268.46 tm
B.M. due to SIDL $M_s = 163.30$ tm
B.M. due to Live loads $M_l = 238.92$ tm
B.M. due to FPLL = 0.00 tm

Prestressing force (after 1st stage), $P_1 = 648.00$ t
Moment due to $P_{ext} = M_{pext} = P_1 \times e = 483.35$ tm
for second stage prestressing
Prestressing force (after friction and slip losses), $P = 285.61$ t
Moment due to $P$, $M_p = P \times e = 294.03$ tm

**Stresses due to self weight of the girder + cast in situ slab**

\[
\sigma_j = \frac{584.73}{0.571} = 1024.3 \text{ t/m}^2
\]
\[
\sigma_c = \frac{584.73}{0.835} = -700.3 \text{ t/m}^2
\]
\[
\sigma_b = \frac{584.73}{0.617} = -947.2 \text{ t/m}^2
\]

**Stresses due to prestress (second stage cables)**

\[
\sigma_j = \frac{285.61}{1.906} = 294.03 \text{ t/m}^2
\]
\[
\sigma_c = \frac{285.61}{1.906} = 294.03 \text{ t/m}^2
\]
\[
\sigma_b = \frac{285.61}{1.906} = 294.03 \text{ t/m}^2
\]

Total Stresses due to prestress

\[
\sigma_j = 0.0 + (-36.1) = -36.1 \text{ t/m}^2
\]
\[
\sigma_c = -260.9 + 19.8 = -241.1 \text{ t/m}^2
\]
\[
\sigma_c = 1164.6 + 415.5 = 1580.1 \text{ t/m}^2
\]
\[
\sigma_b = 1368.8 + 489.9 = 1858.7 \text{ t/m}^2
\]

**Stresses due to prestress & self weight of the girder+deck slab**

\[
\sigma_j = 0.0 + (-36.1) = -36.1 \text{ t/m}^2
\]
\[
\sigma_c = 1024.3 + (-241.1) = 783.2 \text{ t/m}^2
\]
\[
\sigma_c = -700.3 + 1580.1 = 879.8 \text{ t/m}^2
\]
\[
\sigma_b = -947.2 + 1858.7 = 911.4 \text{ t/m}^2
\]

**Losses due to Elastic Shortening**

Loss due to Elastic Shortening = $0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss = 2378 t/m²

(i.e. Elastic Shortening loss) = 19.72 t = 1.79 % of Jacking Force

For first stage cables

Prestressing force after initial losses $P_1 = 642.37$ t
Moment due to $P_1 = M_{p1} = P_1 \times e = 479.15$ tm

For second stage cables

Prestressing force after initial losses $P_i = 279.98$ t
Moment due to $P_i = M_{pi} = P_i \times e = 288.23$ tm

**Loss due to Shrinkage of Concrete**

\[
\varepsilon_s \times E_s = \frac{3776.8}{31.31} = 120.6 \text{ t/m}^2
\]

Loss due to Shrinkage = 36.95 t
Moment due to Shrinkage loss = 43.850 tm
Loss due to relaxation of steel

For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.694
Loss due to friction and slip = $\frac{314.5861 - 285.6}{100} = 29.0 \text{ t}$

% friction and slip loss = $\frac{29.0 \times 100}{314.5861} = 6.45 \%$

Corresponding Relaxation Loss = $6394.2 \text{ t/m}^2$

Moment due to Relaxation & Shrinkage loss = $15.15 \text{ t}$

Loss due to Relaxation & Shrinkage = $20.78 \text{ t}$

Moments due to Relaxation & Shrinkage loss = $24.663 \text{ tm}$

Shrinkage losses (for first stage cables)

$\sigma_t = \frac{-36.95 + 43.850}{1.906} = 8.35 \text{ t/m}^2$

$\sigma_j = \frac{-36.95 + 43.850}{1.906} = 0.00 \text{ t/m}^2$

$\sigma_c = \frac{-36.95 - 43.850}{1.906} = -59.00 \text{ t/m}^2$

$\sigma_b = \frac{-36.95 - 43.850}{1.906} = -70.10 \text{ t/m}^2$

Shrinkage and Relaxation losses (for second stage cables)

$\sigma_t = \frac{-20.78 + 24.663}{1.906} = 4.70 \text{ t/m}^2$

$\sigma_j = \frac{-20.78 + 24.663}{1.906} = 0.00 \text{ t/m}^2$

$\sigma_c = \frac{-20.78 - 24.663}{1.906} = -33.19 \text{ t/m}^2$

$\sigma_b = \frac{-20.78 - 24.663}{1.906} = -39.43 \text{ t/m}^2$

Total Shrinkage and Relaxation losses

$\sigma_t = 8.35 + 4.70 = 13.05 \text{ t/m}^2$

$\sigma_j = 0.00 + 0.00 = 0.01 \text{ t/m}^2$

$\sigma_c = -59.00 + -33.19 = -92.19 \text{ t/m}^2$

$\sigma_b = -70.10 + -39.43 = -109.53 \text{ t/m}^2$

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder + cast insitu deck slab

$\sigma_t = -36.1 + 13.05 = -23.07 \text{ t/m}^2$

$\sigma_j = 783.2 + 0.0 = 783.20 \text{ t/m}^2$

$\sigma_c = 879.8 + -92.2 = 787.66 \text{ t/m}^2$

$\sigma_b = 911.4 + -109.5 = 801.92 \text{ t/m}^2$

Loss due to Creep of concrete

Arithmetic mean of initial and final stress after 28 days = $787.66 \text{ t/m}^2$

Creep loss = $(\sigma/10) \times c_e \times E_s = 6139.9 \text{ t/m}^2$

$= 50.91 \text{ t} = 4.623 \% \text{ of Jacking Force}$

Loss due to Creep = $50.91 \text{ t}$

Moment due to Creep loss = $60.42 \text{ tm}$
Creep losses (for first stage cables)

\[
\begin{align*}
\sigma_t &= -50.91 + 60.418 = 11.51 \text{ t/m}^2 \\
\sigma_i &= -50.91 + 60.418 = 0.01 \text{ t/m}^2 \\
\omega_c &= -50.91 - 60.418 = -81.30 \text{ t/m}^2 \\
\sigma_b &= -50.91 - 60.418 = -96.59 \text{ t/m}^2 \\
\end{align*}
\]

Creep losses (for second stage cables)

\[
\begin{align*}
\sigma_t &= -14.54 + -17.262 = -18.55 \text{ t/m}^2 \\
\sigma_i &= -14.54 + -17.262 = -15.26 \text{ t/m}^2 \\
\omega_c &= -14.54 - -17.262 = 7.97 \text{ t/m}^2 \\
\sigma_b &= -14.54 - -17.262 = 12.34 \text{ t/m}^2 \\
\end{align*}
\]

Final Prestressing Force = 775.57 t

Total Creep losses

\[
\begin{align*}
\sigma_t &= 11.51 + -18.55 = -7.04 \text{ t/m}^2 \\
\sigma_i &= 0.01 + -15.26 = -15.26 \text{ t/m}^2 \\
\omega_c &= -81.30 + 7.97 = -73.33 \text{ t/m}^2 \\
\sigma_b &= -96.59 + 12.34 = -84.25 \text{ t/m}^2 \\
\end{align*}
\]

Total Stresses due to prestress after immediate losses & self weight of the girder + cast insitu deck slab

\[
\begin{align*}
\sigma_t &= -23.07 + -7.04 = -30.11 \text{ t/m}^2 \\
\sigma_i &= 783.20 + -15.26 = 767.95 \text{ t/m}^2 \\
\omega_c &= 787.66 + -73.33 = 714.33 \text{ t/m}^2 \\
\sigma_b &= 801.92 + -84.25 = 717.67 \text{ t/m}^2 \\
\end{align*}
\]

Stresses Due to SIDL

\[
\begin{align*}
\sigma_t &= 163.30 = 103.3 \text{ t/m}^2 \\
\sigma_i &= 163.30 = 72.2 \text{ t/m}^2 \\
\omega_c &= 163.30 = -147.6 \text{ t/m}^2 \\
\sigma_b &= 163.30 = -188.9 \text{ t/m}^2 \\
\end{align*}
\]

Total Stresses due to prestress + DL (Girder + deck slab) + SIDL

\[
\begin{align*}
\sigma_t &= -30.1 + 103.27 = 73.2 \text{ t/m}^2 \\
\sigma_i &= 767.9 + 72.19 = 840.1 \text{ t/m}^2 \\
\omega_c &= 714.3 + -147.55 = 566.8 \text{ t/m}^2 \\
\sigma_b &= 717.7 + -188.88 = 528.8 \text{ t/m}^2 \\
\end{align*}
\]
After laying of SIDL

<table>
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<tr>
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<th>767.9</th>
<th>103.3</th>
<th>73.2</th>
</tr>
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<td></td>
<td>-30.1</td>
<td>72.2</td>
<td></td>
</tr>
<tr>
<td>SIDL</td>
<td>-188.9</td>
<td></td>
<td>528.8</td>
</tr>
</tbody>
</table>

Stresses due to Live load

\begin{align*}
\sigma_i &= 238.92 \times 1.581 = 151.10 \text{ t/m}^2 \\
\sigma_j &= 238.92 \times 2.262 = 105.62 \text{ t/m}^2 \\
\sigma_c &= 238.92 \times 1.107 = -215.88 \text{ t/m}^2 \\
\sigma_b &= 238.92 \times 0.865 = -276.35 \text{ t/m}^2 \\
\end{align*}

Stresses due to Prestress + DL + SIDL + FPLL + LL

\begin{align*}
\sigma_i &= 73.16 + 151.1 = 224.3 \text{ t/m}^2 \\
\sigma_j &= 840.14 + 105.6 = 945.8 \text{ t/m}^2 \\
\sigma_c &= 566.78 + -215.9 = 350.9 \text{ t/m}^2 \\
\sigma_b &= 528.79 + -276.3 = 252.4 \text{ t/m}^2 \\
\end{align*}

Time dependent losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

20% extra time dependent losses = 151.84 t
20% extra time dependent losses at top = 6.9 t/m²
20% extra time dependent losses at bottom = -57.6 t/m²
STRESS CHECK AT 3-8 TH (SEC 2-2)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, \( e_b \) = 0.283 m
Eccentricity of cables from c.g., \( e \) = 0.726 m
Sectional Modulus at C.G. of cables, \( Z_c \) = 0.858 m³ (for precast girder only)

B.M. due to S.Wt. of the girder \( M_d \) = 296.83 tm
B.M. due to Cast in Situ slab \( M_c \) = 301.26 tm
Prestressing force (after friction and slip losses), \( P \) = 725.33 t
Moment due to \( P \), \( M_p = P \times e \) = 526.66 tm

Stresses due to self weight of the girder
\[
\sigma_t = \frac{296.83}{0.571} = 519.97 \text{ t/m}^2 \\
\sigma_c = \frac{296.83}{0.858} = -346.04 \text{ t/m}^2 \\
\sigma_b = \frac{296.83}{0.617} = -480.85 \text{ t/m}^2
\]

Stresses due to prestress at transfer stage
\[
\sigma_t = \frac{725.33}{1.106} - \frac{526.66}{0.571} = -266.9 \text{ t/m}^2 \\
\sigma_c = \frac{725.33}{1.106} + \frac{526.66}{0.858} = 1269.64782 \text{ t/m}^2 \\
\sigma_b = \frac{725.33}{1.106} + \frac{526.66}{0.617} = 1508.8 \text{ t/m}^2
\]

Stresses due to initial prestress & self weight of the girder
\[
\sigma_t = \frac{520.0}{1.106} + \frac{-266.9}{0.571} = 253.1 \text{ t/m}^2 \\
\sigma_c = \frac{-346.0}{1.106} + \frac{1269.6}{0.858} = 923.6 \text{ t/m}^2 \\
\sigma_b = \frac{-480.8}{1.106} + \frac{1508.8}{0.617} = 1028.0 \text{ t/m}^2
\]

Losses due to Elastic Shortening

Loss due to Elastic Shortening = \( 0.5 \times \sigma \times E_s / E_c \)

Therefore, initial loss = 2791 t/m²
(i.e. Elastic Shortening loss) = 16.53 t = 2.10 % of Jacking Force

Prestressing force after initial losses \( P_i \) = 708.81 t
Moment due to \( P_i = M_p = P_i \times e \) = 514.66 tm

Loss due to Shrinkage of Concrete
\[
\sigma_s = \varepsilon_s \times E_s \\
= \frac{1192.7}{7.06 \text{ t/m}^2} = 0.90 \text{ % of Jacking Force}
\]
**Loss due to relaxation of steel**
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

- Ratio of average force to UTS = 0.685
- Loss due to friction & slip = 786.4653 - 725.3 = 61.1 t
- % friction and slip loss = 61.1 x 100 = 5.44 %
- Ratio of average force to UTS = 0.685 - 0.05441 = 0.631

Corresponding Relaxation Loss = 6474.5 t/m
= 38.34 t
4.88 % of Jacking Force

**Loss due to Relaxation & Shrinkage** = 45.41 t

Moment due to Relaxation & Shrinkage loss = 32.968 tm

**Stresses due to Prestress after initial losses**

- \( \sigma_t = \frac{708.81 - 514.66}{1.106} = -260.82 \) t/m²
- \( \sigma_c = \frac{708.81 + 514.66}{1.106} = 1240.7 \) t/m²
- \( \sigma_b = \frac{708.81 + 514.66}{1.106} = 1474.4 \) t/m²

**Stresses due to prestress after initial losses & self weight of the girder**

- \( \sigma_t = \frac{520.0 - 260.8}{1.106} = 259.1 \) t/m²
- \( \sigma_c = \frac{-346.0 + 1240.7}{1.106} = 894.7 \) t/m²
- \( \sigma_b = \frac{-480.8 + 1474.4}{1.106} = 993.6 \) t/m²

**Stresses after Shrinkage and Relaxation losses**

- \( \sigma_t = \frac{259.15 - 45.41 + 32.968}{1.106} = 275.85 \) t/m²
- \( \sigma_c = \frac{894.7 - 45.41 - 32.968}{1.106} = 815.19 \) t/m²
- \( \sigma_b = \frac{993.60 - 45.41 - 32.968}{1.106} = 899.15 \) t/m²

**Loss due to Creep of concrete**

- Arithmetic mean of initial and final stress between 14th and 28 days at C.G. of cables
- Creep loss = \( \left( \frac{\sigma}{100} \right) \times c \times E_s \) = 1832.7 t/m²
- Creep loss = 10.85 t = 1.380 % of Jacking Force

Total Initial & Intermediate losses = 72.79 t = 1.38 % of Jacking Force

(i.e. losses upto 28th day)

Force after intermediate losses, \( P_{ii} \) = 652.5 t

Moment due to \( P_{ii} \) = \( M_{int} = P_{ii} \times e \) = 473.81 tm

**Effect of prestress after Intermediate loss**

- \( \sigma_t = \frac{652.5 - 473.81}{1.106} = -240.1 \) t/m²
- \( \sigma_c = \frac{652.5 + 473.81}{1.106} = 1142.2 \) t/m²
- \( \sigma_b = \frac{652.5 + 473.81}{1.106} = 1357.4 \) t/m²

**Stresses Due to cast-in-situ slab**

- \( \sigma_t = \frac{301.3}{0.571} = 527.73 \) t/m²
- \( \sigma_c = \frac{301.3}{0.858} = -351.21 \) t/m²
- \( \sigma_b = \frac{-488.02}{0.617} = -488.02 \) t/m²
RESULTANT STRESSES

a) Transfer stage

\[
\begin{align*}
520.0 & -266.9 & 253.1 \\
-480.8 & + & 1508.8 \\
\text{Self weight} & & \text{Jacking Stress} \\
1028.0 & + & \\
\end{align*}
\]

b) Just after initial loss

\[
\begin{align*}
520.0 & -260.8 & 259.1 & > -163.2 \text{ t/m}^2 \text{ (O.K.)} \\
-480.8 & + & 1474.4 \\
\text{Self wt. prestress after initial loss} & & \\
993.6 & < 1632 \text{ t/m}^2 \text{ (O.K.)} \\
\text{Refer cl. 7.1. Of IRC : 18 - 2000} & & \\
\end{align*}
\]

c) After intermediate loss

\[
\begin{align*}
520.0 & -240.1 & 279.8 & > -163.2 \text{ t/m}^2 \text{ (O.K.)} \\
-480.8 & + & 1357.4 \\
\text{Self weight} & & \text{prestress after intermediate loss} \\
877 & < 1632 \text{ t/m}^2 \text{ (O.K.)} \\
\end{align*}
\]

\[
\begin{align*}
& \text{Net effect} \\
\end{align*}
\]

d) After casting of Deck Slab

\[
\begin{align*}
279.8 & 527.7 & 807.6 \\
- & -488.0 & + \\
\text{After Inter. Losses} & \text{Cast in situ slab} & > -163.2 \text{ t/m}^2 \text{ (O.K.)} \\
388.5 & + & \\
\end{align*}
\]

STAGE-II (28 TH DAY ONWARD)

\[
\begin{align*}
\text{Section Modulus at top of composite section} & = 1.58123097 \text{ m}^3 \\
\text{Section Modulus at junction of deck slab & girder } Z_j & = 2.2620 \text{ m}^3 \\
\text{Section Modulus at C.G. of cables } Z_c & = 1.136 \text{ m}^3 \\
\text{Section Modulus at bottom of composite section} & = 0.8646 \text{ m}^3 \\
\text{Composite area of deck slab + outer girder} & = 1.9063 \text{ m}^2 \\
\text{first stage cables} & \\
\text{Prestressing force ( after 1st stage ), } P_1 & = 652.55 \text{ t} \\
\text{ecc. of cables from bottom, } eb \text{ (1st stage)} & = 0.283 \text{ m} \\
\text{Eccentricity of cables from c.g., } e \text{ (1st stage)} & = 0.726 \text{ m} \\
\text{second stage cables} & \\
\text{Prestressing force ( after friction and slip losses ), } P & = 290.72 \text{ t} \\
\text{ecc. of cables from bottom, } eb \text{ (2nd stage)} & = 0.542 \text{ m} \\
\text{Eccentricity of cables from c.g., } e \text{ (2nd stage)} & = 0.977 \text{ m} \\
\text{Net eccentricity of cables of 1st and 2nd stages, } eb' & = 0.363 \text{ m (from bottom)} \\
\text{Net eccentricity of cables of 1st and 2nd stages, } e' & = 1.156 \text{ m (from c.g.)}
\end{align*}
\]
B.M. due to S.Wt. of the girder $M_d = 296.83$ tm
B.M. due to Cast in Situ slab $M_c = 245.30$ tm
B.M. due to SIDL $M_i = 164.75$ tm
B.M. due to Live loads $M_L = 207.59$ tm
B.M. due to FPPLL $M_{PLL} = 0.00$ tm

Prestressing force (after 1st stage) $P_1 = 652.55$ t
Moment due to $P_{1it} = M_{1it} = P_1 \times e = 473.81$ tm

for second stage prestressing

Prestressing force (after friction and slip losses) $P = 290.72$ t
Moment due to $P, M_p = P \times e = 284.05$ tm

**Stresses due to self weight of the girder + cast in situ slab**

\[
\begin{align*}
\sigma_j &= 542.13 = 949.7 \text{ t/m}^2 \\
0.571 &
\end{align*}
\]
\[
\begin{align*}
\sigma_c &= 542.13 = -632.0 \text{ t/m}^2 \\
0.858 &
\end{align*}
\]
\[
\begin{align*}
\sigma_b &= 542.13 = -878.2 \text{ t/m}^2 \\
0.617 &
\end{align*}
\]

**Stresses due to prestress (second stage cables)**

\[
\begin{align*}
\sigma_j &= 290.72 - 284.05 = -27.1 \text{ t/m}^2 \\
1.906 &
\end{align*}
\]
\[
\begin{align*}
\sigma_c &= 290.72 - 284.05 = 26.9 \text{ t/m}^2 \\
1.906 &
\end{align*}
\]
\[
\begin{align*}
\sigma_c &= 290.72 + 284.05 = 402.6 \text{ t/m}^2 \\
1.906 &
\end{align*}
\]
\[
\begin{align*}
\sigma_c &= 290.72 + 284.05 = 481.0 \text{ t/m}^2 \\
1.906 &
\end{align*}
\]

**Total Stresses due to prestress**

\[
\begin{align*}
\sigma_i &= 0.0 + -27.1 = -27.1 \text{ t/m}^2 \\
\sigma_j &= -240.1 + 26.9 = -213.2 \text{ t/m}^2 \\
\sigma_c &= 1142.2 + 402.6 = 1544.8 \text{ t/m}^2 \\
\sigma_b &= 1357.4 + 481.0 = 1838.5 \text{ t/m}^2 
\end{align*}
\]

**Stresses due to prestress & self weight of the girder+deck slab**

\[
\begin{align*}
\sigma_i &= 0.0 + -27.1 = -27.1 \text{ t/m}^2 \\
\sigma_j &= 949.7 + -213.2 = 736.5 \text{ t/m}^2 \\
\sigma_c &= -632.0 + 1544.8 = 912.8 \text{ t/m}^2 \\
\sigma_b &= -878.2 + 1838.5 = 960.3 \text{ t/m}^2 
\end{align*}
\]

**Losses due to Elastic Shortening**

Loss due to Elastic Shortening $= 0.5 \times \sigma \times E_s/E_c$

Therefore, initial loss $= 2467 \text{ t/m}^2$

(i.e. Elastic Shortening loss) $= 20.46 \text{ t} = 91.03$ % of Jacking Force

**For first stage cables**

Prestressing force after initial losses $P_1 = 646.70$ t
Moment due to $P_1 = M_{1it} = P_1 \times e = 469.57$ tm

**For second stage cables**

Prestressing force after initial losses $P_i = 284.87$ t
Moment due to $P_i = M_{iit} = P_i \times e = 278.34$ tm

**Loss due to Shrinkage of Concrete**

\[
\begin{align*}
C &= \varepsilon_s \times E_s \\
&= 3776.8 \text{ t/m}^2 \\
&= 31.31 \text{ t} = 2.84$ % of Jacking Force 
\end{align*}
\]

Loss due to Shrinkage $= 37.16$ t

Moment due to Shrinkage loss $= 42.972$ tm
**Loss due to relaxation of steel**

**For second stage cables:**
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.381

Loss due to friction and slip = \( \frac{314.5861 \times 100}{100} = 7.588 \% \)

% friction and slip loss = \( \frac{23.9}{314.58612} \times 100 = 7.588 \% \)

Ratio of average force to UTS = 0.381 - 0.075879 = 0.306

Corresponding Relaxation Loss = 0.0 t/m

0.00 % of Jacking Force

Loss due to Relaxation & Shrinkage = 5.84 t

Moment due to Relaxation & Shrinkage loss = 6.759 tm

**Shrinkage losses (for first stage cables)**

\[ \sigma_1 = \frac{-37.16 + 42.972}{1.906 + 1.581} = 7.68 \text{ t/m}^2 \]

\[ \sigma_2 = \frac{-37.16 + 42.972}{1.906 + 2.262} = -0.49 \text{ t/m}^2 \]

\[ \sigma_3 = \frac{-37.16 - 42.972}{1.906 + 1.136} = -57.33 \text{ t/m}^2 \]

\[ \sigma_4 = \frac{-37.16 - 42.972}{1.906 + 0.865} = -69.20 \text{ t/m}^2 \]

**Shrinkage and Relaxation losses (for second stage cables)**

\[ \sigma_1 = \frac{-5.84 + 6.759}{1.906 + 1.581} = 1.21 \text{ t/m}^2 \]

\[ \sigma_2 = \frac{-5.84 + 6.759}{1.906 + 2.262} = -0.08 \text{ t/m}^2 \]

\[ \sigma_3 = \frac{-5.84 - 6.759}{1.906 + 1.136} = -9.02 \text{ t/m}^2 \]

\[ \sigma_4 = \frac{-5.84 - 6.759}{1.906 + 0.865} = -10.88 \text{ t/m}^2 \]

**Total Shrinkage and relaxation losses**

\[ \sigma_1 = \frac{7.68 + 1.21}{1.906 + 1.581} = 8.89 \text{ t/m}^2 \]

\[ \sigma_2 = \frac{-0.49 + -0.08}{1.906 + 2.262} = -0.57 \text{ t/m}^2 \]

\[ \sigma_3 = \frac{-57.33 + -9.02}{1.906 + 1.136} = -66.34 \text{ t/m}^2 \]

\[ \sigma_4 = \frac{-69.20 + -10.88}{1.906 + 0.865} = -80.08 \text{ t/m}^2 \]

**Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder +cast insitu deck slab**

\[ \sigma_1 = \frac{-27.1 + 8.9}{1.906 + 0.6} = -18.24 \text{ t/m}^2 \]

\[ \sigma_2 = \frac{736.5 + -0.6}{1.906 + 0.6} = 735.91 \text{ t/m}^2 \]

\[ \sigma_3 = \frac{912.8 + -66.3}{1.906 + 0.6} = 846.48 \text{ t/m}^2 \]

\[ \sigma_4 = \frac{960.3 + -80.1}{1.906 + 0.6} = 880.17 \text{ t/m}^2 \]

**Loss due to Creep of concrete**

Arithmetic mean of initial and final stress after 28 days = 846.48 t/m²

Creep loss = \( (\sigma/10) \times e_0 \times E_s \) = 6598.4 t/m²

\[ = \frac{54.71}{4.969 \%} \text{ of Jacking Force} \]

Loss due to Creep = 54.71 t

Moment due to Creep loss = 63.27 tm
Creep losses (for first stage cables)

\[
\begin{align*}
\sigma_t &= -54.71 + \frac{63.268}{1.906} = 11.31 \text{ t/m}^2 \\
\sigma_j &= -54.71 + \frac{63.268}{1.906} = -0.73 \text{ t/m}^2 \\
\sigma_c &= -54.71 - \frac{63.268}{1.906} = -84.40 \text{ t/m}^2 \\
\sigma_b &= -54.71 - \frac{63.268}{1.906} = -101.88 \text{ t/m}^2 \\
\end{align*}
\]

Creep losses (for second stage cables)

\[
\begin{align*}
\sigma_t &= -15.63 + \frac{-18.077}{1.906} = -19.63 \text{ t/m}^2 \\
\sigma_j &= -15.63 + \frac{-18.077}{1.906} = -16.19 \text{ t/m}^2 \\
\sigma_c &= -15.63 - \frac{-18.077}{1.906} = 7.72 \text{ t/m}^2 \\
\sigma_b &= -15.63 - \frac{-18.077}{1.906} = 12.71 \text{ t/m}^2 \\
\end{align*}
\]

Final Prestressing force = 809.47 t

Total Creep losses

\[
\begin{align*}
\sigma_t &= 11.31 + -19.63 = -8.32 \text{ t/m}^2 \\
\sigma_j &= -0.73 + -16.19 = -16.92 \text{ t/m}^2 \\
\sigma_c &= -84.40 + 7.72 = -76.69 \text{ t/m}^2 \\
\sigma_b &= -101.88 + 12.71 = -89.17 \text{ t/m}^2 \\
\end{align*}
\]

Total Stresses due to prestress after immediate losses & self weight of the girder
+cast insitu deck slab

\[
\begin{align*}
\sigma_t &= -18.24 + -8.32 = -26.56 \text{ t/m}^2 \\
\sigma_j &= 735.91 + -16.92 = 718.99 \text{ t/m}^2 \\
\sigma_c &= 846.48 + -76.69 = 769.79 \text{ t/m}^2 \\
\sigma_b &= 880.17 + -89.17 = 791.00 \text{ t/m}^2 \\
\end{align*}
\]

Stresses Due to SIDL

\[
\begin{align*}
\sigma_t &= 164.75 = 104.2 \text{ t/m}^2 \\
\sigma_j &= 164.75 = 72.8 \text{ t/m}^2 \\
\sigma_c &= 164.75 = -145.1 \text{ t/m}^2 \\
\sigma_b &= 164.75 = -190.6 \text{ t/m}^2 \\
\end{align*}
\]

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

\[
\begin{align*}
\sigma_t &= -26.6 + 104.19 = 77.6 \text{ t/m}^2 \\
\sigma_j &= 719.0 + 72.83 = 791.8 \text{ t/m}^2 \\
\sigma_c &= 769.8 + -145.05 = 624.7 \text{ t/m}^2 \\
\sigma_b &= 791.0 + -190.56 = 600.4 \text{ t/m}^2 \\
\end{align*}
\]
After laying of SIDL

Stresses due to Live load

\[
\begin{align*}
\sigma_t &= 207.59 = 131.28 \text{ t/m}^2 \\
\sigma_j &= 207.59 = 91.77 \text{ t/m}^2 \\
\sigma_c &= 207.59 = -182.7717 \text{ t/m}^2 \\
\sigma_b &= 207.59 = -240.11 \text{ t/m}^2
\end{align*}
\]

Stresses due to Prestress + DL + SIDL + FPLL + LL

\[
\begin{align*}
\sigma_t &= 77.63 + 131.3 = 208.9 \text{ t/m}^2 \\
\sigma_j &= 791.82 + 91.8 = 883.6 \text{ t/m}^2 \\
\sigma_c &= 624.74 + -182.8 = 442.0 \text{ t/m}^2 \\
\sigma_b &= 600.45 + -240.1 = 360.3 \text{ t/m}^2
\end{align*}
\]

Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

\[
\begin{align*}
\text{20% extra time dependant losses} &= 142.28 \text{ t} \\
\text{Stress due to 20% extra time dependant losses at top} &= 5.9 \text{ t/m}^2 \\
\text{Stress due to 20% extra time dependant losses at bottom} &= -53.0 \text{ t/m}^2
\end{align*}
\]
STRESS CHECK AT 1-4 TH (SEC 3-3)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, \(e_b\) = 0.341 m

Eccentricity of cables from c.g., \(e\) = 0.668 m

Sectional Modulus at C.G. of cables, \(Z_c\) = 0.932 m³ (for precast girder only)

B.M. due to S.Wt. of the girder \(M_d\) = 238.50 t.m

B.M. due to Cast in Situ slab \(M_c\) = 242.95 t.m

Prestressing force (after friction and slip losses), \(P\) = 709.01 t

Moment due to \(P\), \(M_p\) = \(P \times e\) = 473.62 t.m

Stresses due to self weight of the girder

\(\sigma_t\) = \(\frac{238.50}{0.571}\) = 417.79 t/m²

\(\sigma_c\) = \(\frac{238.50}{0.932}\) = -255.80 t/m²

\(\sigma_b\) = \(\frac{238.50}{0.617}\) = -386.36 t/m²

Stresses due to prestress at transfer stage

\(\sigma_t\) = \(\frac{709.01}{1.106}\) - \(\frac{473.62}{0.571}\) = -188.8 t/m²

\(\sigma_c\) = \(\frac{709.01}{1.106}\) + \(\frac{473.62}{0.932}\) = 1148.89 t/m²

\(\sigma_b\) = \(\frac{709.01}{1.106}\) + \(\frac{473.62}{0.617}\) = 1408.2 t/m²

Stresses due to initial prestress & self weight of the girder

\(\sigma_t\) = \(\frac{417.8}{1.106}\) + \(\frac{-188.8}{0.571}\) = 229.0 t/m²

\(\sigma_c\) = \(\frac{-255.8}{1.106}\) + \(\frac{1148.9}{0.932}\) = 893.1 t/m²

\(\sigma_b\) = \(\frac{-386.4}{1.106}\) + \(\frac{1408.2}{0.617}\) = 1021.8 t/m²

Losses due to Elastic Shortening

Loss due to Elastic Shortening = 0.5 x \(\sigma\) x \(E_s\)/\(E_E\)

Therefore, initial loss = 2699 t/m²

(i.e. Elastic Shortening loss) = 15.98 t = 2.03 % of Jacking Force

Prestressing force after initial losses \(P_i\) = 693.03 t

Moment due to \(P_i\), \(M_{pi}\) = \(P_i \times e\) = 462.95 t.m

Loss due to Shrinkage of Concrete = \(\varepsilon_s\) x \(E_s\)

= 1192.7 t/m²

= 7.06 t = 0.90 % of Jacking Force
Loss due to relaxation of steel

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

\[
\text{Ratio of average force to UTS} = \frac{786.4653}{0.686} = 709.0 = 77.5 \text{ t}
\]

\[
\text{% friction and slip loss} = \frac{77.5 \times 100}{786.46529} = 6.894 \%
\]

Corresponding Relaxation Loss = \( \frac{5780.0}{t/m^2} \) 34.23 t

\[
\text{Loss due to Relaxation & Shrinkage} = 41.29 \text{ t}
\]

Moment due to Relaxation & Shrinkage loss = 27.583 tm

Stresses due to Prestress after initial losses

\[
\sigma_i = 693.03 - 462.95 = -184.50 \text{ t/m}^2
\]

\[
\sigma_c = 693.03 + 462.95 = 1123.0 \text{ t/m}^2
\]

\[
\sigma_b = 693.03 + 462.95 = 1376.4 \text{ t/m}^2
\]

Stresses due to prestress after initial losses & self weight of the girder

\[
\sigma_i = 417.8 + -184.5 = 233.3 \text{ t/m}^2
\]

\[
\sigma_c = -255.8 + 1123.0 = 867.2 \text{ t/m}^2
\]

\[
\sigma_b = -386.4 + 1376.4 = 990.1 \text{ t/m}^2
\]

Stresses after Shrinkage and Relaxation losses

\[
\sigma_i = 233.29 - 41.29 + 27.583 = 244.29 \text{ t/m}^2
\]

\[
\sigma_c = 867.2 - 41.29 - 27.583 = 800.28 \text{ t/m}^2
\]

\[
\sigma_b = 990.06 - 41.29 - 27.583 = 908.05 \text{ t/m}^2
\]

Loss due to Creep of concrete

Arithmetic mean of initial and final stress between 14th and 28 days at C.G. of cables

\[
\text{Creep loss} = \frac{(\sigma_i + \sigma_f)}{2} \cdot e \cdot E_s = 1787.3 \text{ t/m}^2
\]

\[
= 10.58 \text{ t} = 1.346 \% \text{ of Jacking Force}
\]

Total Initial & Intermediate losses = 67.86 t = 1.35 \% of Jacking Force

(i.e. losses upto 28th day)

Force after intermediate losses, \( P_{int} \) = 641.2 t

Moment due to \( P_{int} \) = \( M_{int} = P_{int} \cdot e \cdot s = 428.29 \text{ tm} \)

Effect of prestress after Intermediate loss

\[
\sigma_i = 641.2 - 428.29 = -170.7 \text{ t/m}^2
\]

\[
\sigma_c = 641.2 + 428.29 = 1038.9 \text{ t/m}^2
\]

\[
\sigma_b = 641.2 + 428.29 = 1273.4 \text{ t/m}^2
\]

Stresses Due to cast-in-situ slab

\[
\sigma_i = 243.0 = 425.59 \text{ t/m}^2
\]

\[
\sigma_c = 243.0 = -260.57 \text{ t/m}^2
\]

\[
\sigma_b = 243.0 = -393.56 \text{ t/m}^2
\]
RESULTANT STRESSES

a) Transfer stage

\begin{align*}
\text{Self weight} & -386.4 \\
\text{Jacking Stress} & 1408.2 \\
\text{Total} & 1021.8 \\
\end{align*}

b) Just after initial loss

\begin{align*}
\text{Self wt. prestress after initial loss} & -386.4 \\
\text{Prestress after initial loss} & 990.1 < 1632 \text{ t/m}^2 (O.K.)
\end{align*}

Refer cl. 7.1. Of IRC : 18 - 2000

c) After intermediate loss

\begin{align*}
\text{Self weight} & -386.4 \\
\text{Prestress after intermediate loss} & 1273.4 \\
\text{Net effect} & 887 < 1632 \text{ t/m}^2 (O.K.)
\end{align*}

d) After casting of Deck Slab

\begin{align*}
\text{After Inter. Losses} & -393.6 \\
\text{Cast in situ slab} & 493.5 \\
\text{Total} & 163.2 \text{ t/m}^2 (O.K.)
\end{align*}

STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section = 1.58123097 m³
Section Modulus at junction of deck slab & girder $Z_j$ = 2.2620 m³
Section Modulus at C.G. of cables $Z_c$ = 1.229 m³
Section Modulus at bottom of composite section = 0.8646 m³
Composite area of deck slab + outer girder = 1.9063 m²

First stage cables

\begin{align*}
\text{Prestressing force ( after 1st stage )} & = 641.15 \text{ t} \\
\text{Eccentricity of cables from bottom, } & eb (1st stage) = 0.341 \text{ m} \\
\text{Eccentricity of cables from c.g., } & e (1st stage) = 0.668 \text{ m}
\end{align*}

Second stage cables

\begin{align*}
\text{Prestressing force ( after friction and slip losses )} & = 283.21 \text{ t} \\
\text{Eccentricity of cables from bottom, } & eb (2nd stage) = 0.699 \text{ m} \\
\text{Eccentricity of cables from c.g., } & e (2nd stage) = 0.820 \text{ m}
\end{align*}
Net eccentricity of cables of 1st and 2nd stages, \( e_b' = 0.451 \text{ m (from bottom)} \)
Net eccentricity of cables of 1st and 2nd stages, \( e' = 1.069 \text{ m (from c.g.)} \)

B.M. due to S.Wt. of the girder \( M_d \) = 238.50 tm
B.M. due to Cast in Situ slab \( M_c \) = 186.99 tm
B.M. due to SIDL \( M_s \) = 126.54 tm
B.M. due to Live loads \( M_L \) = 175.11 tm
B.M. due to FPLL = 0.00 tm

Prestressing force (after 1st stage), \( P_1 \) = 641.15 t
Moment due to \( P_1 \) = \( M_{p1} = P_1 \times e \) = 428.29 tm

Prestressing force (after friction and slip losses), \( P \) = 283.21 t
Moment due to \( P \), \( M_p = P \times e \) = 232.36 tm

Stresses due to self weight of the girder + cast in situ slab
\[
\sigma_j = \frac{425.49}{0.571} = 745.3 \text{ t/m}^2
\]
\[
\sigma_c = \frac{425.49}{0.932} = -456.3 \text{ t/m}^2
\]
\[
\sigma_b = \frac{425.49}{0.617} = -689.3 \text{ t/m}^2
\]

Stresses due to prestress (second stage cables)
\[
\sigma_i = \frac{283.21}{1.906} - 232.36 = 1.6 \text{ t/m}^2
\]
\[
\sigma_j = \frac{283.21}{1.906} - 232.36 = 45.8 \text{ t/m}^2
\]
\[
\sigma_c = \frac{283.21}{1.906} + 232.36 = 337.6 \text{ t/m}^2
\]
\[
\sigma_b = \frac{283.21}{1.906} + 232.36 = 417.3 \text{ t/m}^2
\]

Total Stresses due to prestress
\[
\sigma_i = 0.0 + 1.6 = 1.6 \text{ t/m}^2
\]
\[
\sigma_i = -170.7 + 45.8 = -124.8 \text{ t/m}^2
\]
\[
\sigma_c = \frac{1038.9}{1.906} + 337.6 = 1376.5 \text{ t/m}^2
\]
\[
\sigma_b = \frac{1273.4}{1.906} + 417.3 = 1690.7 \text{ t/m}^2
\]

Stresses due to prestress & self weight of the girder + deck slab
\[
\sigma_i = 0.0 + 1.6 = 1.6 \text{ t/m}^2
\]
\[
\sigma_i = 745.3 + -124.8 = 620.5 \text{ t/m}^2
\]
\[
\sigma_c = \frac{-456.3}{1.906} + 1376.5 = 920.2 \text{ t/m}^2
\]
\[
\sigma_b = \frac{-689.3}{1.906} + 1690.7 = 1001.4 \text{ t/m}^2
\]

Losses due to Elastic Shortening
\[
\text{Loss due to Elastic Shortening} = 0.5 \times \sigma \times E_p/E_c
\]
Therefore, initial loss = 2487 t/m²
(i.e. Elastic Shortening loss) = 20.62 t = 1.87 % of Jacking Force

For first stage cables
Prestressing force after initial losses \( P_1 \) = 635.26 t
Moment due to \( P_1 \) = \( M_{p1} = P_1 \times e \) = 424.36 tm

For second stage cables
Prestressing force after initial losses \( P_1 \) = 277.32 t
Moment due to \( P_1 \) = \( M_{p1} = P_1 \times e \) = 227.53 tm
Loss due to Shrinkage of Concrete  
\[ \varepsilon_s \times E_s = 3776.8 \text{ t/m}^2 \]
\[ = 31.31 \text{ t} = 2.84 \% \text{ of Jacking Force} \]

Loss due to Shrinkage  
\[ = 37.20 \text{ t} \]
Moment due to Shrinkage loss  
\[ = 39.760 \text{ tm} \]

Loss due to relaxation of steel

For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS  
\[ = 0.693 \]

Loss due to friction and slip  
\[ = 314.5861 - 283.2 = 31.4 \text{ t} \]
% friction and slip loss  
\[ = \frac{31.4}{314.5861} \times 100 = 6.98 \% \]

Ratio of average force to UTS  
\[ = 0.693 - 0.069817 = 0.624 \]

Corresponding Relaxation Loss  
\[ = 6115.9 \text{ t/m}^2 \]
\[ = 14.49 \text{ t} = 4.61 \% \text{ of Jacking Force} \]

Loss due to Relaxation & Shrinkage  
\[ = 20.38 \text{ t} \]
Moment due to Relaxation & Shrinkage loss  
\[ = 21.779 \text{ tm} \]

Shrinkage losses (for first stage cables)

\[ \sigma_s = \frac{-37.20}{1.906} + \frac{39.760}{1.581} = 5.63 \text{ t/m}^2 \]
\[ \sigma_j = \frac{-37.20}{1.906} + \frac{39.760}{2.262} = -1.94 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-37.20}{1.906} - \frac{39.760}{1.229} = -51.87 \text{ t/m}^2 \]
\[ \sigma_b = \frac{-37.20}{1.906} - \frac{39.760}{0.865} = -65.50 \text{ t/m}^2 \]

Shrinkage and Relaxation losses (for second stage cables)

\[ \sigma_s = \frac{-20.38}{1.906} + \frac{21.779}{1.581} = 3.08 \text{ t/m}^2 \]
\[ \sigma_j = \frac{-20.38}{1.906} + \frac{21.779}{2.262} = -1.06 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-20.38}{1.906} - \frac{21.779}{1.229} = -28.41 \text{ t/m}^2 \]
\[ \sigma_b = \frac{-20.38}{1.906} - \frac{21.779}{0.865} = -35.88 \text{ t/m}^2 \]

Total Shrinkage and Relaxation losses

\[ \sigma_s = 5.63 + 3.08 = 8.71 \text{ t/m}^2 \]
\[ \sigma_j = -1.94 + -1.06 = -3.00 \text{ t/m}^2 \]
\[ \sigma_c = -51.87 + -28.41 = -80.28 \text{ t/m}^2 \]
\[ \sigma_b = -65.50 + -35.88 = -101.39 \text{ t/m}^2 \]

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder + cast insitu deck slab

\[ \sigma_s = 1.6 + 8.7 = 10.33 \text{ t/m}^2 \]
\[ \sigma_j = 620.5 + -3.0 = 617.50 \text{ t/m}^2 \]
\[ \sigma_c = 920.2 + -80.3 = 839.93 \text{ t/m}^2 \]
\[ \sigma_b = 1001.4 + -101.4 = 900.06 \text{ t/m}^2 \]

Loss due to Creep of concrete

Arithmetic mean of initial and final stress after 28 days  
\[ = 839.93 \text{ t/m}^2 \]

Creep loss  
\[ = (\sigma/10) \times \varepsilon_c \times E_s \]
\[ = 6547.4 \text{ t/m}^2 \]
\[ = 54.28 \text{ t} = 4.930 \% \text{ of Jacking Force} \]
Loss due to Creep         = 54.28 t
Moment due to Creep loss = 58.01 tm

Creep losses (for first stage cables)

\[
\sigma_i = -54.28 + \frac{58.012}{1.906} = 8.21 \text{ t/m}^2 \\
\sigma_j = -54.28 + \frac{58.012}{1.591} = -2.83 \text{ t/m}^2 \\
\sigma_c = -54.28 - \frac{58.012}{1.906} = -75.67 \text{ t/m}^2 \\
\sigma_b = -54.28 - \frac{58.012}{1.229} = -95.58 \text{ t/m}^2 \\
\]

Creep losses (for second stage cables)

\[
\sigma_i = -15.51 + \frac{-16.575}{1.906} = -18.62 \text{ t/m}^2 \\
\sigma_j = -15.51 + \frac{-16.575}{1.581} = -15.46 \text{ t/m}^2 \\
\sigma_c = -15.51 - \frac{-16.575}{1.906} = 5.35 \text{ t/m}^2 \\
\sigma_b = -15.51 - \frac{-16.575}{1.229} = 11.04 \text{ t/m}^2 \\
\]

Final Prestressing Force = 761.88 t

Total Creep losses

\[
\sigma_i = 8.21 + -18.62 = -10.41 \text{ t/m}^2 \\
\sigma_j = -2.83 + -15.46 = -18.29 \text{ t/m}^2 \\
\sigma_c = -75.67 + 5.35 = -70.33 \text{ t/m}^2 \\
\sigma_b = -95.58 + 11.04 = -84.54 \text{ t/m}^2 \\
\]

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

\[
\sigma_i = 10.33 + -10.41 = -0.08 \text{ t/m}^2 \\
\sigma_j = 617.50 + -18.29 = 599.21 \text{ t/m}^2 \\
\sigma_c = 839.93 + -70.33 = 769.60 \text{ t/m}^2 \\
\sigma_b = 900.06 + -84.54 = 815.52 \text{ t/m}^2 \\
\]

Stresses Due to SIDL

\[
\sigma_i = 126.54 = 80.0 \text{ t/m}^2 \\
\sigma_j = 126.54 = 55.9 \text{ t/m}^2 \\
\sigma_c = 126.54 = -103.0 \text{ t/m}^2 \\
\sigma_b = 126.54 = -146.4 \text{ t/m}^2 \\
\]

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

\[
\sigma_i = -0.1 + 80.03 = 79.9 \text{ t/m}^2 \\
\sigma_j = 599.2 + 55.94 = 655.2 \text{ t/m}^2 \\
\sigma_c = 769.6 + -102.95 = 666.6 \text{ t/m}^2 \\
\sigma_b = 815.5 + -146.36 = 669.2 \text{ t/m}^2 \\
\]
After laying of SIDL

Stresses due to Live load

\[ \sigma_l = 175.11 \text{ t/m}^2 \]

\[ \sigma_j = 175.11 \times 1.581 \text{ t/m}^2 \]

\[ \sigma_c = 175.11 \times 2.262 \text{ t/m}^2 \]

\[ \sigma_b = 175.11 \times 1.229 \text{ t/m}^2 \]

Stresses due to Prestress + DL + SIDL + FPLL + LL

\[ \sigma_l = 79.95 \text{ t/m}^2 + 110.7 \text{ t/m}^2 = 190.7 \text{ t/m}^2 \]

\[ \sigma_j = 655.15 \text{ t/m}^2 + 77.4 \text{ t/m}^2 = 732.6 \text{ t/m}^2 \]

\[ \sigma_c = 666.65 \text{ t/m}^2 + -142.5 \text{ t/m}^2 = 524.2 \text{ t/m}^2 \]

\[ \sigma_b = 669.16 \text{ t/m}^2 + -202.5 \text{ t/m}^2 = 466.6 \text{ t/m}^2 \]

Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)

20% extra time dependant losses = 151.96 t

Stress due to 20% extra time dependant losses at top = 4.6 t/m²

Stress due to 20% extra time dependant losses at bottom = -53.5 t/m²
STRESS CHECK AT 1-8 TH (SEC 4-4)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb = 0.495 m
Eccentricity of cables from c.g., e = 0.538 m
Sectional Modulus at C.G. of cables, Zc = 1.219 m³
(for precast girder only)

B.M. due to S.Wt. of the girder Mg = 141.29 tm
B.M. due to Cast in Situ slab Mc = 138.33 tm
Prestressing force (after friction and slip losses), P = 689.33 t
Moment due to P, M_p = P x e = 370.81 tm

Stresses due to self weight of the girder
\[ \sigma_t = \frac{141.29}{0.615} \]
\[ \sigma_c = \frac{141.29}{1.219} \]
\[ \sigma_b = \frac{141.29}{0.635} \]

Stresses due to prestress at transfer stage
\[ \sigma_t = \frac{689.33}{1.316} - \frac{370.81}{0.615} \]
\[ \sigma_c = \frac{689.33}{1.316} + \frac{370.81}{1.219} \]
\[ \sigma_b = \frac{689.33}{1.316} + \frac{370.81}{0.635} \]

Stresses due to initial prestress & self weight of the girder
\[ \sigma_t = 229.9 + (-79.6) = 150.3 \text{ t/m}^2 \]
\[ \sigma_c = -115.9 + 828.0 = 712.1 \text{ t/m}^2 \]
\[ \sigma_b = -222.5 + 1107.7 = 885.2 \text{ t/m}^2 \]

Losses due to Elastic Shortening

Loss due to Elastic Shortening = 0.5 x \( \varepsilon_s x E_s / E_c \)

Therefore, initial loss = 2152 t/m²
(i.e. Elastic Shortening loss) = 12.74 t = 1.62 % of Jacking Force

Prestressing force after initial losses P_i = 676.59 t
Moment due to P_i, M_p = P_i x e = 363.96 tm

Loss due to Shrinkage of Concrete
\[ = \varepsilon_s x E_s \]
\[ = 1192.7 \text{ t/m}^2 \]
\[ = 7.06 \text{ t} = 0.90 \% \text{ of Jacking Force} \]
**Loss due to relaxation of steel**

The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.689

Loss due to friction & slip = \( \frac{786.4653 \times 689.3}{100} = 6.845 \% \)

% friction and slip loss = \( \frac{97.1}{786.4653} \times 100 = 8.645 \% \)

Ratio of average force to UTS = 0.689

Corresponding Relaxation Loss = 5056.0 \( \text{t/m}^2 \)

= 29.94 \( \text{t} \)

3.81 % of Jacking Force

Loss due to Relaxation & Shrinkage = 37.00 \( \text{t} \)

Moment due to Relaxation & Shrinkage loss = 19.906 \( \text{tm} \)

**Stresses due to Prestress after initial losses**

\[ \sigma_t = \frac{676.59 + 363.96}{1.316} = 676.59 \text{ t/m}^2 \]

\[ \sigma_c = \frac{676.59 - 363.96}{1.316} = 363.96 \text{ t/m}^2 \]

\[ \sigma_b = \frac{676.59 + 363.96}{1.316} = 676.59 \text{ t/m}^2 \]

**Stresses due to prestress after initial losses & self weight of the girder**

\[ \sigma_t = \frac{229.9 - 78.1}{1.316} = 151.8 \text{ t/m}^2 \]

\[ \sigma_c = \frac{-115.9 + 812.7}{1.316} = 696.8 \text{ t/m}^2 \]

\[ \sigma_b = \frac{-222.5 + 1087.2}{1.316} = 864.8 \text{ t/m}^2 \]

**Stresses after Shrinkage and Relaxation losses**

\[ \sigma_t = \frac{151.8 - 37.00 + 19.906}{1.316} = 151.8 \text{ t/m}^2 \]

\[ \sigma_c = \frac{696.8 - 37.00 - 19.906}{1.316} = 696.8 \text{ t/m}^2 \]

\[ \sigma_b = \frac{864.77 - 37.00 - 19.906}{1.316} = 864.8 \text{ t/m}^2 \]

**Loss due to Creep of concrete**

Arithmetic mean of initial and final stress between 14th and 28 days at C.G. of cables = 674.56 \( \text{t/m}^2 \)

Creep loss = \( \frac{\sigma}{10} \times E_c \times \varepsilon_c \times E_s \)

= 1446.0 \( \text{t/m}^2 \)

= 8.56 \( \text{t} \)

1.089 % of Jacking Force

Total Initial & Intermediate losses = 58.31 \( \text{t} \)

(i.e. losses upto 28th day)

Force after intermediate losses, \( P_{int} \) = 631.0 \( \text{t} \)

Moment due to \( P_{int} \) = \( M_{pint} = P_{int} \times e \)

= 339.45 \( \text{tm} \)

**Effect of prestress after Intermediate loss**

\[ \sigma_t = \frac{631.0 - 339.45}{1.316} = 225.10 \text{ t/m}^2 \]

\[ \sigma_c = \frac{631.0 + 339.45}{1.316} = 631.0 \text{ t/m}^2 \]

\[ \sigma_b = \frac{631.0 + 339.45}{1.316} = 631.0 \text{ t/m}^2 \]

**Stresses Due to cast-in-situ slab**

\[ \sigma_t = \frac{138.3}{0.615} = 225.10 \text{ t/m}^2 \]

\[ \sigma_c = \frac{138.3}{1.219} = 113.45 \text{ t/m}^2 \]

\[ \sigma_b = \frac{138.3}{0.635} = 217.81 \text{ t/m}^2 \]
RESULTANT STRESSES

a) Transfer stage

\[
\begin{align*}
\text{Self weight} & \quad 229.9 \quad -79.6 \quad 150.3 \\
\text{Jacking Stress} & \quad + \quad -79.6 \quad + \\
\text{Total Stress} & \quad + \quad -222.5 \quad 1107.7 \quad 885.2
\end{align*}
\]

b) Just after initial loss

\[
\begin{align*}
\text{Self wt. prestress after initial loss} & \quad 229.9 \quad -78.1 \quad 151.8 \\
\text{Net stress} & \quad + \quad -163.2 \quad (O.K.) \\
\text{Refer cl. 7.1. of IRC : 18 - 2000}
\end{align*}
\]

c) After intermediate loss

\[
\begin{align*}
\text{Self weight} & \quad 229.9 \quad -72.8 \quad 157.1 \\
\text{Net stress} & \quad + \quad -163.2 \quad (O.K.) \quad 792 < 1632 \text{ t/m}^2 \quad (O.K.)
\end{align*}
\]

d) After casting of Deck Slab

\[
\begin{align*}
\text{After Inter. Losses} & \quad 157.1 \quad 225.1 \quad 382.2 \\
\text{Cast in situ slab} & \quad > -163.2 \text{ t/m}^2 \quad (O.K.)
\end{align*}
\]

STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section = 1.44924207 m³
Section Modulus at junction of deck slab & girder Zj = 2.0039 m³
Section Modulus at C.G. of cables Zc = 1.615 m³
Section Modulus at bottom of composite section = 0.9048 m³
Composite area of deck slab + outer girder = 2.0159 m²

First stage cables
Prestressing force (after 1st stage), \( P_1 \) = 631.02 t
Eccentricity of cables from bottom, eb (1st stage) = 0.495 m

Second stage cables
Prestressing force (after friction and slip losses), \( P \) = 275.32 t
Eccentricity of cables from bottom, eb (2nd stage) = 0.960 m
Net eccentricity of cables of 1st and 2nd stages, \( e_b' \) = 0.636 m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, \( e' \) = 0.883 m (from c.g.)

B.M. due to S.Wt. of the girder \( M_g \) = 141.29 tm
B.M. due to Cast in Situ slab \( M_c \) = 82.37 tm
B.M. due to SIDL \( M_s \) = 81.97 tm
B.M. due to Live loads \( M_L \) = 89.10 tm
B.M. due to FPLL = 0.00 tm

Prestressing force (after 1st stage), \( P_1 \) = 631.02 t
Moment due to \( P \times e \) = 324.54 tm

Prestressing force (after friction and slip losses), \( P \) = 275.32 t
Moment due to \( P \times e \) = 154.03 tm

Stresses due to self weight of the girder + cast in situ slab
\[
\begin{align*}
\sigma_j &= \frac{223.66}{6.15} = 364.0 \text{ t/m}^2 \\
\sigma_c &= \frac{223.66}{1.219} = -183.4 \text{ t/m}^2 \\
\sigma_b &= \frac{223.66}{0.635} = -352.2 \text{ t/m}^2
\end{align*}
\]

Stresses due to prestress (second stage cables)
\[
\begin{align*}
\sigma_j &= \frac{275.32}{2.016} - \frac{154.03}{1.449} = 30.3 \text{ t/m}^2 \\
\sigma_c &= \frac{275.32}{2.016} + \frac{154.03}{2.004} = 59.7 \text{ t/m}^2 \\
\sigma_b &= \frac{275.32}{2.016} + \frac{154.03}{1.615} = 232.0 \text{ t/m}^2 \\
\sigma_b &= \frac{275.32}{2.016} + \frac{154.03}{0.905} = 306.8 \text{ t/m}^2
\end{align*}
\]

Total Stresses due to prestress
\[
\begin{align*}
\sigma_1 &= 0 + 30.3 = 30.3 \text{ t/m}^2 \\
\sigma_1 &= -72.8 + 59.7 = -13.1 \text{ t/m}^2 \\
\sigma_c &= 757.9 + 232.0 = 989.9 \text{ t/m}^2 \\
\sigma_b &= 1014.0 + 306.8 = 1320.8 \text{ t/m}^2
\end{align*}
\]

Stresses due to prestress & self weight of the girder+deck slab
\[
\begin{align*}
\sigma_1 &= 0 + 30.3 = 30.3 \text{ t/m}^2 \\
\sigma_1 &= 364.0 + -13.1 = 350.8 \text{ t/m}^2 \\
\sigma_c &= -183.4 + 989.9 = 806.5 \text{ t/m}^2 \\
\sigma_b &= -352.2 + 968.6 = 616.4 \text{ t/m}^2
\end{align*}
\]

Losses due to Elastic Shortening
\[
\text{Loss due to Elastic Shortening} = 0.5 \times \sigma \times E_s / E_c
\]
Therefore, initial loss = 2180 t/m²
(i.e. Elastic Shortening loss) = 18.07 t = 1.64 % of Jacking Force

For first stage cables
Prestressing force after initial losses \( P_{ii} \) = 625.86 t
Moment due to \( P_{ii} \times e \) = 321.79 tm

For second stage cables
Prestressing force after initial losses \( P_i \) = 270.16 t
Moment due to \( P_i \times e \) = 151.14 tm
Loss due to Shrinkage of Concrete  
\[ \varepsilon_s \times E_s \]
\[ = 3776.8 \text{ t/m}^2 \]
\[ = 31.31 \text{ t} = 2.84 \% \text{ of Jacking Force} \]

Loss due to Shrinkage  
\[ = 36.48 \text{ t} \]

Moment due to Shrinkage loss  
\[ = 32.217 \text{ tm} \]

**Loss due to relaxation of steel**

For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS  
\[ = 0.694 \]

Loss due to friction and slip  
\[ = 314.5861 - 275.3 = 39.3 \text{ t} \]

\% friction and slip loss  
\[ = \frac{39.3}{314.5861} \times 100 = 8.74 \% \]

Ratio of average force to UTS  
\[ = 0.694 - 0.087369 = 0.607 \]

Corresponding Relaxation Loss  
\[ = 5287.7 \text{ t/m}^2 \]
\[ = 12.53 \text{ t} \]

3.98 % of Jacking Force

Loss due to Relaxation & Shrinkage  
\[ = 17.69 \text{ t} \]

Moment due to Relaxation & Shrinkage loss  
\[ = 15.623 \text{ tm} \]

Shrinkage losses (for first stage cables)

\[ \sigma_t = \frac{-36.48}{2.016} + \frac{32.217}{1.449} = 4.14 \text{ t/m}^2 \]

\[ \sigma_j = \frac{-36.48}{2.016} + \frac{32.217}{2.004} = -2.02 \text{ t/m}^2 \]

\[ \sigma_c = \frac{-36.48}{2.016} - \frac{32.217}{1.615} = -38.04 \text{ t/m}^2 \]

\[ \sigma_b = \frac{-36.48}{2.016} - \frac{32.217}{0.905} = -53.70 \text{ t/m}^2 \]

Shrinkage and Relaxation losses (for second stage cables)

\[ \sigma_t = \frac{-17.69}{2.016} + \frac{15.623}{1.449} = 2.01 \text{ t/m}^2 \]

\[ \sigma_j = \frac{-17.69}{2.016} + \frac{15.623}{2.004} = -0.98 \text{ t/m}^2 \]

\[ \sigma_c = \frac{-17.69}{2.016} - \frac{15.623}{1.615} = -18.45 \text{ t/m}^2 \]

\[ \sigma_b = \frac{-17.69}{2.016} - \frac{15.623}{0.905} = -26.04 \text{ t/m}^2 \]

Total Shrinkage and Relaxation losses

\[ \sigma_t = 4.14 + 2.01 = 6.14 \text{ t/m}^2 \]

\[ \sigma_j = -2.02 + -0.98 = -2.99 \text{ t/m}^2 \]

\[ \sigma_c = -38.04 + -18.45 = -56.49 \text{ t/m}^2 \]

\[ \sigma_b = -53.70 + -26.04 = -79.74 \text{ t/m}^2 \]

Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder + cast insitu deck slab

\[ \sigma_t = 30.3 + 6.1 = 36.43 \text{ t/m}^2 \]

\[ \sigma_j = 350.8 + -3.0 = 347.83 \text{ t/m}^2 \]

\[ \sigma_c = 806.5 + -56.5 = 749.96 \text{ t/m}^2 \]

\[ \sigma_b = 968.6 + -79.7 = 888.91 \text{ t/m}^2 \]

**Loss due to Creep of concrete**

Arithmetic mean of initial and final stress after 28 days  
\[ = 749.96 \text{ t/m}^2 \]

Creep loss  
\[ = (\sigma/10) \times \varepsilon_c \times E_s \]
\[ = 5846.1 \text{ t/m}^2 \]
\[ = 48.47 \text{ t} = 4.402 \% \text{ of Jacking Force} \]
Loss due to Creep         = 48.47 t  
Moment due to Creep loss = 42.81 tm

**Creep losses (for first stage cables)**

\[
\sigma_t = \frac{-48.47}{2.016} + \frac{42.809}{1.449} = 5.50 \text{ t/m}^2
\]

\[
\sigma_j = \frac{-48.47}{2.016} + \frac{42.809}{2.004} = -2.68 \text{ t/m}^2
\]

\[
\sigma_c = \frac{-48.47}{2.016} - \frac{42.809}{1.615} = -50.55 \text{ t/m}^2
\]

\[
\sigma_b = \frac{-48.47}{2.016} - \frac{42.809}{0.905} = -71.35 \text{ t/m}^2
\]

**Creep losses (for second stage cables)**

\[
\sigma_t = \frac{-13.85}{2.016} + \frac{-12.231}{1.449} = -15.31 \text{ t/m}^2
\]

\[
\sigma_j = \frac{-13.85}{2.016} + \frac{-12.231}{2.004} = -12.97 \text{ t/m}^2
\]

\[
\sigma_c = \frac{-13.85}{2.016} - \frac{-12.231}{1.615} = 0.71 \text{ t/m}^2
\]

\[
\sigma_b = \frac{-13.85}{2.016} - \frac{-12.231}{0.905} = 6.65 \text{ t/m}^2
\]

**Final Prestressing Loss** = 759.26 t

**Total Creep losses**

\[
\sigma_t = \frac{5.50}{2.016} + \frac{-9.81}{1.449} = -9.81 \text{ t/m}^2
\]

\[
\sigma_j = \frac{-2.68}{2.016} + \frac{-15.31}{2.004} = -15.65 \text{ t/m}^2
\]

\[
\sigma_c = \frac{-50.55}{2.016} + \frac{0.71}{1.615} = -49.85 \text{ t/m}^2
\]

\[
\sigma_b = \frac{-71.35}{2.016} + \frac{6.65}{0.905} = -64.71 \text{ t/m}^2
\]

**Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab**

\[
\sigma_t = \frac{36.43}{1.449} + \frac{-9.81}{1.449} = 26.62 \text{ t/m}^2
\]

\[
\sigma_j = \frac{347.83}{2.004} + \frac{-15.65}{2.004} = 332.18 \text{ t/m}^2
\]

\[
\sigma_c = \frac{749.96}{1.615} + \frac{-49.85}{1.615} = 700.11 \text{ t/m}^2
\]

\[
\sigma_b = \frac{888.91}{0.905} + \frac{-64.71}{0.905} = 824.20 \text{ t/m}^2
\]

**Stresses Due to SIDL**

\[
\sigma_t = \frac{81.97}{1.449} = 56.6 \text{ t/m}^2
\]

\[
\sigma_j = \frac{81.97}{2.004} = 40.9 \text{ t/m}^2
\]

\[
\sigma_c = \frac{81.97}{1.615} = -50.8 \text{ t/m}^2
\]

\[
\sigma_b = \frac{81.97}{0.905} = -90.6 \text{ t/m}^2
\]

**Total Stresses due to prestress + DL (Girder+deck slab) + SIDL**

\[
\sigma_t = \frac{26.6}{1.449} + \frac{56.56}{1.449} = 83.2 \text{ t/m}^2
\]

\[
\sigma_j = \frac{332.2}{2.004} + \frac{40.91}{2.004} = 373.1 \text{ t/m}^2
\]

\[
\sigma_c = \frac{700.1}{1.615} + \frac{-50.76}{1.615} = 649.4 \text{ t/m}^2
\]

\[
\sigma_b = \frac{824.2}{0.905} + \frac{-90.59}{0.905} = 733.6 \text{ t/m}^2
\]
After laying of SIDL

Stresses due to Live load
\[
\sigma_l = 89.10 \quad = \quad 61.48 \quad \text{t/m}^2 \\
\sigma_j = 89.10 \quad = \quad 44.46 \quad \text{t/m}^2 \\
\sigma_c = 89.10 \quad = \quad -55.1773 \quad \text{t/m}^2 \\
\sigma_b = 89.10 \quad = \quad -98.47 \quad \text{t/m}^2
\]

Stresses due to Prestress + DL + SIDL + FPLL + LL
\[
\sigma_l = 83.18 \quad + \quad 61.5 \quad = \quad 144.7 \quad \text{t/m}^2 \\
\sigma_j = 373.08 \quad + \quad 44.5 \quad = \quad 417.5 \quad \text{t/m}^2 \\
\sigma_c = 849.35 \quad + \quad -55.2 \quad = \quad 594.2 \quad \text{t/m}^2 \\
\sigma_b = 733.61 \quad + \quad -98.5 \quad = \quad 635.1 \quad \text{t/m}^2
\]

Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)
\[
= 137.87 \quad \text{t} \\
20\% \text{ extra time dependant losses} \quad = \quad 27.57 \quad \text{t} \\
\text{Stress due to 20\% extra time dependant losses at top} \quad = \quad 3.1 \quad \text{t/m}^2 \\
\text{Stress due to 20\% extra time dependant losses at bottom} \quad = \quad -40.6 \quad \text{t/m}^2
STRESS CHECK AT EFF. DEPTH (SEC 5-5)

At first stage 3 cables has been stressed

STAGE -1 (14 TH DAY)

Net ecc. of cables from bottom, eb = 0.584 m
Eccentricity of cables from c.g., e = 0.465 m
Sectional Modulus at C.G. of cables, Zc = 1.491 m³
(for precast girder only)

B.M. due to S.Wt. of the girder M_s = 92.26 tm
B.M. due to Cast in Situ slab M_c = 88.01 tm
Prestressing force (after friction and slip losses), P = 681.31 t
Moment due to P, M_p = P x e = 316.78 tm

Stresses due to self weight of the girder
\[ \sigma_t = \frac{92.26}{0.659} = 139.90 \text{ t/m}^2 \]
\[ \sigma_c = \frac{92.26}{1.491} = -61.89 \text{ t/m}^2 \]
\[ \sigma_b = \frac{92.26}{0.661} = -139.61 \text{ t/m}^2 \]

Stresses due to prestress at transfer stage
\[ \sigma_t = \frac{681.31}{1.539} - \frac{316.78}{0.659} = -37.6 \text{ t/m}^2 \]
\[ \sigma_c = \frac{681.31}{1.539} + \frac{316.78}{1.491} = 655.3 \text{ t/m}^2 \]
\[ \sigma_b = \frac{681.31}{1.539} + \frac{316.78}{0.661} = 922.2 \text{ t/m}^2 \]

Stresses due to initial prestress & self weight of the girder
\[ \sigma_t = 139.9 + (-37.6) = 102.3 \text{ t/m}^2 \]
\[ \sigma_c = -61.9 + 655.3 = 593.4 \text{ t/m}^2 \]
\[ \sigma_b = -139.6 + 922.2 = 782.6 \text{ t/m}^2 \]

Losses due to Elastic Shortening
Loss due to Elastic Shortening = 0.5 x \( \sigma \times E_s/E_e \)

Therefore, initial loss = 1793 t/m²
(i.e. Elastic Shortening loss) = 10.62 t = 1.35 % of Jacking Force

Prestressing force after initial losses P_i = 670.69 t
Moment due to P_i = M_i = P_i x e = 311.85 tm

Loss due to Shrinkage of Concrete
\[ \varepsilon \times E_s \times 7.06 = 1192.7 \text{ t/m}^2 \]
= 0.90 % of Jacking Force
Loss due to relaxation of steel
The loss due to relaxation of steel depends on the stress in HTS after initial losses.
Ratio of average force to UTS = 0.691
Loss due to friction & slip = 786.4653 - 681.3 = 105.2 t
% friction and slip loss 0.09357 = 9.360 %
Ratio of average force to UTS = 0.691 - 0.093597 = 0.597
Corresponding Relaxation Loss = 4796.2 t/m²
= 28.40 t
= 3.61 % of Jacking Force
Loss due to Relaxation & Shrinkage = 35.47 t
Moment due to Relaxation & Shrinkage loss = 16.490 tm

Stresses due to Prestress after initial losses
\[ \sigma_i = \frac{670.69 - 311.85}{1.539} = -36.98 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-36.98}{0.659} = -56.9 \text{ t/m}^2 \]
\[ \sigma_b = \frac{670.69 + 311.85}{1.539} = 472.1 \text{ t/m}^2 \]

Stresses due to prestress after initial losses & self weight of the girder
\[ \sigma_i = \frac{139.9 - 37.0}{1.539} = 102.9 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-37.0}{0.659} = -56.4 \text{ t/m}^2 \]
\[ \sigma_b = \frac{139.9 + 907.8}{1.539} = 701.5 \text{ t/m}^2 \]

Stresses after Shrinkage and Relaxation losses
\[ \sigma_i = \frac{102.92 - 35.47 + 16.490}{1.539} = 702.0 \text{ t/m}^2 \]
\[ \sigma_c = \frac{645.1}{0.659} = 981.4 \text{ t/m}^2 \]
\[ \sigma_b = \frac{907.8}{1.491} = 616.1 \text{ t/m}^2 \]

Loss due to Creep of concrete
Airthmetic mean of initial and final stress between 14th and 28 days at C.G. of cables
Creep loss = (\( \sigma_i / 10 \)) \( e_i \) \( E_c \)
= 1213.6 t/m²
= 7.19 t
= 0.914 % of Jacking Force
Total Initial & Intermediate losses = 53.27 t = 0.91 % of Jacking Force (i.e. losses upto 28th day)
Force after intermediate losses, \( P_{int} \) = 628.0 t
Moment due to \( P_{int} \) = \( M_{int} = P_{int} \times e \) = 292.01 tm

Effect of prestress after Intermediate loss
\[ \sigma_i = \frac{628.0 - 292.01}{1.539} = -34.6 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-34.6}{0.659} = -52.7 \text{ t/m}^2 \]
\[ \sigma_b = \frac{628.0 + 292.01}{1.539} = 850.1 \text{ t/m}^2 \]

Stresses Due to cast-in-situ slab
\[ \sigma_i = \frac{88.0}{0.659} = 133.45 \text{ t/m}^2 \]
\[ \sigma_c = \frac{-59.04}{1.491} = -39.7 \text{ t/m}^2 \]
\[ \sigma_b = \frac{-133.18}{0.661} = -203.1 \text{ t/m}^2 \]
RESULTANT STRESSES

a) Transfer stage

\[
\begin{align*}
139.9 & \quad -37.6 & \quad 102.3 \\
\text{Self weight} & \quad + & \quad + \\
\text{Jacking Stress} & \quad + & \quad + \\
\text{Resultant} & \quad = & \quad 782.6
\end{align*}
\]

b) Just after initial loss

\[
\begin{align*}
139.9 & \quad -37.0 & \quad 102.9 & > -163.2 \text{ t/m}^2 \text{ (O.K.)} \\
\text{Self wt. prestress after initial loss} & \quad + & \quad + \\
\text{Net effect} & \quad = & \quad 768.2 < 1632 \text{ t/m}^2 \text{ (O.K.)} \\
\text{Refer cl. 7.1. Of IRC : 18 - 2000}
\end{align*}
\]

c) After intermediate loss

\[
\begin{align*}
139.9 & \quad -34.6 & \quad 105.3 & > -163.2 \text{ t/m}^2 \text{ (O.K.)} \\
\text{Self weight} & \quad + & \quad + \\
\text{Prestress after intermediate loss} & \quad + & \quad + \\
\text{Net effect} & \quad = & \quad 710 < 1632 \text{ t/m}^2 \text{ (O.K.)}
\end{align*}
\]

d) After casting of Deck Slab

\[
\begin{align*}
105.3 & \quad 133.5 & \quad 238.7 \\
\text{After Inter. Losses} & \quad - & \quad + \\
\text{Cast in situ slab} & \quad + & \quad + \\
\text{Net effect} & \quad = & \quad 577.3 \quad > -163.2 \text{ t/m}^2 \text{ (O.K.)}
\end{align*}
\]

STAGE-II (28 TH DAY ONWARD)

Section Modulus at top of composite section = 1.585883 3 m^3
Section Modulus at junction of deck slab & girder Z_j = 2.1970 m^3
Section Modulus at C.G. of cables Zc = 1.995 m^3
Section Modulus at bottom of composite section = 0.9822 m^3
Composite area of deck slab + outer girder = 2.3386 m^2

first stage cables
Prestressing force (after 1st stage), \( P_1 \) = 628.03 t
ecc. of cables from bottom, \( e_b \) (1st stage) = 0.584 m
Eccentricity of cables from c.g., \( e \) (1st stage) = 0.425 m
second stage cables
Prestressing force (after friction and slip losses), \( P \) = 272.14 t
ecc. of cables from bottom, \( e_b \) (2nd stage) = 1.090 m
Eccentricity of cables from c.g., \( e \) (2nd stage) = 0.429 m
Net eccentricity of cables of 1st and 2nd stages, \( e_b' \) = 0.737 m (from bottom)
Net eccentricity of cables of 1st and 2nd stages, \( e' \) = 0.782 m (from c.g.)
B.M. due to S.Wt. of the girder \( M_g \) = 92.26 tm
B.M. due to Cast in Situ slab \( M_c \) = 32.05 tm
B.M. due to SIDL \( M_s \) = 51.30 tm
B.M. due to Live loads \( M_L \) = 54.36 tm
B.M. due to FPLL = 0.00 tm

Prestressing force (after 1st stage), \( P_1 \) = 628.03 t
Moment due to \( P_{int} \) = \( M_{int} = P_1 \times e \) = 266.92 tm

For second stage prestressing
Prestressing force (after friction and slip losses), \( P \) = 272.14 t
Moment due to \( P \), \( M_p = P \times e \) = 116.85 tm

### Stresses due to self weight of the girder + cast in situ slab

\[
\sigma_j = \frac{124.31}{0.659} = 188.5 \quad \text{t/m}^2
\]
\[
\sigma_c = \frac{124.31}{1.491} = -83.4 \quad \text{t/m}^2
\]
\[
\sigma_b = \frac{124.31}{0.661} = -188.1 \quad \text{t/m}^2
\]

### Stresses due to prestress (second stage cables)

\[
\sigma_1 = \frac{272.14}{2.339} = 116.85 \quad \text{t/m}^2
\]
\[
\sigma_j = \frac{272.14}{1.586} = -63.2 \quad \text{t/m}^2
\]
\[
\sigma_c = \frac{272.14}{2.197} = 124.9 \quad \text{t/m}^2
\]
\[
\sigma_b = \frac{272.14}{1.995} = 135.3 \quad \text{t/m}^2
\]

### Total Stresses due to prestress

\[
\begin{align*}
\sigma_1 &= 0.0 + 42.7 = 42.7 \quad \text{t/m}^2 \\
\sigma_j &= -34.6 + 63.2 = 28.6 \quad \text{t/m}^2 \\
\sigma_c &= 604.1 + 174.9 = 779.0 \quad \text{t/m}^2 \\
\sigma_b &= 850.1 + 235.3 = 1085.4 \quad \text{t/m}^2
\end{align*}
\]

### Stresses due to prestress & self weight of the girder + deck slab

\[
\begin{align*}
\sigma_1 &= 0.0 + 42.7 = 42.7 \quad \text{t/m}^2 \\
\sigma_j &= -34.6 + 63.2 = 28.6 \quad \text{t/m}^2 \\
\sigma_c &= -83.4 + 779.0 = 695.6 \quad \text{t/m}^2 \\
\sigma_b &= -188.1 + 1085.4 = 897.3 \quad \text{t/m}^2
\end{align*}
\]

### Losses due to Elastic Shortening

Loss due to Elastic Shortening = 0.5 x \( \sigma \times E_s/E_c \)

Therefore, initial loss = 1880 t/m²

(i.e. Elastic Shortening loss) = 15.59 t = 1.42 % of Jacking Force

### For first stage cables

Prestressing force after initial losses \( P_{11} \) = 623.58 t
Moment due to \( P_{11} = M_{11} = P_{11} \times e \) = 265.02 tm

### For second stage cables

Prestressing force after initial losses \( P_i \) = 267.69 t
Moment due to \( P_i = M_{1i} = P_i \times e \) = 114.94 tm

### Loss due to Shrinkage of Concrete

\[
\begin{align*}
\epsilon_s \times E_s &= 3776.8 \quad \text{t/m}^2 \\
&= 31.31 \quad \text{t} = 2.84 \% \text{ of Jacking Force}
\end{align*}
\]
Loss due to Shrinkage = 35.77 t
Moment due to Shrinkage loss = 27.983 tm

**Loss due to relaxation of steel**

For second stage cables:
The loss due to relaxation of steel depends on the stress in HTS after initial losses.

Ratio of average force to UTS = 0.695
Loss due to friction and slip = 314.5861 - 272.1 = 42.4 t

\[
\text{% friction and slip loss} = \frac{42.4 \times 100}{314.5861} = 9.44 \%
\]

Corresponding Relaxation Loss = 4976.9 t/m²

\[
\text{Ratio of average force to UTS} = 0.695 - 0.0944 = 0.601
\]

Loss due to Relaxation & Shrinkage = 16.24 t
Moment due to Relaxation & Shrinkage loss = 12.708 tm

**Shrinkage losses (for first stage cables)**

\[
s_1 = -35.77 + 27.983 = 2.35 \text{ t/m}^2
\]

\[
s_2 = -35.77 + 27.983 = -2.56 \text{ t/m}^2
\]

\[
s_3 = -35.77 - 27.983 = -35.77 \text{ t/m}^2
\]

\[
s_4 = -35.77 - 27.983 = -43.78 \text{ t/m}^2
\]

**Shrinkage and Relaxation losses (for second stage cables)**

\[
s_1 = -16.24 + 12.708 = 1.07 \text{ t/m}^2
\]

\[
s_2 = -16.24 + 12.708 = -1.16 \text{ t/m}^2
\]

\[
s_3 = -16.24 - 12.708 = -29.32 \text{ t/m}^2
\]

\[
s_4 = -16.24 - 12.708 = -13.31 \text{ t/m}^2
\]

**Total Shrinkage and Relaxation losses**

\[
s_1 = 2.35 + 1.07 = 3.42 \text{ t/m}^2
\]

\[
s_2 = -2.56 + -1.16 = -3.72 \text{ t/m}^2
\]

\[
s_3 = -29.32 + -13.31 = -42.63 \text{ t/m}^2
\]

\[
s_4 = -43.78 + -19.88 = -63.67 \text{ t/m}^2
\]

**Stresses due to prestress after shrinkage & relaxation losses & self weight of the girder + cast insitu deck slab**

\[
\sigma_1 = 42.7 + 3.4 = 46.10 \text{ t/m}^2
\]

\[
\sigma_2 = 217.1 + -3.7 = 213.34 \text{ t/m}^2
\]

\[
\sigma_3 = 695.6 + -42.6 = 652.96 \text{ t/m}^2
\]

\[
\sigma_4 = 897.3 + -63.7 = 833.62 \text{ t/m}^2
\]

**Loss due to Creep of concrete**

Arithmetic mean of initial and final stress after 28 days = 652.96 t/m²

Creep loss = (\sigma/10) x e_c x E_s = 5090.0 t/m²

\[
\text{Creep loss} = 42.20 \text{ t} = 3.833 \% \text{ of Jacking Force}
\]

Loss due to Creep = 42.20 t
Moment due to Creep loss = 33.02 tm
Creep losses (for first stage cables)

\[
\begin{align*}
\sigma_t &= -42.20 + 33.016 = 2.77 \text{ t/m}^2 \\
\sigma_j &= -42.20 + 33.016 = -3.02 \text{ t/m}^2 \\
\sigma_c &= -42.20 - 33.016 = -34.59 \text{ t/m}^2 \\
\sigma_b &= -42.20 - 33.016 = -51.66 \text{ t/m}^2
\end{align*}
\]

Creep losses (for second stage cables)

\[
\begin{align*}
\sigma_t &= -12.06 - 9.433 = -21.49 \text{ t/m}^2 \\
\sigma_j &= -12.06 - 9.433 = -21.49 \text{ t/m}^2 \\
\sigma_c &= -12.06 + 9.433 = 2.63 \text{ t/m}^2 \\
\sigma_b &= -12.06 + 9.433 = 2.63 \text{ t/m}^2
\end{align*}
\]

Final Prestressing Force = 766.54 t

Total Creep losses

\[
\begin{align*}
\sigma_t &= 2.77 + -11.10 = -8.33 \text{ t/m}^2 \\
\sigma_j &= -3.02 + -9.45 = -12.47 \text{ t/m}^2 \\
\sigma_c &= -34.59 + -0.43 = -35.02 \text{ t/m}^2 \\
\sigma_b &= -51.66 + 4.45 = -47.21 \text{ t/m}^2
\end{align*}
\]

Total Stresses due to prestress after immediate losses & self weight of the girder +cast insitu deck slab

\[
\begin{align*}
\sigma_t &= 46.10 + -8.33 = 37.77 \text{ t/m}^2 \\
\sigma_j &= 213.34 + -12.47 = 199.87 \text{ t/m}^2 \\
\sigma_c &= 652.96 + -35.02 = 617.95 \text{ t/m}^2 \\
\sigma_b &= 833.62 + -47.21 = 786.41 \text{ t/m}^2
\end{align*}
\]

Stresses Due to SIDL

\[
\begin{align*}
\sigma_t &= 51.30 = 32.3 \text{ t/m}^2 \\
\sigma_j &= 51.30 = 23.4 \text{ t/m}^2 \\
\sigma_c &= 51.30 = -25.7 \text{ t/m}^2 \\
\sigma_b &= 51.30 = -52.2 \text{ t/m}^2
\end{align*}
\]

Total Stresses due to prestress + DL (Girder+deck slab) + SIDL

\[
\begin{align*}
\sigma_t &= 37.8 + 32.35 = 70.1 \text{ t/m}^2 \\
\sigma_j &= 200.9 + 23.35 = 224.2 \text{ t/m}^2 \\
\sigma_c &= 617.9 + -25.71 = 592.2 \text{ t/m}^2 \\
\sigma_b &= 786.4 + -52.23 = 734.2 \text{ t/m}^2
\end{align*}
\]
After laying of SIDL

Stresses due to Live load
\[ \sigma_t = 54.36 \text{ t/m}^2 \]
\[ \sigma_j = 54.36 \text{ t/m}^2 \]
\[ \sigma_c = 54.36 \text{ t/m}^2 \]
\[ \sigma_b = 54.36 \text{ t/m}^2 \]

Stresses due to Prestress + DL + SIDL + FPLL + LL
\[ \sigma_t = 70.12 \text{ t/m}^2 \]
\[ \sigma_j = 224.22 \text{ t/m}^2 \]
\[ \sigma_c = 592.24 \text{ t/m}^2 \]
\[ \sigma_b = 734.18 \text{ t/m}^2 \]

Time dependant losses (i.e. due to relaxation of steel, shrinkage of concrete & creep of concrete)
\[ = 127.95 \text{ t} \]
\[ = 25.59 \text{ t} \]
\[ = 1.7 \text{ t/m}^2 \]
\[ = -31.3 \text{ t/m}^2 \]
### SUMMARY OF MAXIMUM BENDING MOMENT & SHEAR FORCE

#### GIRDER G1:

**a) BENDING MOMENT (tm)**

<table>
<thead>
<tr>
<th>LOADING</th>
<th>SECTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-1</td>
</tr>
<tr>
<td>a) Self Weight of Girder</td>
<td>316.27</td>
</tr>
<tr>
<td>b) Deck Slab</td>
<td>268.46</td>
</tr>
<tr>
<td>c) SIDL</td>
<td>163.30</td>
</tr>
<tr>
<td>d) FPLL</td>
<td>0.00</td>
</tr>
<tr>
<td>e) LL</td>
<td>238.92</td>
</tr>
<tr>
<td>f) TOTAL</td>
<td>986.95</td>
</tr>
<tr>
<td>g) LL BM corresponding to Max S.F.case</td>
<td>198.15</td>
</tr>
</tbody>
</table>

**b) SHEAR FORCE (t)**

<table>
<thead>
<tr>
<th>LOADING</th>
<th>SECTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-1</td>
</tr>
<tr>
<td>a) Self Weight of Girder</td>
<td>0.00</td>
</tr>
<tr>
<td>b) Deck Slab</td>
<td>1.49</td>
</tr>
<tr>
<td>c) SIDL</td>
<td>2.00</td>
</tr>
<tr>
<td>d) FPLL</td>
<td>0.00</td>
</tr>
<tr>
<td>e) LL</td>
<td>12.22</td>
</tr>
<tr>
<td>f) TOTAL</td>
<td>15.71</td>
</tr>
<tr>
<td>g) LL Shear corresponding to max BM case</td>
<td>7.65</td>
</tr>
</tbody>
</table>

### ULTIMATE BENDING MOMENT AND SHEAR FORCE

**a) Maximum Bending Moment & Corresponding Shear Force**

<table>
<thead>
<tr>
<th>Section</th>
<th>1-1</th>
<th>2-2</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
<th>5-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25DL</td>
<td>730.9</td>
<td>682.1</td>
<td>682.1</td>
<td>549.4</td>
<td>319.1</td>
<td>205.9</td>
</tr>
<tr>
<td>2SIDL</td>
<td>326.6</td>
<td>329.5</td>
<td>329.5</td>
<td>253.1</td>
<td>163.9</td>
<td>102.6</td>
</tr>
<tr>
<td>2.5LL</td>
<td>597.3</td>
<td>519.0</td>
<td>519.0</td>
<td>437.8</td>
<td>222.8</td>
<td>135.9</td>
</tr>
<tr>
<td>TOTAL $M_u$</td>
<td>1654.8</td>
<td>1530.5</td>
<td>1530.5</td>
<td>1240.3</td>
<td>705.8</td>
<td>444.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
<th>1.25DL</th>
<th>2SIDL</th>
<th>2.5LL</th>
<th>TOTAL $V_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.9</td>
<td>4.0</td>
<td>19.1</td>
<td>25.0</td>
<td></td>
</tr>
<tr>
<td>24.2</td>
<td>17.2</td>
<td>22.4</td>
<td>63.8</td>
<td></td>
</tr>
<tr>
<td>24.2</td>
<td>17.2</td>
<td>54.9</td>
<td>96.3</td>
<td></td>
</tr>
<tr>
<td>50.3</td>
<td>20.6</td>
<td>54.9</td>
<td>125.7</td>
<td></td>
</tr>
<tr>
<td>72.6</td>
<td>40.7</td>
<td>59.7</td>
<td>173.0</td>
<td></td>
</tr>
<tr>
<td>82.4</td>
<td>43.2</td>
<td>59.4</td>
<td>185.0</td>
<td></td>
</tr>
</tbody>
</table>

**b) Maximum Shear force and Corresponding Bending Moment**

<table>
<thead>
<tr>
<th>SHEAR FORCE (t)</th>
<th>SECTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-1</td>
</tr>
<tr>
<td>1.25DL</td>
<td>1.9</td>
</tr>
<tr>
<td>2SIDL</td>
<td>4.0</td>
</tr>
<tr>
<td>2.5LL</td>
<td>30.6</td>
</tr>
<tr>
<td>TOTAL $V_u$</td>
<td>36.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>BENDING MOMENTS CORRESPONDING TO MAX SHEAR (tm)</th>
<th>SECTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25DL</td>
<td>730.9</td>
</tr>
<tr>
<td>2SIDL</td>
<td>326.6</td>
</tr>
<tr>
<td>2.5LL</td>
<td>495.4</td>
</tr>
<tr>
<td>TOTAL $M_u$</td>
<td>1552.9</td>
</tr>
</tbody>
</table>
### SECTION PROPERTIES

<table>
<thead>
<tr>
<th>Section</th>
<th>1-1</th>
<th>2-2</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
<th>5-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sectional area, A m²</td>
<td>1.906</td>
<td>1.906</td>
<td>1.906</td>
<td>1.906</td>
<td>2.016</td>
<td>2.239</td>
</tr>
<tr>
<td>Moment of Inertia, I m⁴</td>
<td>1.314</td>
<td>1.314</td>
<td>1.314</td>
<td>1.314</td>
<td>1.309</td>
<td>1.362</td>
</tr>
<tr>
<td>C.G. of section from top, Yt m</td>
<td>0.831</td>
<td>0.831</td>
<td>0.831</td>
<td>0.831</td>
<td>0.903</td>
<td>0.933</td>
</tr>
<tr>
<td>C.G. of section from bottom, Yb m</td>
<td>1.519</td>
<td>1.519</td>
<td>1.519</td>
<td>1.519</td>
<td>1.447</td>
<td>1.417</td>
</tr>
<tr>
<td>Sectional modulus at top, Zt m³</td>
<td>1.581</td>
<td>1.581</td>
<td>1.581</td>
<td>1.581</td>
<td>1.449</td>
<td>1.460</td>
</tr>
<tr>
<td>Sectional modulus at bottom, Zb m³</td>
<td>0.865</td>
<td>0.865</td>
<td>0.865</td>
<td>0.865</td>
<td>0.905</td>
<td>0.962</td>
</tr>
<tr>
<td>Width of the web, b m</td>
<td>0.275</td>
<td>0.275</td>
<td>0.275</td>
<td>0.275</td>
<td>0.375</td>
<td>0.6</td>
</tr>
</tbody>
</table>

| Overall Depth of Section, d m | 2.35 |

<table>
<thead>
<tr>
<th>Total No. of cables</th>
<th>7</th>
<th>7</th>
<th>7</th>
<th>7</th>
<th>7</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final prestressing force, Pₚ m</td>
<td>775.57</td>
<td>809.47</td>
<td>809.47</td>
<td>761.88</td>
<td>759.26</td>
<td>766.54</td>
</tr>
<tr>
<td>Eccentricity from bottom, eₚ m</td>
<td>0.332</td>
<td>0.363</td>
<td>0.363</td>
<td>0.451</td>
<td>0.636</td>
<td>0.737</td>
</tr>
</tbody>
</table>

Average thickness of compression flange, t = 0.347 m  
Bₜ = 3200 mm

### ULTIMATE MOMENT CAPACITY

As per Clause 13 of IRC:18 - 2000

<table>
<thead>
<tr>
<th>Section</th>
<th>1-1</th>
<th>2-2</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
<th>5-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mₙₑₛ* = 0.9 dₚ Aₛ fₚ</td>
<td>2856.2</td>
<td>2813.2</td>
<td>2813.2</td>
<td>2688.9</td>
<td>2426.3</td>
<td>2283.5</td>
</tr>
<tr>
<td>Mₙₑₛ** = 0.176 b dₚ² fₚ + 2/3 x 0.8 (Bₜ - b)(dₚ - t/2) x 1 x fₚ</td>
<td>Part1</td>
<td>803.8</td>
<td>779.8</td>
<td>779.8</td>
<td>712.4</td>
<td>922.9</td>
</tr>
<tr>
<td>Part2</td>
<td>4069.8</td>
<td>4002.8</td>
<td>4002.8</td>
<td>3809.0</td>
<td>3210.9</td>
<td>2824.2</td>
</tr>
<tr>
<td>Total</td>
<td>4873.6</td>
<td>4782.6</td>
<td>4782.6</td>
<td>4521.4</td>
<td>4133.7</td>
<td>3945.2</td>
</tr>
<tr>
<td>Mₑₛ = min(Mₑₛ, and Mₑₛ)</td>
<td>2856.2</td>
<td>2813.2</td>
<td>2813.2</td>
<td>2688.9</td>
<td>2426.3</td>
<td>2283.5</td>
</tr>
<tr>
<td>Mₑ = 1654.8</td>
<td>1530.5</td>
<td>1530.5</td>
<td>1240.3</td>
<td>705.8</td>
<td>444.4</td>
<td></td>
</tr>
</tbody>
</table>

* Mₑₛ: Moment capacity due to yield of steel.  
** Mₑₛ: Moment capacity due to crushing of concrete.

### CALCULATION OF DEPTH OF TENSILE ZONE

<table>
<thead>
<tr>
<th>Section</th>
<th>1-1</th>
<th>2-2</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
<th>5-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>σₜ = 0.8 x (Pₚ/A - Mₑₚ/Zₑ) + Mₑₚ/Zₑ m²</td>
<td>1666.1</td>
<td>1645.2</td>
<td>1645.2</td>
<td>1608.3</td>
<td>1635.7</td>
<td>1552.9</td>
</tr>
<tr>
<td>σₜ = 0.8 x (Pₚ/A+Mₑₚ/Zₑ) - Mₑₚ/Zₑ m²</td>
<td>-2126.3</td>
<td>-2047.9</td>
<td>-2047.9</td>
<td>-2036.9</td>
<td>-1836.0</td>
<td>-1667.3</td>
</tr>
<tr>
<td>Depth of tensile zone, z m</td>
<td>1.318</td>
<td>1.303</td>
<td>1.303</td>
<td>1.313</td>
<td>1.243</td>
<td>1.217</td>
</tr>
<tr>
<td>C.G. of cables in tensile zone from bottom, eₚ m</td>
<td>0.332</td>
<td>0.363</td>
<td>0.363</td>
<td>0.451</td>
<td>0.636</td>
<td>0.737</td>
</tr>
<tr>
<td>eₚ = (D - eₚ_max) but not &lt; 0.8D m</td>
<td>2.018</td>
<td>1.987</td>
<td>1.987</td>
<td>1.899</td>
<td>1.880</td>
<td>1.880</td>
</tr>
</tbody>
</table>
**SHEAR CAPACITY AND SHEAR REINFORCEMENT**

\( f_t = \text{Maximum principal tensile stress} \)
\[ = 0.24f_{ck} = 154.83 \text{ t/m}^2 \text{ (As per Clause 14.1.2 of IRC:18)} \]

\( f_{cp} = \text{Stress due to prestress only at centroidal axis of composite section} \)
\( f_{pt} = \text{Stress due to prestress only at tensile fibre of composite section} \)
\( d = \text{Depth from extreme compression fibre either to the longitudinal bars or to the centroid of the tendons whichever is greater} \)
\[ = d - 0.060 = 2.290 \text{ m} \]

Maximum Ultimate shear stress = 479.4 t/m\(^2\) (As per Table 6 of IRC:18)

<table>
<thead>
<tr>
<th>b = web thickness = 275 - 75 x 2/3</th>
<th>units</th>
<th>1-1</th>
<th>2-2</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
<th>5-5</th>
</tr>
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<tbody>
<tr>
<td>m</td>
<td></td>
<td>0.225</td>
<td>0.225</td>
<td>0.225</td>
<td>0.225</td>
<td>0.388</td>
<td>0.550</td>
</tr>
<tr>
<td>( f_{cp} )</td>
<td>t/m(^2)</td>
<td>406.86</td>
<td>424.64</td>
<td>424.64</td>
<td>399.68</td>
<td>376.64</td>
<td>342.41</td>
</tr>
<tr>
<td>( V_{co} = 0.67bd\sqrt{(f_t^2 + 0.8f_{cp})} )</td>
<td>t</td>
<td>96.6</td>
<td>98.0</td>
<td>98.0</td>
<td>96.0</td>
<td>162.1</td>
<td>223.1</td>
</tr>
<tr>
<td>( d_b = d - e_b ) m</td>
<td></td>
<td>2.018</td>
<td>1.987</td>
<td>1.987</td>
<td>1.899</td>
<td>1.880</td>
<td>1.880</td>
</tr>
<tr>
<td>( M_t = (0.37f_{ck} + 0.8f_{pt})I/y )</td>
<td>t/m(^2)</td>
<td>1224.2</td>
<td>1249.0</td>
<td>1249.0</td>
<td>1134.2</td>
<td>981.0</td>
<td>909.8</td>
</tr>
<tr>
<td>( V_{cr} = 0.037bd\sqrt{f_{ck} + (M_t/Mu)} )</td>
<td>t</td>
<td>39.5</td>
<td>73.7</td>
<td>73.7</td>
<td>127.9</td>
<td>258.3</td>
<td>407.8</td>
</tr>
<tr>
<td>( V_{crmin} = 0.1 bd\sqrt{f_{ck}} )</td>
<td>t</td>
<td>34.1</td>
<td>34.1</td>
<td>34.1</td>
<td>34.1</td>
<td>58.7</td>
<td>83.4</td>
</tr>
<tr>
<td>( V_{cr} = \max(V_c, V_{crmin}) )</td>
<td>t</td>
<td>39.5</td>
<td>73.7</td>
<td>73.7</td>
<td>127.9</td>
<td>258.3</td>
<td>407.8</td>
</tr>
<tr>
<td>( V_c = \min(V_{co}, V_{cr}) )</td>
<td>t</td>
<td>39.5</td>
<td>73.7</td>
<td>73.7</td>
<td>96.0</td>
<td>162.1</td>
<td>223.1</td>
</tr>
<tr>
<td>( V_u = 0.037bd\sqrt{f_{ck} + (M_t/Mu)} )</td>
<td>t</td>
<td>36.4</td>
<td>75.7</td>
<td>75.7</td>
<td>127.5</td>
<td>173.2</td>
<td>186.8</td>
</tr>
</tbody>
</table>

**Remarks**

\( V_{uo} = \text{Uncracked shear capacity of the section} \)
\( V_u = \text{Cracked shear capacity} \)
\( V_c = \text{Shear capacity} \)
\( V_u = \text{Ultimate shear} \)
\( V_{cr} = \text{Shear reinforcement} \)
\( f_{ty} = \text{Yield strength of shear reinforcement} \)
\( v_u = \text{Shear stress} \)

\( dt = \text{Depth from extreme compression fibre either to the longitudinal bars or to the centroid of the tendons whichever is greater} \)
### ULTIMATE TORSION & SHEAR

#### GIRDER G

**CASE 1 ** MAX. TORSIONAL MOMENT (t.m) & CORR. SHEAR FORCE(t)

<table>
<thead>
<tr>
<th>SEC.</th>
<th>DL</th>
<th>1.25 DL</th>
<th>SIDL</th>
<th>2.0 SIDL</th>
<th>LL</th>
<th>FPLL</th>
<th>+ FPLL</th>
<th>CORR. SHEAR FORCE</th>
<th>2.5 (LL*I.F.)</th>
<th>ULT</th>
<th>1.25 DL</th>
<th>SIDL</th>
<th>2.0 SIDL</th>
<th>LL</th>
<th>FPLL</th>
<th>+ FPLL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
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<td>0.00</td>
<td>2.88</td>
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<td>0.54</td>
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<td>7.12</td>
<td>1.49</td>
<td>1.86</td>
<td>2.00</td>
<td>4.00</td>
<td>4.20</td>
<td>0.00</td>
<td>10.50</td>
<td>16.36</td>
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<tr>
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<td>0.00</td>
<td>3.10</td>
<td>6.20</td>
<td>0.65</td>
<td>0.00</td>
<td>1.62</td>
<td>7.82</td>
<td>19.36</td>
<td>24.20</td>
<td>8.60</td>
<td>17.20</td>
<td>2.77</td>
<td>0.00</td>
<td>6.93</td>
<td>48.33</td>
</tr>
<tr>
<td>3-3</td>
<td>0.00</td>
<td>0.00</td>
<td>1.74</td>
<td>3.48</td>
<td>2.02</td>
<td>0.00</td>
<td>5.04</td>
<td>8.52</td>
<td>40.20</td>
<td>50.25</td>
<td>10.30</td>
<td>20.60</td>
<td>19.88</td>
<td>0.00</td>
<td>49.70</td>
<td>120.55</td>
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<td>0.00</td>
<td>0.00</td>
<td>5.73</td>
<td>11.46</td>
<td>2.81</td>
<td>0.00</td>
<td>7.03</td>
<td>18.49</td>
<td>58.07</td>
<td>72.59</td>
<td>20.35</td>
<td>40.70</td>
<td>19.38</td>
<td>0.00</td>
<td>48.45</td>
<td>161.74</td>
</tr>
<tr>
<td>5-5</td>
<td>0.00</td>
<td>0.00</td>
<td>5.73</td>
<td>11.46</td>
<td>2.81</td>
<td>0.00</td>
<td>7.03</td>
<td>18.49</td>
<td>65.94</td>
<td>82.43</td>
<td>21.61</td>
<td>43.22</td>
<td>19.38</td>
<td>0.00</td>
<td>48.45</td>
<td>174.10</td>
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</table>

**CASE 2 ** MAXIMUM SHEAR FORCE (t) & CORR. TORSIONAL MOMENT(t.m)

<table>
<thead>
<tr>
<th>SEC.</th>
<th>DL</th>
<th>1.25 DL</th>
<th>SIDL</th>
<th>2.0 SIDL</th>
<th>LL</th>
<th>FPLL</th>
<th>+ FPLL</th>
<th>CORR. TORSIONAL MOMENT</th>
<th>2.5 (LL*I.F.)</th>
<th>ULT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>1.49</td>
<td>1.86</td>
<td>2.00</td>
<td>4.00</td>
<td>11.53</td>
<td>0.00</td>
<td>28.83</td>
<td>34.69</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2-2</td>
<td>19.36</td>
<td>24.20</td>
<td>8.60</td>
<td>17.20</td>
<td>12.16</td>
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<td>30.40</td>
<td>71.80</td>
<td>0.00</td>
<td>0.00</td>
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<td>40.20</td>
<td>50.25</td>
<td>10.30</td>
<td>20.60</td>
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<td>0.00</td>
<td>50.73</td>
<td>121.58</td>
<td>0.00</td>
<td>0.00</td>
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<td>4-4</td>
<td>58.07</td>
<td>72.59</td>
<td>20.35</td>
<td>40.70</td>
<td>20.50</td>
<td>0.00</td>
<td>51.25</td>
<td>164.54</td>
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<td>21.61</td>
<td>43.22</td>
<td>20.95</td>
<td>0.00</td>
<td>52.38</td>
<td>178.02</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

#### CHECK FOR TOTAL ULTIMATE SHEAR STRESS

Each component Rectangle will be subjected to a Torque = T(hmax * hmin^3)/(C(hmax * hmin^3)) = T x C (say)

<table>
<thead>
<tr>
<th>Component Rectangle</th>
<th>hmax</th>
<th>hmin</th>
<th>hmax * hmin^3</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3200</td>
<td>250</td>
<td>5.00E+10</td>
<td>0.30</td>
</tr>
<tr>
<td>2</td>
<td>1250</td>
<td>230</td>
<td>1.52E+10</td>
<td>0.09</td>
</tr>
<tr>
<td>3</td>
<td>1450</td>
<td>275</td>
<td>3.02E+10</td>
<td>0.18</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>420</td>
<td>7.41E+10</td>
<td>0.44</td>
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</tbody>
</table>

Σ 1.69E+11
Torsional Shear Stress = \( \frac{2T}{(h_{\text{min}})^2/(h_{\text{max}}-h_{\text{min}}/3)} \)

**CASE 1** MAX. TORSIONAL MOMENT & CORR. SHEAR FORCE

<table>
<thead>
<tr>
<th>SEC.</th>
<th>MAX. TOR. MOM. (t.m)</th>
<th>MOMENT IN RECTANGULAR COMPONENT (t.m)</th>
<th>TOR. SHEAR STRESS IN COMPONENT RECTANGLES OF GIRDER</th>
<th>CORR. S.F. (t)</th>
<th>ULTIMATE SHEAR STRESS DUE TO S.F.</th>
<th>TOTAL ULTIMATE SHEAR STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
</tr>
<tr>
<td>1-1</td>
<td>7.12</td>
<td>2.10</td>
<td>0.64</td>
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<td>3.11</td>
<td>21.6</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-2</td>
<td>7.82</td>
<td>2.31</td>
<td>0.70</td>
<td>1.39</td>
<td>3.42</td>
<td>23.7</td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-3</td>
<td>8.52</td>
<td>2.51</td>
<td>0.76</td>
<td>1.52</td>
<td>3.73</td>
<td>25.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-4</td>
<td>18.49</td>
<td>5.45</td>
<td>1.66</td>
<td>3.29</td>
<td>8.08</td>
<td>56.0</td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-5</td>
<td>18.49</td>
<td>5.45</td>
<td>1.66</td>
<td>3.29</td>
<td>8.08</td>
<td>56.0</td>
</tr>
</tbody>
</table>

**PERMISSIBLE VALUE OF TOTAL ULT. SHEAR STR** = 484.5 t/m²

**CASE 2** MAX. SHEAR FORCE & CORR. TORSIONAL MOMENT

<table>
<thead>
<tr>
<th>SEC.</th>
<th>CORR. TOR. MOM. (t.m)</th>
<th>MOMENT IN COMPONENT RECTANGLES (t.m)</th>
<th>TOR. SHEAR STRESS IN COMPONENT RECTANGLES OF GIRDER</th>
<th>MAX. S.F. (t)</th>
<th>ULTIMATE SHEAR STRESS DUE TO S.F.</th>
<th>TOTAL ULTIMATE SHEAR STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>1-1</td>
<td>5.99</td>
<td>1.77</td>
<td>0.54</td>
<td>1.07</td>
<td>2.62</td>
<td>18.13</td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-2</td>
<td>6.90</td>
<td>2.04</td>
<td>0.62</td>
<td>1.23</td>
<td>3.02</td>
<td>20.90</td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-3</td>
<td>5.21</td>
<td>1.34</td>
<td>0.47</td>
<td>0.93</td>
<td>2.28</td>
<td>15.77</td>
</tr>
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<td></td>
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<tr>
<td>4-4</td>
<td>14.16</td>
<td>4.18</td>
<td>1.27</td>
<td>2.52</td>
<td>6.19</td>
<td>42.90</td>
</tr>
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</tr>
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<td>1.29</td>
<td>2.56</td>
<td>6.29</td>
<td>43.58</td>
</tr>
</tbody>
</table>

CALCULATION OF TORSION REINFORCEMENT

**AS PER CLAUSE 14.2.3.1 OF IRC : 18 - 2000**

**REINFORCEMENT FOR TORSION**

\[ \text{Asv/Sv} \geq T / (0.8 \times X1 \times Y1 \times 0.87 \times Fyv) \]

**AREA OR LONG. REINFORCEMENT**

\[ \text{As} \geq \text{Asv/Sv} \times (X1 + Y1) \times (\text{Fyv} / \text{Fyl}) \]

\[ \text{Fyv} \text{ (t/m²)} = 42330 \]

\[ X1 \text{ is the smaller dimension of the link measured between centres of legs} \]

\[ Y1 \text{ is the larger dimension of the link measured between centres of legs} \]

According to clause 14.2.2 of IRC-18:2000, torsion reinforcement is not required if \( Vtu < 42.75 \text{ t/m²} \)

\[ 42.75 \text{ t/m²} \]
### Bridge Design Report

#### COMPONENT RECTANGLES

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_{\text{max}})</td>
<td>3.2</td>
<td>1.25</td>
<td>1.45</td>
<td>1</td>
</tr>
<tr>
<td>(H_{\text{min}})</td>
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<td>0.23</td>
<td>0.275</td>
<td>0.42</td>
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</table>

#### CASE 1  MAX. TORSIONAL MOMENT & CORR. SHEAR FORCE

**MAX. TOR. MOM. (t)**

<table>
<thead>
<tr>
<th>SEC</th>
<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{\text{max}})</td>
<td>7.12</td>
<td>7.82</td>
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<td>18.49</td>
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<tr>
<td>(M_{\text{min}})</td>
<td>0.25</td>
<td>0.70</td>
<td>0.76</td>
<td>1.66</td>
</tr>
</tbody>
</table>

**MOMENT IN COMPONENT RECTANGLES (t.m)**

<table>
<thead>
<tr>
<th>SEC</th>
<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{\text{max}})</td>
<td>2.10</td>
<td>2.31</td>
<td>2.51</td>
<td>5.45</td>
</tr>
<tr>
<td>(M_{\text{min}})</td>
<td>0.64</td>
<td>0.70</td>
<td>0.76</td>
<td>1.66</td>
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</table>

**TORSIONAL MOMENT**

<table>
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<tr>
<th>SEC</th>
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<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{\text{max}})</td>
<td>3.11</td>
<td>3.42</td>
<td>3.73</td>
<td>NOT REQ.</td>
</tr>
<tr>
<td>(M_{\text{min}})</td>
<td>N.R.</td>
<td>N.R.</td>
<td>N.R.</td>
<td>NOT REQ.</td>
</tr>
</tbody>
</table>

**AREA OF LONG. REINFORCEMENT**

<table>
<thead>
<tr>
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<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A_{\text{sv}}/S_{\text{v}}) (cm²/m)</td>
<td>2.49</td>
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<td>6.09</td>
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<tr>
<td>(A_{\text{sv}}/S_{\text{v}}) (cm²)</td>
<td>2.49</td>
<td>3.80</td>
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<td>6.09</td>
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</table>

#### CASE 2  MAX. SHEAR FORCE & CORR. TORSIONAL MOMENT

**MAX. TOR. MOM. (t)**

<table>
<thead>
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<th>SEC</th>
<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{\text{max}})</td>
<td>5.99</td>
<td>6.90</td>
<td>5.21</td>
<td>14.16</td>
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<tr>
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<td>1.54</td>
<td>4.18</td>
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**MOMENT IN COMPONENT RECTANGLES (t.m)**

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<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
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<td>1.54</td>
<td>0.62</td>
<td>1.52</td>
<td>4.16</td>
</tr>
<tr>
<td>(M_{\text{min}})</td>
<td>0.54</td>
<td>1.07</td>
<td>0.76</td>
<td>1.27</td>
</tr>
</tbody>
</table>

**TORSIONAL MOMENT**

<table>
<thead>
<tr>
<th>SEC</th>
<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{\text{max}})</td>
<td>NOT REQ.</td>
<td>NOT REQ.</td>
<td>NOT REQ.</td>
<td>9.71</td>
</tr>
<tr>
<td>(M_{\text{min}})</td>
<td>NOT REQ.</td>
<td>NOT REQ.</td>
<td>NOT REQ.</td>
<td>NOT REQ.</td>
</tr>
</tbody>
</table>

**AREA OF LONG. REINFORCEMENT**

<table>
<thead>
<tr>
<th>SEC</th>
<th>1-1</th>
<th>2-2</th>
<th>3-3</th>
<th>4-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A_{\text{sv}}/S_{\text{v}}) (cm²/m)</td>
<td>2.49</td>
<td>3.80</td>
<td>4.29</td>
<td>6.09</td>
</tr>
<tr>
<td>(A_{\text{sv}}/S_{\text{v}}) (cm²)</td>
<td>2.49</td>
<td>3.80</td>
<td>4.29</td>
<td>6.09</td>
</tr>
</tbody>
</table>

#### TOTAL R/F IN WEB

<table>
<thead>
<tr>
<th>SEC</th>
<th>(S_{\text{v}})</th>
<th>(\text{Torsion})</th>
<th>(\text{Total})</th>
<th>(S_{\text{v}})</th>
<th>(\text{Torsion})</th>
<th>(\text{Total})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>2.49</td>
<td>0.00</td>
<td>2.49</td>
<td>FALSE</td>
<td>2.49</td>
<td>FALSE</td>
</tr>
<tr>
<td>2-2</td>
<td>3.80</td>
<td>0.00</td>
<td>3.80</td>
<td>FALSE</td>
<td>3.80</td>
<td>FALSE</td>
</tr>
<tr>
<td>3-3</td>
<td>4.29</td>
<td>NOT REQ.</td>
<td>4.29</td>
<td>FALSE</td>
<td>4.29</td>
<td>FALSE</td>
</tr>
<tr>
<td>4-4</td>
<td>6.09</td>
<td>4.49</td>
<td>6.09</td>
<td>3.44</td>
<td>9.54</td>
<td></td>
</tr>
<tr>
<td>5-5</td>
<td>6.09</td>
<td>4.49</td>
<td>10.59</td>
<td>3.50</td>
<td>9.59</td>
<td></td>
</tr>
</tbody>
</table>
**DESIGN OF SHEAR CONNECTOR**

Reference: clause 611.4.2 of IRC : 22 - 1986 code for composite construction

The ultimate longitudinal shear $V_{L}$ per unit length,

$$V_{L} = \frac{V \cdot Ac \cdot Y}{I}$$

Where,

$V = \text{Vertical shear due to dead load placed after composite section is effective and working live load with impact.}$

$$= 21.61 \times 1.5 + 24.5 \times 2.5$$

$$= 93.57 \text{ t}$$

$Ac = \text{Area of transformed section on one side of interface}$

$$= 0.25 \times 3.2 \quad (\text{refer section property})$$

$$= 0.8 \text{ m}^2$$

$Y = \text{Dist. Of centroid of the area under consideration from the neutral axis of the composite section}$

$$= 0.89877 - 0.125 \quad (\text{refer section property})$$

$$= 0.77377 \text{ m}$$

$I = \text{M. I. of the composite section}$

$$= 1.42534 \times 10^4 \text{ mm}^4 \quad (\text{refer section property})$$

$$= 1.42534 \text{ m}^4$$

The ultimate longitudinal shear $V_{L}$ per unit length,

$$= \frac{93.57 \times 0.8 \times 0.7738}{1.42534}$$

$$= 40.63 \text{ t/m}$$

$$\Sigma Q_u = A_s f_u \times 0.001$$

Where,

$Q_u = \text{The ultimate shear resisting capacity of anchorage connector in kN}$

$A_s = \text{The c / s. Area of the anchorage connector in sq mm}$

$f_u = \text{The ultimate tensile strength of anchorage connector in Mpa}$

$$\Sigma Q_u = \left[ 2 \times 113 \times \frac{1000}{125} \times 415 \right]$$

$$= 750.32 \text{ kN}$$

$$= 75.0 \text{ t}$$

$$> 40.63 \text{ t} \quad \text{O.K.}$$

Provide 2L - 12 @ 125 c / c in addition to shear reinforcement.
DESIGN OF DECK SLAB
DESIGN OF DECK SLAB

The Deck slab Panel is designed as a Two way slab spanning between the main girders and Cross diaphragm, it is continuous.

C/c of main girders = 2.8 m  
Web width of main girders = 0.275 m  
Clear span (lx) between main girders = 2.525 m  
C/c of Cross diaphragms = 7.5 m  
Clear span (ly) between cross diaphragms = 7.225 m

Ratio of span k = ly/lx = 2.86 > 2.0

The behaviour of the deck slab panel is considered to be as Oneway slab. And will be analysed and designed accordingly.

ANALYSIS AND DESIGN

The deck slab panel will be analysed using the tables given in "DESIGN TABLES FOR CONCRETE BRIDGE DECK SLAB". A handbook published by SERC, ROORKEE both for the effects of dead load and concentrated strip loads due to IRC loadings.

Pigueads method will be adopted for evaluating the design moments.

Panel dimension = 2.525 x 7.225 = 18.24 m²  
Slab thickness = 250 mm  
Wearing coat thickness = 56 mm

Key charts corresponding to the nearest panel size,

<table>
<thead>
<tr>
<th>Panel size</th>
<th>Key chart No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.500 x 7</td>
<td>207</td>
</tr>
<tr>
<td>2.5 x 7.25</td>
<td>224</td>
</tr>
<tr>
<td>2.75 x 7</td>
<td>208</td>
</tr>
<tr>
<td>2.75 x 7.25</td>
<td>225</td>
</tr>
</tbody>
</table>

Dead Load on the Panel

Dead load due to Deck slab = 0.25 x 18.24 x 25 = 114.0 kN  
Dead load due to Wearing coat = 0.056 x 18.24 x 22 = 22.5 kN  
Total load (W) = \( \frac{136.5 \text{ kN}}{0.75 \text{ t/m}^2} \)
**DEAD LOAD MOMENTS**

Treating the slab as simply supported on all four sides, Dead load moments as per Table 4 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207)

- \( M_{xc} = 151.43 \times 0.75 = 113.3 \text{ kg-m/m} \)
- \( M_{yc} = 723.55 \times 0.75 = 541.4 \text{ kg-m/m} \)

II) For panel size = 2.5 x 7.25 (Key chart No. = 224)

- \( M_{xc} = 147.64 \times 0.75 = 110.5 \text{ kg-m/m} \)
- \( M_{yc} = 730.7 \times 0.75 = 546.7 \text{ kg-m/m} \)

For panel size = 2.5 x 2.53 m by interpolation

- \( M_{xc} = 110.5 \times \left( \frac{110.5 - 113.3}{7.25 - 7} \right) \times 4.73 = 164.06 \text{ kg-m/m} \)
- \( M_{yc} = 546.7 \times \left( \frac{546.7 - 541.4}{7.25 - 7} \right) \times 4.73 = 445.60 \text{ kg-m/m} \)

III) For panel size = 2.75 x 7 (Key chart No. = 208)

- \( M_{xc} = 196.66 \times 0.75 = 147.1 \text{ kg-m/m} \)
- \( M_{yc} = 847.73 \times 0.75 = 634.3 \text{ kg-m/m} \)

IV) For panel size = 2.75 x 7.25 (Key chart No. = 225)

- \( M_{xc} = 191.88 \times 0.75 = 143.6 \text{ kg-m/m} \)
- \( M_{yc} = 858.77 \times 0.75 = 642.5 \text{ kg-m/m} \)

For panel size = 2.75 x 2.53 m by interpolation

- \( M_{xc} = 143.6 \times \left( \frac{143.6 - 147.1}{7.25 - 7} \right) \times 4.73 = 211.16 \text{ kg-m/m} \)
- \( M_{yc} = 642.5 \times \left( \frac{642.5 - 634.3}{7.25 - 7} \right) \times 4.73 = 486.42 \text{ kg-m/m} \)

Actual panel size = 7.23 x 2.53 m by interpolation

- \( M_{xc} = 164.06 \times \left( \frac{164.06 - 211.16}{2.75 - 2.5} \right) \times -4.48 = -679.03 \text{ kg-m/m} \)
- \( M_{yc} = 486.4 \times \left( \frac{486.4 - 445.6}{2.75 - 2.5} \right) \times -4.48 = 1216.98 \text{ kg-m/m} \)
- \( M_{yc} = 1216.98 \text{ kg-m/m} \)
- \( M_{yc} = 12.17 \text{ kN-m/m} \)
LIVE LOAD MOMENTS

a) IRC - CLASS A LOADING

Treating the slab as simply supported on all four sides, CLASS A live load moments are evaluated as per Table 3 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207)
   
   Mxc = 835.49 kg-m/m
   Myc = 1621.06 kg-m/m

II) For panel size = 2.5 x 7.25 (Key chart No. = 224)
    
   Mxc = 833.74 kg-m/m
   Myc = 1622.11 kg-m/m

   For panel size = 2.5 x 2.53 m by interpolation
   
   Mxc = 833.7 \times \frac{833.7 - 835.5}{7.25 - 7} \times 4.73
   = 866.82 kg-m/m

   Myc = 1622.1 \times \frac{1622.1 - 1621.1}{7.25 - 7} \times 4.73
   = 1602.27 kg-m/m

III) For panel size = 2.75 x 7 (Key chart No. = 208)
    
   Mxc = 907.47 kg-m/m
   Myc = 1723.37 kg-m/m

IV) For panel size = 2.75 x 7.25 (Key chart No. = 225)
    
   Mxc = 906 kg-m/m
   Myc = 1726.38 kg-m/m

   For panel size = 2.75 x 2.53 m by interpolation
   
   Mxc = 906.0 \times \frac{906.0 - 907.5}{7.25 - 7} \times 4.73
   = 933.78 kg-m/m

   Myc = 1726.4 \times \frac{1726.4 - 1723.4}{7.25 - 7} \times 4.73
   = 1669.49 kg-m/m
Actual panel size = 7.23 x 2.53 m by interpolation

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>=</td>
<td>866.82</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>-331.91</td>
<td>kg-m/m</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>-3.32</td>
<td>kN-m/m</td>
</tr>
<tr>
<td>Myc</td>
<td>=</td>
<td>1669.5</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>2872.84</td>
<td>kg-m/m</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>28.73</td>
<td>kN-m/m</td>
</tr>
</tbody>
</table>

Impact Factors For IRC Class A
As per 211.2 of IRC 6 for CLASS A
Impact Factor for Mxc = 1 + \left( \frac{4.5}{6 + \frac{2.53}{7.23}} \right) = 1.34 < 1.5
Impact Factor for Myc = 1 + \left( \frac{4.5}{6 + \frac{2.53}{7.23}} \right) = 1.528 > 1.5

"CLASS A" - Live Load Moment with Impact are

\[
\begin{align*}
\text{Mxc} &= -3.319 x 1.34 = -4.45 \text{ kN-m} \\
\text{Myc} &= 28.728 x 1.50 = 43.09 \text{ kN-m}
\end{align*}
\]

b) IRC - CLASS 70 R LOADING

Treating the slab as simply supported on all four sides, CLASS 70R live load moments are evaluated as per Table 1 of SERC

I) For panel size = 2.5 x 7 (Key chart No. = 207 )

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>=</td>
<td>1305.09</td>
</tr>
<tr>
<td>Myc</td>
<td>=</td>
<td>3080.15</td>
</tr>
</tbody>
</table>

II) For panel size = 2.5 x 7.25 (Key chart No. = 224 )

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>=</td>
<td>1300.56</td>
</tr>
<tr>
<td>Myc</td>
<td>=</td>
<td>3093.95</td>
</tr>
</tbody>
</table>

For panel size = 2.5 x 2.53 m by interpolation

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>=</td>
<td>1300.6</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>1386.18</td>
<td>kg-m/m</td>
</tr>
<tr>
<td>Myc</td>
<td>=</td>
<td>3094.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>=</td>
<td>2833.13</td>
<td>kg-m/m</td>
</tr>
</tbody>
</table>

III) For panel size = 2.75 x 7 (Key chart No. = 208 )

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>=</td>
<td>1445.17</td>
</tr>
<tr>
<td>Myc</td>
<td>=</td>
<td>3386.74</td>
</tr>
</tbody>
</table>
IV) For panel size = 2.75 x 7.25 (Key chart No. = 225 )

Mxc = 1437.82 kg-m/m  
Myc = 3384.6 kg-m/m

For panel size = 2.75 x 2.53 m by interpolation  
Mxc = 1437.8 - ( 1437.8 - 1445.2 ) x 4.73  
= 1576.74 kg-m/m  
Myc = 3384.6 - ( 3384.6 - 3366.7 ) x 4.73  
= 3047.05 kg-m/m

Actual panel size = 7.23 x 2.53 m by interpolation  
Mxc = 1386.18 - ( 1386.18 - 1576.74 ) x -4.48  
= -2024.81 kg-m/m  
= -20.25 kN-m/m  
Myc = 3047.0 - ( 3047.0 - 2833.1 ) x -4.48  
= 6876.14 kg-m/m  
= 68.76 kN-m/m

As per 211.3 of IRC 6 for CLASS 70 R wheeled vehicle Impaact Factor = 1.25

"CLASS 70 R " - Live Load Moment with Impact are

Mxc = -20.25 x 1.250 = -25.31 kN-m > -4.45 ( Class A)  
Myc = 68.76 x 1.250 = 85.95 kN-m > 43.09 ( Class A)

Hence IRC Class 70R loading Governs the Design

Total BM

<table>
<thead>
<tr>
<th></th>
<th>DL B.M</th>
<th>LL B.M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mxc</td>
<td>-6.79</td>
<td>-25.31</td>
</tr>
<tr>
<td>Myc</td>
<td>12.17</td>
<td>85.95</td>
</tr>
</tbody>
</table>

Moments after considering the coefficient of continuity

Mxc = -32.10 x 0.8 = -25.7 kN-m  
Myc = 98.12 x 0.8 = 78.5 kN-m
**DESIGN OF SLAB**

Grade of Concrete = M 40
Grade of steel = Fe 415

As Per 303.3.3

\[
\sigma_{cbc} = 13.32 \text{ Mpa} \\
m = 10 \\
\sigma_{st} = 200 \text{ Mpa} \\
n_1 = 1 + \frac{10 \times 13.32}{1000} = 0.400 \\
j = 0.867 \\
R = 0.50 \times 13.32 \times 0.400 \times 0.867 = 2.308 \\
d_{reqd} = \left( \frac{78.50 \times 10^6}{2.308 \times 1000} \right)^{0.5} = 184.44 \text{ mm} \quad (< 215 \text{ mm (Effective depth Provided)})
\]

**Reinforcement for Myc**

\[
\text{Ast reqd for Myc} = \frac{78.50 \times 10^6}{200 \times 0.867 \times 215} = 2106 \text{ mm}^2 / \text{m}
\]

Provide Tor - 20 @ 140 mm c/c at top and bottom

Ast Provided = 2244 mm²/m

**Reinforcement for Mxc**

\[
\text{Ast reqd for Mxc} = \frac{-25.68 \times 10^6}{200 \times 0.867 \times 215} = -689 \text{ mm}^2 / \text{m}
\]

Provide Tor - 12 @ 140 mm c/c at top and bottom

Ast Provided = 808 mm²/m
DESIGN OF DIAPHRAGM
**DESIGN OF END DIAPHRAGM**

Sagging moment = 246.92 Kn m

Hogging moment = 622.76 Kn m

Shear force = 1221.1 Kn

**Structural Design**

Diaphragm span (c/c of girders) = 2.8 m

As per clause 29.2 of IS:456

1.15 times span = 3.22 > 2.8 m

Hence adopt 2.8 m as the effective span

Diaphragm depth = 1.8 m

\[
\frac{I}{D} = \frac{2.8}{1.56} < \frac{2.5}{1.8}
\]

Hence the end diaphragm is designed as a deep beam as per clause 29 of IS:456 - 2000.

Lever arm \( Z \) = 0.2 \( (L + 1.5D) \)

\[
Z = 0.2 \times (2.8 + 1.5 \times x) = 1.1 \text{ m}
\]

Effective span = 1.8 m

Allowable stress in H.Y.S.D \( \sigma_{st} \) = 13.33 N/mm²

Grade of concrete \( M_{30} \) \( \sigma_{cbr} \) = 200 Mpa

\( k = 0.4000 \)

\( j = 0.867 \)

\( Q = 2.311 \text{ N/mm}^2 \)

\[ + \text{ve Reinforcement} \]

\[
\frac{246.92 \times 10^6}{200 \times 1100} = 1122.364 \text{ mm}^2
\]

Min. Reinforcement = 0.2 \times 450 \times 1800 = 1620 mm²

Hence provide 3 nos. Tor - 20 + 3 nos. Tor - 20 reinforcement provided at bottom in 0.25D-0.05L from bottom i.e in 310 mm from bottom of diaphragm.

\[ - \text{ve Reinforcement} \]

\[
Ast = \frac{622.76 \times 10^6}{200 \times 1100} = 2830.727 \text{ mm}^2
\]

\[ > 1620 \text{ mm}^2 \]
Distribution of -ve Reinforcement

a) at \( 0.2 \times D \) = 0.36 m

\[
\text{Ast} = 2830.727 \times \left\{ 0.5 \times \left( \frac{2.8}{1.8} - 0.5 \right) \right\} = 1494.0
\]

\[= 1494\]

Hence provide 3 nos. Tor- 20 + 3 nos. Tor- 20 = 1884.96 mm²

b) Balance Ast = 945.77 mm²

= to be distributed in 0.6D below the Reinforcement at (a) above.

\[0.6 \times 1.8 = 1.08 \text{ m}\]

Hence, provide 13 nos Tor- 10 at middle = 1021.02 mm²

**SHEAR**

Since this is a deep beam shear does not govern. Hence provide Min. shear Reinforcement of 0.15 %

\[
\text{Asv} = \frac{0.15 \times 1800 \times 450}{100} = 1215 \text{ mm}^2
\]

provide 16 φ at 125 mm c/c = 1608.50 mm²

Total Shear = 1221.1 KN

Required Ast as hanging R/F = 6105.5 mm²

Required Ast per m length = 847.9861 mm²/m

Provide 2 Legg 10 φ @ 150 mm c/c = 1047.198 mm²/m

**Side face reinforcement**

0.1% of web area on either face with spacing not more than 450mm

Required Ast = 810 mm²/m

Provide 5 nos 16 φ on either side. = 1005.31 mm²/m
Design of cross girders

Concrete grade "M" 40 N/mm²
Steel grade "Fe" 415 N/mm²
Bending compressive stress in concrete (c) 13,333 N/mm²
Bending tensile stress in steel (t) 200 N/mm²
Modular ratio 10

Intermediate cross girder

- Thickness of cross girder 0.3 m
- Depth of cross girder including deck slab 2.75 m

End cross girder

- Thickness of cross girder 0.45 m
- Depth of cross girder including deck slab 2.75 m

Depth of deck slab 0.25 m
Spacing of longitudinal girders c/c 2.40 m
Web thickness at mid span 0.28 m
Web thickness at end of span 0.65 m

Hogging Bending Moment

Maximum Hogging Bending moment 768.09 kN.m
For continuous beam, Lever arm z = 0.2 (l+1.5D) 1.305 m
Reinforcement required at top Ast = M / t z 2943 mm²
Proportion of reinforcement in zone 0.2 D from top edge = 0.5 (l/D - 0.5) 0.14
Area of reinf. in this zone 401 mm²

Provide:

- Dia of bars 20 0
- No. of bars required 1 0
- No. of bars provided 4 0
- Area provided 1257 0
- Total area 1257 mm²

Provide this reinf. within a zone of 0.2D from top face i.e. 0.55 m

Proportion of reinf. in remaining zone 0.3 D on either side from mid depth 0.86
Area of reinf. in this zone 2542 mm²

Provide:

- Dia of bars 16 12 mm
- No. of bars required 13 0
- No. of bars provided on both faces 16 0
- No. of bars provided on each face 8 0
- Area provided 3217 0
- Total area 3217 mm²

Hence cross girder will be designed as deep beam

As per IS:456-2000, clause 29.1 Span / Depth < 2.50

Overall depth 2.75 m
Effective span 2.40 m
Span / Depth 0.87

Intermediate cross girder

Maximum Hogging Bending moment 768.09 kN.m
Maximum Sagging Bending moment 1367.5 kN.m
Maximum Shear Force 190.13 kN

Span c/c of supports 2.40 m
Clear span 2.13 m
Overall depth 2.75 m
Span / Depth 0.87

As per IS:456-2000, clause 29.1 Span / Depth < 2.50

Hence cross girder will be designed as deep beam
**Sagging Bending Moment**

Maximum Sagging Bending moment due to all loads \(1367.5\) kN.m

Reinforcement required at bottom \(\frac{A_s}{M} = \frac{M}{t \cdot z}\) 5239 mm²

Minimum reinforcement percentage 0.2 %

Minimum area of reinf. Required 1650 mm²

Reinforcement to be provided 5239 mm²

Provide:

| Dia of bars | 32 20 mm |
| No. of bars required | 7 0 |
| No. of bars provided | 7 0 |
| Area provided | 5630 0 mm² |

Total area 5630 mm²

Provide this reinf. within a zone of \((0.25 \cdot D - 0.05 \cdot l)\) from bottom face i.e. 0.57 m

**Design for Shear Force**

Maximum Shear Force 190.13 kN

As per IS:456-2000, clause 29.3.3, deep beam will be designed for hanging action

Reinforcement required for this action 951 mm²

Reinforcement required per meter length 792 mm²

Dia of stirrups 12 mm

No. of legs 2

Spacing required 286 mm

Spacing provided 125 mm
INPUT FILE: END.STD
1. STAAD PLANE END CROSS GIRDER DURING JACKED UP CONDITION
2. INPUT WIDTH 79
3. UNIT METER MTON
4. JOINT COORDINATES
   1 0 0 0; 2 0.51 0 0; 3 2.29 0 0; 4 2.8 0 0; 5 3.31 0 0
   6 5.09 0 0; 7 5.6 0 0; 8 6.11 0 0; 9 7.89 0 0; 10 8.4 0 0
5. MEMBER INCIDENCES
   8 1 1 2 9 1 1
6. MEMBER PROPERTY INDIAN
   1 TO 9 PRIS YD 1.0 ZD 1.0
7. SUPPORTS
   2 3 5 6 8 9 PINNED
8. CONSTANTS
   E 3.6E+006
   POISSON 0.15 ALL
9. *REACTION ON BEARING DUE TO DL+SIDL
10. LOAD 1
11. JOINT LOAD
12. 1 10 FY -122.11
13. 4 7 FY -99.38
14. PERFORM ANALYSIS
15. PRINT MEMBER FORCES

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2. START JOB INFORMATION
3. ENGINEER DATE 08-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
8. 1 0.000  0.0 0.0
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13. 6 5.400  0.0 0.0
14. 7 7.900  0.0 0.0
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22. 15 28.113  0.0 0.0
23. 16 30.400  0.0 0.0
24. 17 30.800  0.0 0.0
25. MEMBER INCIDENCES
26. 1 1 2; 2 3; 3 4; 4 5; 5 6; 6 7; 7 8; 8 9; 9 10; 10 10 11
27. 11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17
28. CONSTANTS
29. E 3.65E06 ALL
30. POISSON 0.15 ALL
31. DENSITY 2.5 ALL
32. MEMBER PROPERTY INDIAN
33. ***OUTER GIRDER
34. 6 TO 11 PRIS AX 0.9098 IX 0.0248 IZ 0.3951
35. 1 TO 3 14 TO 16 PRIS AX 1.0983 IX 0.0545 IZ 0.4184
36. 4 5 12 13 PRIS AX 1.2868 IX 0.0843 IZ 0.4417
37. SUPPORTS
38. 2 PINNED
39. 16 FIXED BUT FX MZ
40. ****************DEAD LOAD (SELF WEIGHT OF THE GIRDER)
41. LOAD 1
42. MEMBER LOAD
43. 6 TO 11 UNI GY -2.76525
44. 1 TO 2 15 TO 16 UNI GY -3.84664
45. 3 14 UNI GY -3.846624
46. 4 13 UNI GY -3.306124
47. 5 12 UNI GY -2.76525
48. ****************DIAPHRAM**************
49. LOAD 2
50. JOINT LOAD
51. 2 16 FY -4.268
52. 7 9 11 FY -2.9754
53. PERFORM ANALYSIS
54. PRINT MEMBER FORCES LIST 2  4 6 7 8

□ MEMBER FORCES LIST 2 □
MEMBER END FORCES  STRUCTURE TYPE = PLANE
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SUPPORT REACTIONS -UNIT MTON METE  STRUCTURE TYPE = PLANE
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56. PRINT MAXFORCE ENVELOPE LIST 1 TO 16

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********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

57. FINISH

********** END OF THE STAAD.Pro RUN **********

**** DATE= NOV 22,2006   TIME= 17:37:15 ****
**LIVE LOAD**

**DEFINE MOVING LOAD FILE SATY1**

**TYPE 1 CLASSA 1.125**

**TYPE 2 70RW 1.125**

**TYPE 3 70RT 1.1**

**FROM MEDIAN SIDE**

**TWO LANE OF CLASS A**

**LOAD GENERATION 105**

**TYPE 1 -18.8 0 4.45 XINC 0.5**

**TYPE 2 -13.4 0 5.81 XINC 0.5**

**TYPE 3 -4.576 0 5.93 XINC 0.5**

**FROM FOOTPATH SIDE**

**TWO LANE OF CLASS A**

**LOAD GENERATION 105**

**TYPE 1 -18.8 0 4.3 XINC 0.5**

**TYPE 2 -13.4 0 5.66 XINC 0.5**

**TYPE 3 -4.576 0 5.78 XINC 0.5**

**PERFORM ANALYSIS**

**PRINT FORCE ENVELOPE NSECTION 2 LIST 18 34 50 66**

**PRINT FORCE ENVELOPE NSECTION 2 LIST 20 36 52 68**

**PRINT FORCE ENVELOPE NSECTION 2 LIST 22 38 54 70**

**FORCE ENVELOPE NSECTION 2**

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**MAX/MIN FORCE VALUES FOR MEMB 18, AMONGST ALL SECT LOCATIONS**

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**Note:**
- **MIN** represents the minimum value.
- **MAX** represents the maximum value.
- **FY/ DIST** refers to the force component for a specific member.
- **LD** refers to the location designation.
- **MZ/ DIST** refers to the moment component for a specific member.
- **MY/ DIST** refers to the shear component for a specific member.
- **FX/ DIST** refers to the axial force component for a specific member.

**Table Continued...**
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| 40   | 0.00     | MAX   | 19.27 | 232   | 2.94  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -12.84 | 223   | -188.10 | 142  | 0.00  | 272   | 0.00  |
| 1.87 |          | MAX   | 13.02 | 236   | 2.83  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -20.07 | 227   | -190.96 | 146  | 0.00  | 272   | 0.00  |
| 3.75 |          | MAX   | 8.91  | 240   | 2.73  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -27.70 | 231   | -177.70 | 147  | 0.00  | 272   | 0.00  |

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| 56   | 0.00     | MAX   | 12.36 | 55    | 2.90  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -6.36 | 223   | -145.28 | 52   | 0.00  | 272   | 0.00  |
| 1.87 |          | MAX   | 9.48  | 236   | 2.38  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -7.32 | 134   | -148.00 | 56   | 0.00  | 272   | 0.00  |
| 3.75 |          | MAX   | 8.06  | 240   | 1.87  | 106   | 0.00  | 272   | 0.00  |
|      |          | MIN   | -8.38 | 138   | -146.68 | 147  | 0.00  | 272   | 0.00  |

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| 72     | 0.00  | MAX    | 6.11  | 64      | 2.22  | 106   |
|        | MIN   | -0.97  | 24    | -111.98 | 52    | 0.00  |
| 1.87   | MAX   | 5.87   | 59    | 2.12    | 106   | 0.00  |
|        | MIN   | -0.97  | 24    | -117.69 | 53    | 0.00  |
| 3.75   | MAX   | 5.65   | 63    | 2.01    | 106   | 0.00  |
|        | MIN   | -0.97  | 24    | -123.46 | 54    | 0.00  |

| MAX/MIN FORCE VALUES FOR MEMB 72, AMONGST ALL SECT LOCATIONS |
| FY/   | DIST | LD |
| FZ/   | DIST | LD |
| MY/   | DIST | LD |
| FX/   | DIST | LD |

- MAX. 6.11 0.00 64 2.22 0.00 106
- 0.00 0.00 1 0.00 0.00 1
- MIN. -0.97 3.75 24 -123.46 3.75 54
- 0.00 3.75 272 0.00 3.75 272

********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

145. LOAD LIST 148 158 132
146. PRINT MEMBER FORCES LIST 24

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z
24 132 25 0.00 -1.74 0.00 0.54 0.00 -158.60
26 0.00 4.20 0.00 -0.54 0.00 144.50
148 25 0.00 11.85 0.00 0.41 0.00 -201.84
26 0.00 -7.65 0.00 -0.41 0.00 238.92
158 25 0.00 13.37 0.00 0.33 0.00 -152.03
26 0.00 -12.22 0.00 -0.33 0.00 198.15

147. *****
148. LOAD LIST 143 150 139
149. PRINT MEMBER FORCES LIST 23

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z
23 139 24 0.00 7.69 0.00 0.65 0.00 -174.39
25 0.00 -2.77 0.00 -0.65 0.00 198.64
143 24 0.00 12.41 0.00 0.62 0.00 -169.78
25 0.00 -8.94 0.00 -0.62 0.00 207.59
150 24 0.00 14.86 0.00 0.63 0.00 -144.01
25 0.00 -13.70 0.00 -0.63 0.00 195.41

150. *************
151. LOAD LIST 140 143 150
152. PRINT MEMBER FORCES LIST 22

MEMBER LOAD JT AXIAL SHEAR-Y SHEAR-Z TORSION MOM-Y MOM-Z
22 140 23 0.00 23.68 0.00 1.72 0.00 -117.69
24 0.00 -21.94 0.00 -1.72 0.00 175.11
143 23 0.00 22.64 0.00 1.87 0.00 -113.76
24 0.00 -22.64 0.00 -1.87 0.00 170.34
150 23 0.00 19.88 0.00 2.02 0.00 -94.93
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153. LOAD LIST 137 136 150
154. PRINT MEMBER FORCES LIST 20

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155. LOAD LIST 138 140 150
156. PRINT MEMBER FORCES LIST 18

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157. FINISH

*********** END OF THE STAAD.Pro RUN ***********
INPUT FILE: SIDL.STD

1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 26-SEP-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
8. 1 0.000 0.0 0.0
9. 2 1.080 0.0 0.0
10. 3 3.368 0.0 0.0
11. 4 3.580 0.0 0.0
12. 5 4.830 0.0 0.0
13. 6 6.080 0.0 0.0
14. 7 8.580 0.0 0.0
15. 8 12.330 0.0 0.0
16. 9 16.080 0.0 0.0
17. 10 19.830 0.0 0.0
18. 11 23.580 0.0 0.0
19. 12 26.080 0.0 0.0
20. 13 27.330 0.0 0.0
21. 14 28.580 0.0 0.0
22. 15 28.793 0.0 0.0
23. 16 31.080 0.0 0.0
24. 17 32.160 0.0 0.0
25. REPEAT ALL 1 0.0 0.0 1.8
26. REPEAT 3 0.0 0.0 2.8
27. REPEAT 1 0.0 0.0 1.8
28. MEMBER INCIDENCES
29. 1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11
30. 11 11 12; 12 12 13; 13 13 14; 14 14 15; 15 15 16; 16 16 17; 17 17 18; 18 18 19; 19 19 20
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32. 28 29 29; 29 29 30; 30 30 31; 31 31 32; 32 32 33; 33 33 34; 34 34 35; 35 35 36; 36 36 37
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42. 105 53 70; 106 70 87; 107 5 22; 108 22 39; 109 39 56; 110 56 73; 111 73 90
43. 112 7 24; 113 24 41; 114 41 58; 115 58 75; 116 75 92; 117 8 25; 118 25 42
44. 119 42 59; 120 59 76; 121 76 93; 122 9 26; 123 26 43; 124 43 60; 125 60 77
45. 126 77 94; 127 10 27; 128 27 44; 129 44 61; 130 61 78; 131 78 95; 132 11 28
46. 133 28 45; 134 45 62; 135 62 79; 136 79 96; 137 13 30; 138 30 47; 139 47 64
47. 140 64 81; 141 81 98; 142 16 33; 143 33 50; 144 50 67; 145 67 84; 146 84 101
48. 147 17 34; 148 34 51; 149 51 68; 150 68 85; 151 85 102
49. MEMBER PROPERTY INDIAN
50. *OUTER GIRDER
51. 22 TO 27 70 TO 75 PRIS AX 1.906 IX 0.0404 IZ 1.3135
52. 17 18 31 32 65 66 79 80 PRIS AX 2.339 IX 0.1104 IZ 1.4253
53. 19 TO 21 28 TO 30 67 TO 69 76 TO 78 PRIS AX 2.122 IX 0.0754 IZ 1.3694
54. *INNER GIRDER
55. 38 TO 43 54 TO 59 PRIS AX 1.806 IX 0.0393 IZ 1.2605
56. 33 34 47 TO 50 63 64 PRIS AX 2.239 IX 0.1093 IZ 1.3623
57. 35 TO 37 44 TO 46 51 TO 53 60 TO 62 PRIS AX 2.022 IX 0.0743 IZ 1.3114
58. *END DIAPHRAM
59. 103 TO 105 143 TO 145 PRIS AX 1.549 IX 0.054 IZ 0.6285
60. *CENTRAL DIAPHRAM
61. 113 TO 115 123 TO 133 PRIS AX 1.478 IX 0.0240 IZ 0.5107
62. *CANTILEVER PORTION
63. 1 TO 16 81 TO 96 PRIS AX 0.001 IX 0.0001 IZ 0.0001
64. *TRANSVERSE MEMBERS
65. 107 TO112 116 TO122 126 TO132 136 TO141 PRIS AX 0.938 IX0.0098 IZ 0.0049
66. 102 106 142 146 PRIS AX 0.739 IX 0.0077 IZ 0.0038
67. 97 TO 101 147 TO 151 PRIS AX 0.0001 IX 0.0001 IZ 0.0001
68. CONSTANTS
69. E 2.2146E06
70. DEN CONC ALL
71. POISSON 0.15
72. SUPPORTS
73. 19 36 53 70 PINNED
74. 33 50 67 84 FIXED BUT FX MX MY MZ
75. **** SIDL INCLUDING FOOTPATH LL
76. LOAD 1
77. MEMBER LOAD
78. 1 TO 16 UNI GY -1.503
79. 17 TO 32 UNI GY -0.862
80. 33 TO 48 UNI GY -0.552
81. 49 TO 64 UNI GY -0.552
82. 65 TO 80 UNI GY -0.862
83. 81 TO 96 UNI GY -1.503
84. PERFORM ANALYSIS
85. PRINT MEMBER FORCES LIST 24 40 56 72
86. PRINT MEMBER FORCES LIST 18 20 22 23 24

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<th>SHEAR-Z</th>
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<th>MOM-Z</th>
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86. PRINT MEMBER FORCES LIST 18 20 22 23 24
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87. FINISH
INPUT FILE: SLAB.STD
1. STAAD PLANE
2. START JOB INFORMATION
3. ENGINEER DATE 26-SEP-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
8. 1 0.000 0.0 0.0
9. 2 1.080 0.0 0.0
10. 3 3.368 0.0 0.0
11. 4 3.580 0.0 0.0
12. 5 4.830 0.0 0.0
13. 6 6.080 0.0 0.0
14. 7 8.580 0.0 0.0
15. 8 12.330 0.0 0.0
16. 9 16.080 0.0 0.0
17. 10 19.830 0.0 0.0
18. 11 23.580 0.0 0.0
19. 12 26.080 0.0 0.0
20. 13 27.330 0.0 0.0
21. 14 28.580 0.0 0.0
22. 15 28.793 0.0 0.0
23. 16 31.080 0.0 0.0
24. 17 32.160 0.0 0.0
25. MEMBER INCIDENCES
26. 1 1; 2 2; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11
27. 11 11; 12 12; 13 13; 14 14; 15 15; 16 16; 17 17
28. DEFINE MATERIAL START
29. ISOTROPIC MATERIAL1
30. E 3.65E+006
31. POISSON 0.15
32. DENSITY 2.4026
33. DAMP 0.001
34. END DEFINE MATERIAL
35. MEMBER PROPERTY INDIAN
36. OUTER GIRDER
37. 6 TO 11 PRIS AX 0.9098 IX 0.0248 IZ 0.3951
38. 1 TO 3 14 TO 16 PRIS AX 1.0983 IX 0.0545 IZ 0.4184
39. 4 5 12 13 PRIS AX 1.2868 IX 0.0843 IZ 0.4417
40. CONSTANTS
41. MATERIAL MATERIAL1 MEMB 1 TO 16
42. SUPPORTS
43. 2 16 PINNED
44. DEAD LOAD (SELF WEIGHT OF THE SLAB)
45. LOAD 1
46. MEMBER LOAD
47. *(2.8*0.5+1.8)*0.25*2.4
48. 1 TO 16 UNI GY -2
49. *** SHUTTERING SLAB
50. LOAD 2
51. MEMBER LOAD
52. 1 TO 16 UNI GY -0.5
53. PERFORM ANALYSIS
54. PRINT MEMBER FORCES LIST 2 4 6 TO 8
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55. PRINT SUPPORT REACTION
SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE

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56. FINISH
BRIDGE AT CH:29+400
DESIGN OF SUBSTRUCTURE
(ABUTMENT - PIER)
**Design Data:**
For design purposes, following parameters have been considered.

- Grade of concrete = M - 25
- Abutment Cap = M - 25
- Grade of reinforcement steel = Fe - 415
- Centre to Centre distance of A / Expansion joints = 9.200 m
- Centre to Centre distance of Bearing = 8.800 m
- Depth of superstructure = 670 mm
- Thickness of wearing coat = 56.00 mm
- Formation level along C of carriage way = 216.189 m
- Soffit level = 215.463
- Pedestal top level = 215.463
- Height of bearing and Pedestal = 0.000 m
- L.W.L./Bed level = 209.907 m
- H.F.L = 214.172 m
- M.S.L = 202.700 m
- Founding Level = 204.975 m
- Abutment cap top level = 215.463 m
- Gross safe bearing capacity = 29.400 t/m²

- Live Load (a) Class A two Lane
- (b) Class 70R wheeled

Bearing neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:

1. IRC : 6 - 2000
2. IRC : 21 - 2000
3. IRC : 78 - 2000
As per clause 214.2 of IRC:6, horizontal braking force $F_h$, for each span is:

For Class A 2 lane:  
$$F_h = 0.2 \times 49.5 = 9.896 \text{ t}$$

For class 70R wheeled:  
$$F_h = 0.2 \times 47.72 = 9.544 \text{ t}$$

### Summary of Longitudinal Forces:

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### Dead load

\[
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\uparrow & 0.76 & \\
8.50 & & 0.67 \\
\end{array}
\]

- Dead load of slab = 14.561 t/m
- Wearing coat = 0.924 t/m
- Crashbarrier = 1.000 t/m
- Dead load reaction = 66.978 t
- Sidl reaction = 13.45 t
### Dry Condition with L.L

**a) Vertical load and their moments about C/L of Foundation base.**

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<td>e_T</td>
<td>M_T</td>
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<th>P</th>
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<tr>
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<td>below G.L</td>
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<td>213.71</td>
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<td>7 Earth wt on</td>
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<td>118.942</td>
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<td>-263.22</td>
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<td>heel side</td>
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<td><strong>-263</strong></td>
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</table>

**b) Horizontal Forces and Moments with respect to Base**

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.77</td>
<td>10.488</td>
<td>50.049</td>
<td>10.488</td>
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<tr>
<td>2 Earth pressure</td>
<td></td>
<td>462.11</td>
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</tr>
</tbody>
</table>

**Summary**

| P       | = 463.05 t |
| M_L     | = 268 t-m  |
| M_T     | = 55.355 t-m |

| A       | = 30.813 |
| Z_L     | = 37.232 |
| Z_T     | = 21.826 |
Check for Maximum Allowable Base Pressure:

\[ P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} \]

\[ = \frac{463.05}{30.813} + \frac{268.16}{37.232} + \frac{55.355}{21.826} = 24.767 \text{ t/m}^2 \]

\[ P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} \]

\[ = \frac{463.05}{30.813} - \frac{268.16}{37.232} - \frac{55.355}{21.826} = 5.285 \text{ t/m}^2 \]

Design of Toe slab

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area factor</th>
<th>Force</th>
<th>L.A</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1</td>
<td>2.034</td>
<td>1.4125</td>
<td>2.873</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>2.5425</td>
<td>0.9417</td>
<td>2.3942</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>3</td>
<td>1</td>
<td>-16.618</td>
<td>1.4125</td>
<td>-23.472</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>-7.9283</td>
<td>0.9417</td>
<td>-7.4658</td>
</tr>
</tbody>
</table>
Net bending moment at face of stem $= 25.671$ t-m/m
Effective depth required $= 0.4772$ m
Effective depth provided at face of stem $= 0.965$ m
Area of reinforcement required $= 1445.8$ mm$^2$
Minimum steel req $= 1447.5$ mm$^2$

Provide $16 \phi 120$ mm c/c $= 1675.5$ mm$^2$

**Shear**

Effective depth of footing $= 0.967$ m
Total depth of section at distance $d$ from Abutment $= 0.7933$ m
Effective depth at distance $d$ from face of Abutment $= 0.7023$ m

\[
\tan(\beta) = 0.404
\]

Net shear force $= 9.15$ t
Shear stress $= 13.026$ t/m$^2$
% of reinforcement $= 0.2386$
Permissible shear stress $= 26.89$ t/m$^2$

NO SHEAR REINFORCEMENT REQ

**Design of Heel slab**

\[
\begin{array}{cccccc}
\text{Loadings} & \text{Element} & \text{Area factor} & \text{Force} & \text{L.A} & \text{Moment} \\
\text{Downward loads} & 1 & 1 & 1.3378 & 0.929 & 1.2428 \\
& 2 & 0.5 & 1.6722 & 0.6193 & 1.0356 \\
\text{Upward base pressure} & 3 & 1 & -18.539 & 0.929 & -17.223 \\
& 4 & 0.5 & -3.4295 & 0.6193 & -2.124 \\
\end{array}
\]

Effective depth at a distance of $d_{eff} = 0.7023$ m
Shear at critical section $= 18.96$ t
Bending moment at critical section $= 17.07$ t-m

Design of Heel slab

\[
\text{Base pressure diagram}
\]

Heel

\[
\begin{array}{cccc}
\text{Design of Heel slab} & 2.825 & \text{Heel} \\
22.231 & \text{3} & 7.8257 \\
13.439 & \text{4} & \\
\end{array}
\]

\[
\text{Base pressure diagram}
\]

\[
\begin{array}{cccc}
\text{Design of Heel slab} & 2.825 & \text{Heel} \\
22.231 & \text{3} & 7.8257 \\
13.439 & \text{4} & \\
\end{array}
\]

\[
\text{Base pressure diagram}
\]

\[
\begin{array}{cccc}
\text{Design of Heel slab} & 2.825 & \text{Heel} \\
22.231 & \text{3} & 7.8257 \\
13.439 & \text{4} & \\
\end{array}
\]

\[
\text{Base pressure diagram}
\]

\[
\begin{array}{cccc}
\text{Design of Heel slab} & 2.825 & \text{Heel} \\
22.231 & \text{3} & 7.8257 \\
13.439 & \text{4} & \\
\end{array}
\]

\[
\text{Base pressure diagram}
\]
Loadings | Element | Area factor | Force | L.A  | Moment  
--- | --- | --- | --- | --- | ---
Downward loads | 1 | 1  | 2.034 | 1.4125 | 2.873 |
| 2 | 0.5  | 2.5425 | 0.9417 | 2.3942 |
| earth | 55.498 | 1.4125 | 78.39 |
Upward base pressure | 3 | 1  | -7.8257 | 1.4125 | -11.054 |
| 4 | 0.5  | -7.9283 | 0.9417 | -7.4658 |

Net bending moment at face of stem = 65.138 t-m/m
Effective depth required = 0.7601 m
Effective depth Provided at face of stem = 0.9975 m
Area of reinforcement required = 3549 mm²
Minimum steel req = 1496.3 mm²
Provide 25 φ 125 mm c/c = 3927 mm²

Shear
Effective depth d of footing = 0.9625 m
Total depth of section at distance d from Abutment = 0.7945 m
Effective depth at distance d from face of Abutment = 0.6945 m

width b = 4.25

Loadings | Element | Area factor | Force | L.A  | Moment  
--- | --- | --- | --- | --- | ---
Downward loads | 1 | 1  | 1.3378 | 0.929 | 1.2428 |
| 2 | 0.5  | 1.6722 | 0.6193 | 1.0356 |
| earth | 36.501 | 0.929 | 33.909 |
Upward base pressure | 3 | 1  | -7.8257 | 0.929 | -7.2701 |
| 4 | 0.5  | -3.4295 | 0.6193 | -2.124 |

Effective depth at a distance of d eff = 0.6945 m
Shear at critical section = 28.26 t
Bending moment at critical section = 26.79 t-m
\[ \tan(\beta) = 0.404 \]
Net shear force = 12.68 t
Shear stress = 18.261 t/m²
% of reinforcement = 0.5655
Permissible shear stress = 30.478 t/m²
NO SHEAR REINFORCEMENT REQ
Dry Condition with L.L

<table>
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<tr>
<th>II Substructure</th>
<th>V</th>
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<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<tr>
<td>a Left span</td>
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<td>0.15</td>
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<td>Abut cap Tr</td>
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<td>11.47</td>
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<td>0</td>
</tr>
<tr>
<td>3 Abutment shaft</td>
<td>10.334</td>
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<td>24.801</td>
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<tr>
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<td>9.8903</td>
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b) Horizontal Forces and Moments with respect to Abutment Shaft bottom

<table>
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<tr>
<th>1 Longitudinal Forces at bearing level</th>
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<tr>
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<td>4.77</td>
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<tr>
<td>4.77</td>
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Summary for design of Abutment

- \( P = 192.02 \) t
- \( M_L = 414.81 \) t-m
- \( M_T = 55.355 \) t-m
### H.F.L Condition with L.L

**a) Vertical load and their moments about C/L of Foundation base.**

<table>
<thead>
<tr>
<th>I</th>
<th>1 D.L Reaction</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<tbody>
<tr>
<td></td>
<td>a Left span</td>
<td>66.978</td>
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<td>10.047</td>
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<td>2 S.I.D.L</td>
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<tr>
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<td>a Left span</td>
<td>13.450</td>
<td>0.150</td>
<td>2.018</td>
<td>0.000</td>
<td>0.000</td>
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</tbody>
</table>

\[
80.429 \quad 12.064 \quad 0.000
\]

| 3 L.L | a Left span | 70R Wheeled | 47.720 | 0.150 | 7.158 | 1.160 | 55.355 |
|       | Class A 2 Lane | 49.480 | 0.150 | 7.422 | 0.700 | 34.636 |

\[
128.149 \quad 19.222 \quad 55.355
\]

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>V</th>
<th>p</th>
<th>P</th>
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<td></td>
<td></td>
<td></td>
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<td>a</td>
<td>Left span</td>
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<td>2.400</td>
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<td>3 Abut cap</td>
<td>4.779</td>
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<td>11.470</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>4 Footing</td>
<td>20.224</td>
<td>2.400</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<td>32.688</td>
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<tr>
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<td>6 Earth Pressure</td>
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<tr>
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<td>7 Earth wt on heel side</td>
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<td>-2.213</td>
<td>-146.23</td>
<td>196.351</td>
<td>136.639</td>
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</table>

**b) Horizontal Forces and Moments with respect to Base**

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<tr>
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<td>4.772</td>
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<td>10.488</td>
<td>50.049</td>
<td>10.488</td>
<td>0.000</td>
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</table>

**Summary**

\[
P = 324.500 \text{ t}
\]

\[
M_L = 205.910 \text{ t-m}
\]

\[
M_T = 55.355 \text{ t-m}
\]
Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]
\[
= \frac{324.500}{30.813} + \frac{205.910}{37.232} + \frac{55.355}{21.826} = 18.598 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]
\[
= \frac{324.500}{30.813} - \frac{205.910}{37.232} - \frac{55.355}{21.826} = 2.465 \text{ t/m}^2
\]

H.F.L. Condition with L.L

\(a)\) Vertical load and their moments about Abutment Shaft bottom

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>V</th>
<th>(\rho)</th>
<th>(P)</th>
<th>(e_L)</th>
<th>(M_L)</th>
<th>(e_T)</th>
<th>(M_T)</th>
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<tr>
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</tr>
<tr>
<td></td>
<td>a Left span</td>
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<td>2.400</td>
<td>0.000</td>
<td>0.150</td>
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<td>0.000</td>
<td>0.000</td>
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<tr>
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<td>0.000</td>
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<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Abut cap Tr</td>
<td>4.779</td>
<td>2.400</td>
<td>11.470</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
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<td>3 Abut shaft up to H.F.L</td>
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<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>below H.F.L</td>
<td>20.224</td>
<td>1.400</td>
<td>28.313</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>43.647</td>
<td>0.000</td>
<td>55.355</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

\(b)\) Horizontal Forces and Moments with respect to Abutment Shaft bottom

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>(H_L)</th>
<th>(H_T)</th>
<th>(e_L)</th>
<th>(M_L)</th>
<th>(e_T)</th>
<th>(M_T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.772</td>
<td>0.000</td>
<td>9.438</td>
<td>45.038</td>
<td>9.438</td>
<td>0.000</td>
</tr>
<tr>
<td>Earth pressure</td>
<td>4.772</td>
<td>0.000</td>
<td>219.644</td>
<td>264.682</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Summary for design of Abutment shaft

\[
P = 171.795 \text{ t}
\]
\[
M_L = 283.904 \text{ t-m}
\]
\[
M_T = 55.355 \text{ t-m}
\]
Summary of Loads at Abutment Shaft bottom:

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M&lt;sub&gt;L&lt;/sub&gt;</th>
<th>M&lt;sub&gt;T&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 DRY condition With L.L</td>
<td>192.02</td>
<td>414.81</td>
<td>55.36</td>
</tr>
<tr>
<td>2 H.F.L  With L.L</td>
<td>171.80</td>
<td>283.90</td>
<td>55.36</td>
</tr>
</tbody>
</table>
Dimensions of Substructure & Foundation

1 Pedestal

Length = 0.7
Width = 0.55 Volume = 0
Height = 0

2 Pier cap

a) Top uniform portion
Width = 0.90
Depth = 0.225 Volume = 1.60988 m³
Length = 7.95

b) Top uniform portion
Width = 0.90
Depth = 0.075 Volume = 0.53663 m³
Length = 7.95

c) Bottom trapezoidal portion
Width = 0.90
Depth = 1.2 Volume = 4.779 m³
Length = 7.95

Area at level 215.163 m = 7.95 x 0.9 = 7.155 m²
Area at level 213.963 m = 0.9 x 0.9 = 0.81 m² = 6.9255 m³

H.F.L
Pier cap = 1.60988 m³
= 0.53663 m³
= 4.779 m³

3 Abutment shaft

Area at abutment shaft bottom = 3.45062 m²
Area at abutment shaft top = 1.64485 m²
Average area = 2.54773 m²
Height of Abutment shaft = 7.94
Height above Ground level = 4.056 m Volume = 10.3336
Height below Ground level = 3.882 m Volume = 9.8903
= 20.2239 m³

Height above H.F.L = 0.000 m Volume = 0
Height below H.F.L = 7.94 m Volume = 20.2239 m³

4 Pedestal at footing top
Width = 1.6
Length = 2.50 Volume = 0 m³
Height = 0
5 Footing

at foundation level

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>7.250 m</td>
</tr>
<tr>
<td>Length</td>
<td>4.25  m</td>
</tr>
<tr>
<td>Thickness at Root</td>
<td>1.05  m</td>
</tr>
<tr>
<td>Thickness at Tip</td>
<td>0.3   m</td>
</tr>
</tbody>
</table>

Volume of Overburden earth below ground level

Total volume = 7.250 m x 4.250 m x 4.932 m = 151.967 m³
Net volume below Ground level = 118.729 m³
Wt of earth on Heel side above ground level = 66.0788 t

Sectional Properties of Footing

<table>
<thead>
<tr>
<th>Property</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>7.250 m x 4.250 m</td>
<td>30.813 m²</td>
</tr>
<tr>
<td>Z_L</td>
<td>4.250 m x 8.76042 m</td>
<td>37.232 m³</td>
</tr>
<tr>
<td>Z_T</td>
<td>7.250 m x 3.01042 m</td>
<td>21.826 m³</td>
</tr>
</tbody>
</table>
DESIGN OF ABUTMENT CAP:

For Outer bearing

\[ \frac{a}{D} = \frac{2.125}{1.425} = \frac{1.4912}{1.5} > 1 \]

= Pier cap designed as Cantilever

Available effective depth

using 25 mm dia of bars

\[ d = 1.425 \text{ m} \]

\[ a = 2.125 \]

\[ D = 1.50 \]

\[ = 1.425 \text{ m} \]

\[ = 1.4912 > 1 \]

\[ \text{Pier cap designed as Cantilever} \]
Bearings A1 & A4 are effective.

<table>
<thead>
<tr>
<th>A1</th>
<th>16.745</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>3.3626</td>
</tr>
<tr>
<td>LL</td>
<td>11.93</td>
</tr>
<tr>
<td>70RW</td>
<td>55.355</td>
</tr>
</tbody>
</table>

due to Transverse moment  
\[ A1 = 47.72 \times 1.16 = 55.355 \]

\[ A1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t} \]

**Summary of loads from superstructure:**

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>A1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>16.745</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3626</td>
</tr>
<tr>
<td>LL</td>
<td>7.3807</td>
</tr>
<tr>
<td>70RW</td>
<td>11.93</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>39.418</strong></td>
</tr>
</tbody>
</table>

Selfweight of piercap upto section through outerbearing:

\[ = 0.3 + 1.50 \times 3.00 \times 0.9 \times 2.4 \times 2.00 = 5.832 \text{ t} \]

Distance of c.g from section  
\[ = 1.1667 \text{ m} \]

**Calculation of Cantilever moment at bearing section**

Moment at the face due to load from outerbearing  
\[ = 39.42 \times 2.13 = 83.76297 \text{ t-m} \]

Due to selfweight of Abutment cap  
\[ = 5.832 \times 1.1667 = 6.804 \text{ t-m} \]

Total moment at the face of support  
\[ = 90.567 \text{ t-m} \]

Torsion at outerbearing section:

\[ = 39.418 \]

Eccentricity  
\[ = 0.15 \]

Moment  
\[ = 5.9127 \text{ t-m} \]
Longitudinal Reinforcement:

\[ M = 90.567 \text{ t-m} \]

For, \( M = 25 \) & \( F_e = 415 \)

\[ \sigma_{cbc} = 8.33 \text{ N/mm}^2 \]
\[ m = 10 \]
\[ \sigma_{st} = 200 \text{ N/mm}^2 \]
\[ K = 0.294 \]
\[ j = 0.902 \]
\[ R = 1.105 \]

Effective depth required

\[ = \frac{90.567 \times 10^7}{1.105 \times 900} \]
\[ = 954.15 < 1425 \text{ O.K} \]

\[ A_{st} \text{ required} = \frac{90.567}{200 \times 0.902 \times 1425} \]
\[ = 3523.2002 \text{ mm}^2 \]

Provide 6 nos 20 mm dia bars in 2 layers 3769.9 mm$^2$

This reinforcement will provided in full length of Abutment cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 39.418 t

Downward Load due to self wt of Abutment cap = 5.832 t

Total downward Load = 45.25 t

After shear correction, \( S.F = \frac{V - M \tan \beta}{d} \)

\[ = 45.25 - \frac{90.567 \times 0.4}{1.425} \]
\[ = 19.827561 \text{ t} \]

Equivalent shear \( IRC:21-2000,cl:304.2.3.1 \)

\[ b = 0.9 + \frac{0.9}{2} = 0.9 \text{ m} \]

\[ V_e = V + \frac{1.6 \times T}{b} = 30.339 \text{ t} \]

Shear stress

\[ = \frac{30.339 \times 10000}{900 \times 1425} = 0.2366 \text{ N/mm}^2 \]

Area of tension reinforcement

\[ \tau_e = 0.294 \% \]

Provide minimum shear
EARTH PRESSURE CALCULATION

Formation level = 216.189 m
Founding level = 204.975 m
Low Water Level = 209.907 m
Highest flood level = 214.172 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb's theory of active earth pressure

\[ K_a = \frac{\sin^2 (\phi + \delta)}{\sin^2 \alpha \sin^2 \delta \left[ 1 + \frac{\sin (\phi + \delta) \sin (\phi - i)}{\sin (\alpha - \delta) \sin (\alpha + i)} \right]^2} \]

Here
- Angle of internal friction, \( \phi = 30^\circ \)
- Angle of friction between soil and concrete, \( \delta = 20^\circ \)
- Surcharge angle, \( i = 0^\circ \)
- Angle of wall face with horizontal, \( \alpha = 90^\circ \)
- Bulk density of earth, \( \gamma = 1.8 \text{ t/m}^3 \)
- Submerged density of earth, \( \gamma_{sub} = 1.0 \text{ t/m}^3 \)
- Width of abutment = 2.50 m

\[ K_a = \frac{\sin^2 1.57 \times \sin 2.09 \times \sin 1.22 \times \left[ \frac{1}{\sin 0.87 \times \sin 1.57} \right] \times \left( \frac{\sin^2 2.09}{\sin^2 2.09} \right)}{2.523} = 0.2973 \]

CALCULATION OF EARTH PRESSURE IN DRY CONDITION

\[ P_1 = 0.2973 \times 1.8 \times 11.21 = 6 \text{ t/m}^2 \]
\[ P_2 = 0.2973 \times 1.0 \times 0.00 = 0 \text{ t/m}^2 \]
\[ F_1 = 0.5 \times 6.001 \times 11.21 \times 2.5 \times \cos 0.3 = 79.051 \text{ t say } 80 \text{ t} \]
\[ M = 79.051 \times 0.42 \times 11.21 = 372 \text{ t-m} \]

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

\[ P_4 = 0.2973 \times 1.8 \times 2.02 = 1.08 \text{ t/m}^2 \]
\[ P_5 = 0.2973 \times 1.0 \times 9.20 = 2.73 \text{ t/m}^2 \]
\[ F_4 = 0.5 \times 1.079 \times 2.02 \times 2.5 \times \cos 0.3 = 2.5574 \text{ t} \]
\[ F_5 = 0.5 \times 2.734 \times 9.20 \times 2.5 \times \cos 0.3 = 29.54 \text{ t} \]
\[ F_6 = 1.08 \times 9.20 \times 2.5 \times \cos 0.3 = 23.32 \text{ t} \]
\[ \text{Total force, } F = 2.5574 + 29.54 + 23.32 = 55 \text{ t} \]
\[ M = 2.5574 \times (0.42 \times 2.02 + 9.20) + 29.54 \times 0.33 \times 9.20 + 23.3 \times \frac{9.20}{2} = 223 \text{ t-m} \]
**CALCULATION OF LIVE LOAD SURCHARGE**

**Dry**

\[
P6 = 0.2973 \times 1.2 \times 1.8 = 0.64 \text{ t/m}^2
\]

\[
F7 = 0.64 \times 11.21 \times 3 \times \cos 0.3
\]

\[= 17 \text{ t}\]

\[M = 17 \times \frac{10.61}{2} = 90 \text{ t m}\]

**H.F.L**

\[
P6 = 0.2973 \times 1.2 \times 1.0 = 0.36 \text{ t/m}^2
\]

\[
F7 = 0.6422 \times 2.02 \times 3 \times \cos 0.3 = 3.043011 \text{ t}
\]

\[= 0.3568 \times 9.20 \times 3 \times \cos 0.3 = 7.708525 \text{ t}
\]

\[= 10.75154\]

\[M = 3.043 + 7.708524503 \times 5.607 = 60.28386 \text{ tm}\]
EARTH PRESSURE CALCULATION

Formation level = 216.189 m
Abutment shaft bottom level = 206.025 m
Low Water Level = 209.907 m
Highest flood level = 214.172 m

CALCULATION OF ACTIVE EARTH PRESSURE

From Coulomb’s theory of active earth pressure

$$Ka = \frac{\sin^2 (\alpha + \phi)}{\sin^2 \alpha \sin(\alpha - \delta) \left[ 1 + \frac{\sin (\phi + \delta) \cdot \sin (\phi - i)}{\sin (\alpha - \delta) \cdot \sin (\alpha + i)} \right]^2}$$

Here
- Angle of internal friction, \(\phi\) = 30°
- Angle of friction between soil and concrete, \(\delta\) = 20°
- Surcharge angle \(i\) = 0°
- Angle of wall face with horizontal \(\alpha\) = 90°
- Bulk density of earth \(\gamma\) = 1.8 t/m³
- Submerged density of earth \(\gamma_{sub}\) = 1.0 t/m³
- Width of abutment = 2.50 m

$$K_a = \left( \frac{\sin^2 1.57 \times \sin 1.22 \times \sqrt{1 + \frac{\sin 0.87 \times \sin 0.52}{\sin 1.22 \times \sin 1.57}}}{\sin^2 2.09}\right)$$

* (value of angles in radian)

$$= \frac{0.750}{2.523} = 0.2973$$

CALCULATION OF EARTH PRESSURE IN DRY CONDITION

\(P_1\) = 0.2973 x 1.8 x 10.16 = 5.44 t/m²
\(P_2\) = 0.2973 x 1.0 x 0.00 = 0 t/m²

\(F_1\) = 0.5 x 5.439 x 10.16 x 2.5 x \(\cos 0.3\)
= 64.941 t say 65 t

\(M = 64.941 \times 0.42 \times 10.16 = 277\) t-m

CALCULATION OF EARTH PRESSURE IN FULL SUPPLY LEVEL CONDITION

\(P_4 = 0.2973 \times 1.8 \times 2.02 = 1.08\) t/m²
\(P_5 = 0.2973 \times 1.0 \times 8.15 = 2.42\) t/m²

\(F_4 = 0.5 \times 1.079 \times 2.02 \times 2.5 \times \cos 0.3\)
= 2.5574 t

\(F_5 = 0.5 \times 2.422 \times 8.15 \times 3 \times \cos 0.3\)
= 23.18 t

\(F_6 = 1.08 \times 8.15 \times 2.5 \times \cos 0.3\)
= 20.66 t

Total force, \(F = 2.5574 + 23.18 + 20.66\)
= 46 t

\(M = 2.5574 \times (0.42 \times 2.02 + 8.15)\)
+ 23.18 \times 0.33 \times 8.15 + 20.7 \times \frac{8.15}{2}
= 169\) t-m
CALCULATION OF LIVE LOAD SURCHARGE

Dry

\[ P_6 = 0.2973 \times 1.2 \times 1.8 = 0.64 \text{ t/m}^2 \]

\[ F_7 = 0.64 \times 10.16 \times 3 \times \cos 0.3 \]

\[ = 15 \text{ t} \]

\[ M = 15 \times \frac{9.56}{2} = 73 \text{ t-m} \]

H.F.L

\[ P_6 = 0.2973 \times 1.2 \times 1.0 = 0.36 \text{ t/m}^2 \]

\[ F_7 = 0.6422 \times 2.02 \times 3 \times \cos 0.3 = 3.043011 \text{ t} \]

\[ = 0.3568 \times 8.15 \times 3 \times \cos 0.3 = 6.82846 \text{ t} \]

\[ = 9.87147 \]

\[ M = 3.043 + 6.82846 \times 5.082 = 50.16682 \text{ tm} \]
ABUT SHAFT .case 1 L.W.L

Depth of Section = 1.600 m
Width of Section = 2.500 m

along width-compression face- no of bar: 18 tension face- no of bar: 18
Dia (mm) : 20 32
Cover (cm) : 7.50 7.5

along depth-compression face- no of bar: 6 tension face- no of bar: 6
Dia (mm) : 16 16
Cover (cm) : 7.50 7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 192.020 T
Mxx = 414.810 Tm
Myy = 55.360 Tm

Intercept of Neutral axis : X axis := 9.879 m
:y axis := .543 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 72.81 Kg/cm^2
Stress in Steel due to Loads = 1496.34 Kg/cm^2
Percentage of Steel = .56 %

ABUT SHAFT .case 2 H.F.L

Depth of Section = 1.600 m
Width of Section = 2.500 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 171.800 T
Mxx = 284.270 Tm
Myy = 55.360 Tm

Intercept of Neutral axis : X axis := 7.574 m
:y axis := .614 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 52.35 Kg/cm^2
Stress in Steel due to Loads = 944.22 Kg/cm^2
Percentage of Steel = .56 %
INPUT FILE: CLASS A.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
   7. 1 0.0 0 0;2 0.4 0 0;3 8.8 0 0;4 9.2 0 0
7. MEMBER INCIDENCES
8. 1 1 2 3
10. MEMBER PROPERTY CANADIAN
11. 1 TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
20. LOAD GENERATION 100
21. TYPE 1 -18.8 0. 0. XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 74
24. PRINT SUPPORT REACTION
25. FINISH

SUPPORT REACTIONS -UNIT MTON METER STRUCTURE TYPE = SPACE
-------------------
JOINT  LOAD  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM Z

   2 74    0.00     24.74     0.00     0.00     0.00     0.00
   3 74    0.00     11.66     0.00     0.00     0.00     0.00

*************** END OF LATEST ANALYSIS RESULT ***************
INPUT FILE: 70RW.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
7. 1 0.0 0 0; 2 0.4 0 0; 3 8.8 0 0; 4 9.2 0 0
8. MEMBER INCIDENCES
9. 1 1 2 3
10. MEMBER PROPERTY CANADIAN
11. E TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
20. LOAD GENERATION 175
21. TYPE 1 -13.4 0. 0. XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 30
24. PRINT SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE
______________________________

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>30</td>
<td>0.00</td>
<td>47.72</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>0.00</td>
<td>20.28</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

*************** END OF LATEST ANALYSIS RESULT ***************

25. FINISH
INPUT FILE: 9.2 M SPAN SIDL.STD
1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 09-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER MTON
7. JOINT COORDINATES
   8. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 3 0.4 0.0 0.0; 10 8.4 0.0 0.0; 11 8.8 0.0 0.0; 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0
   9. 22 18.4 0.0 0.0; 23 18.38 0.0 0.0; 24 18.4 0.0 0.0
10. REPEAT ALL 1 0.0 0.0 1.0
11. REPEAT 3 0.0 0.0 2.0
12. REPEAT 1 0.0 0.0 1.0
13. MEMBER INCIDENCES
   14. 1 1 2 23 1 1
   15. 139 1 25 143 1 24
   16. REPEAT 5 23 24
   17. 12 35 58 81 104 127 138 -
   18. REPEAT 5 23 24
   19. CONSTANTS
   20. E 2.7386E+006
   21. POISSON 0.15
   22. DENSITY 2.4 MEMB 1 TO 138
   23. DENSITY 0.0001 MEMB 139 TO 258
   24. MEMBER PROPERTY INDIAN
   25. 2 TO 11 13 TO 22 117 TO 126 128 TO 137 PRIS AX 0.330 IX 0.024 IZ 0.012
   26. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
   27. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
   28. 1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
   29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
   30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
   31. SUPPORTS
   32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
   33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
   34. 35 59 83 107 PINNED
   35. 46 70 94 118 PINNED
   36. MEMBER RELEASE
   37. 1 24 47 93 116 END MZ
   38. 23 46 92 115 138 START MZ
   39. 12 35 58 81 104 127 START MZ
   40. 12 35 58 81 104 127 END MZ
   41. LOAD 1 SIDL
   42. *** CRASH BARRIER
   43. MEMBER LOAD
   44. 25 TO 45 UNI GY -1.0
   45. 94 TO 114 UNI GY -1.0
   46. 25 TO 45 UMOM GX -1.0
   47. 94 TO 114 UMOM GX 1.0
   48. LOAD 2 WEARINGCOAT
   49. MEMBER LOAD
   50. 25 TO 114 UNI GY -0.2464
   51. PERFORM ANALYSIS
52. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70 94 118

<table>
<thead>
<tr>
<th>SUPPORT REACTION LIST</th>
<th>27</th>
<th>51</th>
<th>75</th>
<th>99</th>
<th>38</th>
<th>62</th>
<th>86</th>
<th>110</th>
<th>35</th>
<th>59</th>
<th>83</th>
<th>107</th>
<th>46</th>
<th>70</th>
<th>94</th>
<th>118</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUPPORT REACTIONS -UNIT MTON METE</td>
<td>STRUCTURE TYPE = FLOOR</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>JOINT</td>
<td>LOAD</td>
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************** END OF LATEST ANALYSIS RESULT **************

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  2. START JOB INFORMATION
  3. ENGINEER DATE 09-JUN-06
  4. END JOB INFORMATION
  5. INPUT WIDTH 79
  6. UNIT METER MTON
  7. JOINT COORDINATES
     8. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 3 0.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0
     9. 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0 22 18 0.0 0.0 23 18 0.0 0.0
    10. 24 18.4 0.0 0.0
  11. REPEAT ALL 1 0.0 0.0 1.0
  12. REPEAT 3 0.0 0.0 2.0
  13. REPEAT 1 0.0 0.0 1.0
  14. MEMBER INCIDENCES
     15. 1 1 2 23 1 1
     16. REPEAT 5 23 24
     17. 139 1 25 143 1 24
     18. REPEAT 23 5 1
  19. CONSTANTS
     20. E 2.7386E+006
     21. POISSON 0.15
     22. DENSITY 2.4 MEMB 1 TO 138
     23. DENSITY 0.0001 MEMB 139 TO 258
  24. MEMBER PROPERTY INDIAN
     25. 2 TO 11 13 TO 127 PRIS AX 0.330 IX 0.024 IZ 0.012
     26. 25 TO 34 36 TO 45 94 TO 103 105 TO 114 PRIS AX 0.990 IX 0.072 IZ 0.036
     27. 48 TO 57 59 TO 68 71 TO 80 82 TO 91 PRIS AX 1.320 IX 0.096 IZ 0.048
     28. 1 12 23 24 35 46 47 58 69 70 81 92 93 104 115 -
     29. 116 127 138 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
     30. 139 TO 258 PRIS AX 0.0001 IX 0.0002 IZ 0.0001
  31. SUPPORTS
     32. 27 51 75 99 ENFORCED BUT FX FZ MX MY MZ
     33. 38 62 86 110 ENFORCED BUT FX FZ MX MY MZ
     34. 35 59 83 107 PINNED
     35. 46 70 94 118 PINNED
  36. MEMBER RELEASE
     37. 1 24 47 93 116 END MZ
     38. 23 46 92 115 138 START MZ
     39. 12 35 58 81 104 127 START MZ
     40. 12 35 58 81 104 127 END MZ
  41. LOAD 1 SELFWEIGHT
     42. SELF Y -1
     43. PERFORM ANALYSIS
     44. PRINT SUPPORT REACTION LIST 27 51 75 99 38 62 86 110 35 59 83 107 46 70 94 118
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************** END OF LATEST ANALYSIS RESULT **************

45. FINISH
DESIGN OF SUBSTRUCTURE (PIER)
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<td>M.S.L</td>
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</tr>
<tr>
<td>Founding Level</td>
<td>204.800 m</td>
</tr>
<tr>
<td>Pier cap top level</td>
<td>215.438 m</td>
</tr>
<tr>
<td>Gross safe bearing capacity</td>
<td>29.715 t/m²</td>
</tr>
</tbody>
</table>

**Live Load**
(a) Class A two Lane
(b) Class 70R wheeled

Bearing: neoprene but during raising Tar paper bearing may be kept.

Seismic zone = II

The following codes are used for the design of substructure:
1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000
As per clause 214.2 of IRC:6, horizontal braking force $F_h$, for each span is:

For Class A 2 lane: $F_h = \left( 0.2 \times 59.2 \right) = 11.836$ t

For class 70R wheeled: $F_h = \left( 0.2 \times 66.1 \right) = 13.210$ t

**Summary of Longitudinal Forces:**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 2 lane</td>
<td>5.92</td>
</tr>
<tr>
<td>70R</td>
<td>6.61</td>
</tr>
<tr>
<td>Span dislodged condition</td>
<td>0.00</td>
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</tbody>
</table>

Dead load

<table>
<thead>
<tr>
<th></th>
<th>0.5</th>
<th>7.50</th>
<th>0.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load of slab</td>
<td>14.561 t/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wearing coat</td>
<td>0.924 t/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crashbarrier</td>
<td>1 t/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead load reaction</td>
<td>66.978 t</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sidl reaction</td>
<td>13.45 t</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### H.F.L. Condition with L.L

**a) Vertical load and their moments about C/L of Foundation base.**

<table>
<thead>
<tr>
<th>1 D.L. Reaction</th>
<th>P</th>
<th>eL</th>
<th>ML</th>
<th>eT</th>
<th>MT</th>
</tr>
</thead>
<tbody>
<tr>
<td>a Left span</td>
<td>66.978</td>
<td>-0.400</td>
<td>-26.791</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>66.978</td>
<td>0.400</td>
<td>26.791</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

**2 S.I.D.L**

<table>
<thead>
<tr>
<th>a Left span</th>
<th>b Right span</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.450</td>
<td>13.450</td>
</tr>
</tbody>
</table>

**Notes:**
- L.L: Left Lane
- D.L: Design Load
- Reaction: Reaction force
- MI: Moment about C/L
- eT: End moment

### 3 L.L. Max Reaction case

<table>
<thead>
<tr>
<th>1 L.L. Max Reaction case</th>
<th>2 70R Wheeled</th>
<th>a Left span</th>
<th>33.600</th>
<th>-0.400</th>
<th>-14.400</th>
<th>1.160</th>
<th>41.760</th>
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</thead>
<tbody>
<tr>
<td>b Right span</td>
<td>30.050</td>
<td>0.400</td>
<td>12.020</td>
<td>1.160</td>
<td>34.858</td>
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</tbody>
</table>

**4 L.L. Max Long. Moment case**

<table>
<thead>
<tr>
<th>1 L.L. Max Long. Moment case</th>
<th>2 70R Wheeled</th>
<th>a Left span</th>
<th>6.880</th>
<th>-0.400</th>
<th>-2.752</th>
<th>0.700</th>
<th>4.816</th>
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</thead>
<tbody>
<tr>
<td>b Right span</td>
<td>47.740</td>
<td>0.400</td>
<td>19.096</td>
<td>0.700</td>
<td>33.418</td>
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</tr>
</tbody>
</table>

### Summary

- Max Reaction case: 226.907 1.420 76.618
- Max Long. moment case: 208.007 18.860 54.694

### II Substructure

<table>
<thead>
<tr>
<th>V</th>
<th>p</th>
<th>P</th>
<th>eL</th>
<th>ML</th>
<th>eT</th>
<th>MT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a Left span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>-0.400</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>2.550</td>
<td>2.400</td>
<td>6.120</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.700</td>
<td>2.400</td>
<td>13.680</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3 Pier shaft up to H.F.L below H.F.L</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4 Footing</td>
<td>23.224</td>
<td>1.400</td>
<td>32.513</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>5 Earth above footing</td>
<td>113.752</td>
<td>1.000</td>
<td>113.752</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>6 Water current</td>
<td>0.000</td>
<td>0.000</td>
<td>17.615</td>
<td>0.000</td>
<td>7.040</td>
<td></td>
</tr>
</tbody>
</table>

| 187.643 | 17.615 | 7.040 |

### b) Horizontal Forces and Moments withrespect to Base

<table>
<thead>
<tr>
<th>HL</th>
<th>HT</th>
<th>eL</th>
<th>ML</th>
<th>eT</th>
<th>MT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Longitudinal Forces at bearing level</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.605</td>
<td>0.000</td>
<td>10.663</td>
<td>70.429</td>
<td>10.638</td>
<td>0.000</td>
</tr>
<tr>
<td>6.605</td>
<td>0.000</td>
<td>70.429</td>
<td>0.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Summary**

- Max Reaction case: 414.551 395.651
- Max Long. Moment case: 89.464 106.904
- Max Long. Moment case: 83.658 61.734
Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{414.551}{31.900} + \frac{89.464}{30.837} + \frac{83.658}{29.242} = 18.757 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{414.551}{31.900} - \frac{89.464}{30.837} - \frac{83.658}{29.242} = 7.233 \text{ t/m}^2
\]

Max Long

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{395.651}{31.900} + \frac{106.904}{30.837} + \frac{61.734}{29.242} = 17.981 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{395.651}{31.900} - \frac{106.904}{30.837} - \frac{61.734}{29.242} = 6.825 \text{ t/m}^2
\]

H.F.L Condition with L.L

(a) Vertical load and their moments about Pier Shaft bottom

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>(e_L)</th>
<th>(M_L)</th>
<th>(e_T)</th>
<th>(M_T)</th>
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</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>a Left span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>-0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>2.550</td>
<td>2.400</td>
<td>6.120</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.700</td>
<td>2.400</td>
<td>13.680</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3 Pier shaft up to H.F.L</td>
<td>0.000</td>
<td>1.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>15.413</td>
<td>1.400</td>
<td>21.579</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4 Water current</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>41.379</td>
<td>17.593</td>
<td>6.976</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

(b) Horizontal Forces and Moments with respect to Pier Shaft bottom

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_L)</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>6.605</td>
</tr>
<tr>
<td>6.605</td>
</tr>
</tbody>
</table>
### Gross Pressure Diagram

**Summary for design of Pier**

<table>
<thead>
<tr>
<th></th>
<th>Max Reaction case</th>
<th>Max Long.moment case</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P )</td>
<td>268.286 ( t )</td>
<td>249.386 ( t )</td>
</tr>
<tr>
<td>( M_L )</td>
<td>82.507 ( t \cdot m )</td>
<td>99.947 ( t \cdot m )</td>
</tr>
<tr>
<td>( M_T )</td>
<td>83.594 ( t \cdot m )</td>
<td>61.670 ( t \cdot m )</td>
</tr>
</tbody>
</table>

**H.F.L. Condition One. Span dislodged**

*a) Vertical load and their moments about C/L of Foundation base.*

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>( V ) ( \rho )</th>
<th>( P )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>a Left span</td>
<td>0.000</td>
<td>2.400</td>
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<td>0.000</td>
<td>0.000</td>
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<tr>
<td>b Right span</td>
<td>0.000</td>
<td>2.400</td>
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<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>2.550</td>
<td>2.400</td>
<td>6.120</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.700</td>
<td>2.400</td>
<td>13.680</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>up to H.F.L</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>15.413</td>
<td>1.400</td>
<td>21.579</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>4 Footing</td>
<td>23.224</td>
<td>1.400</td>
<td>32.513</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>5 Earth above</td>
<td>113.752</td>
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<td>113.752</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
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<tr>
<td>6 Water current</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

|                | 187.643          | 17.615 | 7.040 |

**Summary**

|                | 348.501          | 44.407 | 7.040 |

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
P_{\text{max}} = \frac{348.501}{31.900} + \frac{44.407}{30.837} + \frac{7.040}{29.242} = 12.606 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
P_{\text{min}} = \frac{348.501}{31.900} - \frac{44.407}{30.837} - \frac{7.040}{29.242} = 9.244 \text{ t/m}^2
\]
### H.F.L. Condition One. Span dislodged

**a) Vertical load and their moments about Abutment Shaft bottom.**

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>V</th>
<th>$p$</th>
<th>$P$</th>
<th>$e_L$</th>
<th>$M_L$</th>
<th>$e_T$</th>
<th>$M_T$</th>
</tr>
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<tbody>
<tr>
<td>1 Pedestal</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>-0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>b Right span</td>
<td>0.000</td>
<td>2.400</td>
<td>0.000</td>
<td>0.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>2.550</td>
<td>2.400</td>
<td>6.120</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>3 Pier cap Tr</td>
<td>5.700</td>
<td>2.400</td>
<td>13.680</td>
<td>0.000</td>
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</tr>
<tr>
<td>3 Pier shaft</td>
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<td>2.400</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
<td>up to H.F.L</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>below H.F.L</td>
<td>15.413</td>
<td>1.400</td>
<td>21.579</td>
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<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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<tr>
<td>4 Water current</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

Summary for design of Pier:

- $P = 202.236$ t
- $M_L = 44.384$ t-m
- $M_T = 6.976$ t-m
## Dry Condition with L.L.

### a) Vertical load and their moments about C/L of Foundation base.

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>D.L. Reaction</td>
<td>P</td>
<td>eL</td>
<td>ML</td>
<td>eT</td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>66.98</td>
<td>-0.4</td>
<td>-26.791</td>
<td>0</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>66.98</td>
<td>0.4</td>
<td>26.791</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>S.I.D.L</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>13.45</td>
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<td>-5.3802</td>
<td>0</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>13.45</td>
<td>0.4</td>
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</tbody>
</table>

#### 4 L.L. Max Reaction case

<p>| | | | | | |</p>
<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>a</td>
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<td>70R Wheeled + Class A</td>
<td>36</td>
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<tr>
<td></td>
<td>Class A 3 Lane</td>
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<td>-13.44</td>
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</tr>
<tr>
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<td>Right span</td>
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<td>0.4</td>
<td>12.02</td>
</tr>
<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>25.58</td>
<td>0.4</td>
<td>10.232</td>
<td></td>
</tr>
</tbody>
</table>

#### 4 L.L. Max Longitudinal Moment case

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Left span</td>
<td>70R Wheeled + Class A</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>6.88</td>
<td>-0.4</td>
<td>-2.752</td>
<td></td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>70R Wheeled + Class A</td>
<td>47.15</td>
<td>0.4</td>
<td>18.86</td>
</tr>
<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>47.74</td>
<td>0.4</td>
<td>19.096</td>
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</tbody>
</table>

**Max Reaction case** 226.91  
**Max Longitudinal Moment** 208.01

### II Substructure

<table>
<thead>
<tr>
<th></th>
<th>V</th>
<th>P</th>
<th>eL</th>
<th>ML</th>
<th>eT</th>
<th>MT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pedestal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>Left span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Pier cap R</td>
<td>2.55</td>
<td>2.4</td>
<td>6.12</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>5.7</td>
<td>2.4</td>
<td>13.68</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Pier shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>up to G.L</td>
<td>8.8386446</td>
<td>2.4</td>
<td>21.213</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>below G.L</td>
<td>6.574671</td>
<td>2.4</td>
<td>15.779</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Footing</td>
<td>23.22375</td>
<td>2.4</td>
<td>55.737</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Earth above footing</td>
<td>113.75158</td>
<td>2</td>
<td>227.5</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

**340.03** 0 0
b) Horizontal Forces and Moments with respect to Base

### 1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th></th>
<th>( H_L )</th>
<th>( H_T )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6.61</td>
<td>0</td>
<td>10.663</td>
<td>70.429</td>
<td>10.638</td>
<td>0</td>
</tr>
</tbody>
</table>

#### Summary

- Max Reaction = 566.94 t
- Max Longitudinal Moment = 89.289 t-m
- Max Transverse Moment = 54.694 t-m

<table>
<thead>
<tr>
<th>A</th>
<th>Z_L</th>
<th>Z_T</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>31.9</td>
<td>30.837</td>
</tr>
<tr>
<td></td>
<td>29.242</td>
<td></td>
</tr>
</tbody>
</table>

Max reaction case

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P + M_L + M_T}{A} = \frac{566.93952 + 71.849 + 76.618}{31.9} = 22.723 \, \text{t/m}^2
\]

\[
P_{\text{min}} = \frac{P - M_L - M_T}{A} = \frac{566.93952 - 71.849 - 76.618}{31.9} = 12.822 \, \text{t/m}^2
\]

Max Longitudinal moment case

\[
P_{\text{max}} = \frac{P + M_L + M_T}{A} = \frac{548.03952 + 89.289 + 54.694}{31.9} = 21.946 \, \text{t/m}^2
\]

\[
P_{\text{min}} = \frac{P - M_L - M_T}{A} = \frac{548.03952 - 89.289 - 54.694}{31.9} = 12.414 \, \text{t/m}^2
\]
Design of Footing in Transverse direction

![Diagram of Footing in Transverse direction]

Bending moment at the face of pier \( b-b = 239.82 \) t-m
Footing is checked for shear at distance of \( d \) from face pier
Effective depth \( d \) of footing \( = 0.9625 \) m
Total depth of section at distance \( d \) from pier \( = 0.7601 \) m
Effective depth at distance \( d \) from face of pier \( = 0.6691 \) m

Bending moment at a distance of \( d \) from face of pier \( = 42.06 \) t-m
Shear force at a distance \( d \) from face of pier \( = 120.27 \) t
Net shear force at a distance \( d \) from face of pier \( = 101.34 \) t (after correction)

Design for Flexure:

\[ M = 25 \quad Fe = 415 \]

Permissible compressive stress \( \sigma_{cbc} = 8.33 \text{ N/mm}^2 \)
Permissible tensile stress \( \sigma_t = 200 \text{ N/mm}^2 \)
\( m = 10 \)
\( k = 0.29 \)
\( j = 0.9 \)
\( Q = 1.1053 \)
width \( b = 5.50 \)

Effective depth required \( d = 628.08 \) mm

Effective depth provided = 962.5 mm

\( A_{st} \) Required = 2511.3 mm²

Provide 6 nos 25 dia bars/m width

**Check for shear**

Shear stress at effective depth from pier = 27.538 t/m²

% of steel provided = 0.2392

Permissible shear stress = 23.46 t/m²

**Minimum Shear Reinforcement**:

\[
\frac{A_{ov}}{b \times s} = \frac{0.4}{0.87 \times f_y}
\]

\[
\frac{A_{ov}}{s} = \frac{0.4 \times b}{0.87 \times f_y} = 6.0933 \text{ cm}^2/\text{m}
\]

Provide 2 Legged 10 @ 200 = 7.854 cm²/m

**Dry Condition with L.L.**

a) *Vertical load and their moments about Pier Shaft bottom*

<table>
<thead>
<tr>
<th>II</th>
<th>Substructure</th>
<th>V</th>
<th>( \rho )</th>
<th>P</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
<td>a Left span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>b Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>2.55</td>
<td>2.4</td>
<td>6.12</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Pier cap Tr</td>
<td>5.7</td>
<td>2.4</td>
<td>13.68</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td>up to G.L</td>
<td>8.8386446</td>
<td>2.4</td>
<td>21.213</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>below G.L</td>
<td>6.574671</td>
<td>2.4</td>
<td>15.779</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

56.792 0 0
b) *Horizontal Forces and Moments with respect to Pier Shaft bottom*

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_L</td>
</tr>
<tr>
<td>6.61</td>
</tr>
<tr>
<td>6.61</td>
</tr>
</tbody>
</table>

*Summary for design of Pier*

Max Reaction case: Max Longitudinal moment

- P = 283.7 t
- M_L = 64.914 t-m
- M_T = 76.618 t-m

**Dry Condition One: Span dislodged**

*a) Vertical load and their moments about C/L of Foundation base.*

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_L</td>
</tr>
<tr>
<td>0.00</td>
</tr>
</tbody>
</table>

*Summary*

- P = 500.89 t
- M_L = 26.791 t-m
- M_T = 0 t-m

**Check for Maximum Allowable Base Pressure:**

\[
P_{\text{max}} = \frac{P + M_L + M_T}{A + Z_L + Z_T} = \frac{500.88952 + 26.791 + 0}{31.9 + 30.837 + 29.242} = \frac{547.68051}{92.03} = 6.071 \text{ t/m}^2
\]

\[
P_{\text{min}} = \frac{P - M_L - M_T}{A + Z_L + Z_T} = \frac{500.88952 - 26.791 - 0}{31.9 + 30.837 + 29.242} = \frac{474.09851}{92.03} = 5.173 \text{ t/m}^2
\]

**Dry Condition One: Span dislodged**

*a) Vertical load and their moments about Abutment Shaft bottom.*

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_L</td>
</tr>
<tr>
<td>0.00</td>
</tr>
</tbody>
</table>

*Summary for design of Pier*

- P = 217.65 t
- M_L = 26.791 t-m
- M_T = 0 t-m
Summary of Loads at Pier Shaft bottom:

1 DRY condition
   With L.L.  283.70  64.91  76.62
   Span dislodged  217.65  26.79  0.00

2 H.F.L.
   With L.L.  268.29  82.51  83.59
   Span dislodged  202.24  44.38  6.98
Dimensions of Substructure & Foundation

1 Pedestal

Length = 0.7
Width = 0.55
Height = 0
Volume = 0

2 Pier cap

a Top uniform portion

Width = 1.00
Depth = 0.3
Length = 8.5
Volume = 2.55 m³

b Bottom trapezoidal portion

Width = 1
Depth = 1.2
Length = 8.5
Volume = 5.7 m³

Area at level

Area at level 215.138 m² = 8.5 m x 1 = 8.5 m²
Area at level 213.938 m² = 1 m x 1 = 1 m² = 8.25 m³
3 Pier shaft
Dimension of pier shaft in longitudinal direction = 0.82
Dimension of pier shaft in Transverse direction
R = 1.68  Area = 1.91 m²
C = 0.41
Height of pier shaft = 8.09
Height above Ground level = 4.638 m  Volume = 8.83864
Height below Ground level = 3.450 m  Volume = 6.57467
= 15.4133 m³
Height above H.F.L = 0.000 m  Volume = 0
Height below H.F.L = 8.09 m  Volume = 15.4133
= 15.4133 m³
4 Pedestal at footing top
Width = 0.82
Length = 5.50  Volume = 0 m³
Height = 0
5 Footing
at foundation level
Width = 5.800 m
Length = 5.50 m
Thickness at Root = 1.05 m
Thickness Gross Pressure Diagram = 0.3 m
Volume = 23.2238 m³
Net Pressure diagram

Volume of Overburdened earth below ground level

Total volume $5.800 \times 5.500 \times 4.500 = 143.55 \text{ m}^3$

Net volume below ground level $= 113.752 \text{ m}^3$

Sectional Properties of Footing

$A = 5.800 \times 5.500 = 31.900 \text{ m}^2$

$Z_L = 5.500 \times 5.60667 = 30.837 \text{ m}^3$

$Z_T = 5.800 \times 5.04167 = 29.242 \text{ m}^3$

Forces and Moments due to Watercurrent Force

Intensity of water current pressure $= 52 \text{ k V}^2 \text{ kg/m}^2$

Assuming 20 degree variation in water current direction

Water current Pressure in Transverse direction

<table>
<thead>
<tr>
<th>R.L</th>
<th>k</th>
<th>Pressure = $52 \text{ k V}^2 \cos 20$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H.F.L</td>
<td>9.00</td>
<td>290.252257</td>
</tr>
<tr>
<td>Pier Shaft bottom</td>
<td>205.850</td>
<td>0.66</td>
</tr>
<tr>
<td>Top of footing</td>
<td>205.850</td>
<td>1.5</td>
</tr>
<tr>
<td>Bottom of footing</td>
<td>204.800</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Water current Force in Transverse direction

<table>
<thead>
<tr>
<th>Component</th>
<th>Length</th>
<th>Height</th>
<th>Force</th>
<th>HT of force</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier</td>
<td>0.82</td>
<td>8.322</td>
<td>1.03555</td>
<td>6.73646</td>
<td>6.97593 t-m</td>
</tr>
<tr>
<td>Top of footing</td>
<td>5.800</td>
<td>1.050</td>
<td>0.0917</td>
<td>0.69991</td>
<td>0.06418 t-m</td>
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</tbody>
</table>
### Water Current Pressure in Longitudinal direction

<table>
<thead>
<tr>
<th>Component</th>
<th>R.L</th>
<th>k</th>
<th>Pressure</th>
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</thead>
<tbody>
<tr>
<td>H.F.L</td>
<td>214.172</td>
<td>1.5</td>
<td>240.098141</td>
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<tr>
<td>Pier Shaft bottom</td>
<td>205.850</td>
<td>1.5</td>
<td>10.9586078</td>
</tr>
<tr>
<td>Top of footing</td>
<td>205.850</td>
<td>1.5</td>
<td>10.9586078</td>
</tr>
<tr>
<td>Bottom of footing</td>
<td>204.800</td>
<td>1.5</td>
<td>0.00266776</td>
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</table>

### Water current Force in Longitudinal direction

<table>
<thead>
<tr>
<th>Component</th>
<th>Length (m)</th>
<th>Height (m)</th>
<th>Force (t)</th>
<th>HT of force</th>
<th>Moment (t-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier</td>
<td>2.50</td>
<td>8.322</td>
<td>2.61162</td>
<td>6.73646</td>
<td>17.5931</td>
</tr>
<tr>
<td>Top of footing</td>
<td>5.500</td>
<td>1.050</td>
<td>0.03165</td>
<td>0.69991</td>
<td>0.02215</td>
</tr>
</tbody>
</table>

### Summary

<table>
<thead>
<tr>
<th>Summary</th>
<th>F</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse direction</td>
<td>1.03555</td>
<td>6.97592582 t-m</td>
</tr>
<tr>
<td>Longitudinal direction</td>
<td>2.61162</td>
<td>17.5930526 t-m</td>
</tr>
</tbody>
</table>
PIER SHAFT .case 1 L.W.L (MAX REACTION CASE)

Depth of Section = 0.820 m
Width of Section = 2.500 m

along width-compression face- no of bar: 18 tension face- no of bar: 18
Dia (mm) 20 20
Cover (cm) 5.00 5.0

along depth-compression face- no of bar: 6 tension face- no of bar: 6
Dia (mm) 16 16
Cover (cm) 5.00 5.0

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 283.700 T
Mxx = 64.914 Tm
Myy = 76.618 Tm

Intercept of Neutral axis : X axis : = 5.253 m
: y axis : = 0.757 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 47.85 Kg/cm^2
Stress in Steel due to Loads = 231.46 Kg/cm^2
Percentage of Steel = 67 %

PIER SHAFT .case 2 H.F.L (MAX REACTION CASE)

Depth of Section = 0.820 m
Width of Section = 2.500 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 268.286 T
Mxx = 82.507 Tm
Myy = 83.594 Tm

Intercept of Neutral axis : X axis : = 4.959 m
: y axis : = 0.594 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 61.64 Kg/cm^2
Stress in Steel due to Loads = 486.49 Kg/cm^2
Percentage of Steel = 67 %
PIER SHAFT .case 3 L.W.L (SPAN DISLODGED)

Depth of Section = 0.820 m
Width of Section = 2.500 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 217.650 T
Mxx = 26.791 Tm
Myy = 0.010 Tm

Intercept of Neutral axis : X axis : = ******** m
: y axis : = 0.891 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 18.55 Kg/cm^2
Stress in Steel due to Loads = -25.21 Kg/cm^2
Percentage of Steel = 0.67%

PIER SHAFT .case 4 H.F.L (SPAN DISLODGED)

Depth of Section = 0.820 m
Width of Section = 2.500 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2000.00 Kg/cm^2

Axial Load = 202.236 T
Mxx = 44.384 Tm
Myy = 6.976 Tm

Intercept of Neutral axis : X axis : = 33.465 m
: y axis : = 0.655 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 25.31 Kg/cm^2
Stress in Steel due to Loads = 63.11 Kg/cm^2
Percentage of Steel = 0.67%
For Outer bearing

\[
a = 2.125 \\
D = 1.5
\]

Available effective depth
using 25 mm dia of bars

\[
d = 1.5 - 0.05 - 0.025 = 1.425 \text{ m}
\]

\[
a \frac{2.125}{1.425} = 1.4912 > 1
\]

Pier cap designed as Cantilever
Loads on bearing

Bearings A1 & B1 are effective.

<table>
<thead>
<tr>
<th>Load</th>
<th>A1</th>
<th>B1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>16.745</td>
<td>16.745</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3625</td>
<td>3.3625</td>
</tr>
<tr>
<td>LL</td>
<td>7.5125</td>
<td>9</td>
</tr>
</tbody>
</table>

Due to Transverse moment

\[
A1 = 30.05 \times 1.16 = 34.858 \\
B1 = 36 \times 1.16 = 41.76
\]

\[
A1 = \frac{34.858 \times 3.38}{25.31} = 4.6477 \text{ t} \\
B1 = \frac{41.76 \times 3.38}{25.31} = 5.568 \text{ t}
\]

Summary of loads from superstructure:

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>Left span</th>
<th>Right span</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>16.745</td>
<td>16.745</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3625</td>
<td>3.3625</td>
</tr>
<tr>
<td>LL</td>
<td>5.568</td>
<td>4.6477</td>
</tr>
<tr>
<td>9</td>
<td>7.5125</td>
<td></td>
</tr>
<tr>
<td>34.675</td>
<td>32.267</td>
<td></td>
</tr>
</tbody>
</table>

Selfweight of piercap upto section through outerbearing:

\[
= 0.3 + 1.50 \frac{3.00}{2.00} \times 1 \times 2.4 = 6.48 \text{ t}
\]

Distance of c.g from section = 1.1667 m

Calculation of Cantilevermoment at bearing section

Moment at the face due to load from outerbearing

\[
= 66.942 \times 2.125 = 142.25225 \text{ t-m}
\]

Due to selfweight of pier cap

\[
= 6.48 \times 1.1667 = 7.56 \text{ t-m}
\]

Total moment at the face of support

\[
= 149.81 \text{ t-m}
\]

Torsion at outerbearing section:

\[
= 34.675 - 32.267 = 2.4078
\]

Eccentricity = 0.4

Moment = 0.9631 t-m
**Longitudinal Reinforcement:**

\[
M = 149.81 \text{ t-m}
\]

\[
\text{For, } M = 25 \text{ & } Fe = 415
\]

\[
\sigma_{cbc} = 8.33 \text{ N/mm}^2
\]

\[
m = 10
\]

\[
\sigma_{st} = 200 \text{ N/mm}^2
\]

\[
K = 0.294
\]

\[
j = 0.902
\]

\[
R = 1.105
\]

Effective depth required

\[
= \frac{149.81 \times 10^7}{1.105 \times 1000}
\]

\[
= 1164.2 \text{ < 1425 O.K}
\]

\[
A_{st} \text{ required} = \frac{149.81}{200 \times 0.902 \times 1425}
\]

\[
= 5827.9363 \text{ mm}^2
\]

Provide 7 nos 25 mm dia bars in 2 layers 6872.2 mm²

This reinforcement will provided in full length of pier cap.

**Check for Shear:**

At bearing section

Available Effective depth at root \(= 1425 \text{ mm}\)

Downward Load from Superstructure \(= 34.675 + 32.267 = 66.942 \text{ t}\)

Downward Load due to self wt of pier cap \(= 6.48 \text{ t}\)

Total downward Load \(= 73.422 \text{ t}\)

After shear correction, \(S.F = \frac{V + \frac{M \tan \beta}{d}}{1.425}\)

\[
= 73.422 + \frac{149.81 \times 0.4}{1.425}
\]

\[
= 31.369673 \text{ t}
\]

Equivalent shear \(IRC:21-2000,cl:304.2.3.1\)

\[
b = \frac{1 + 1}{2} = 1 \text{ m}
\]

\[
V = 31.37 \text{ t}
\]

Shear stress \(= \frac{31.37 \times 10000}{1000 \times 1425} = 0.2201 \text{ N/mm}^2\)

Area of tension reinforcement \(= 0.4823 \%\)

\(\tau_c = 0.2966 \text{ N/mm}^2\)

Provide minimum shear
For Outer bearing

\[ a = 2.125 \]
\[ D = 1.5 \]

Available effective depth using 25 mm dia of bars

\[ d = 1.5 - 0.05 - 0.025 = 1.425 \text{ m} \]

\[ \frac{a}{d} = \frac{2.125}{1.425} = 1.4912 > 1 \]

\[ = \text{Pier cap designed as Cantilever} \]
Loads on bearing

Bearings A1 & B1 are effective.

<table>
<thead>
<tr>
<th></th>
<th>A1</th>
<th>B1</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>16.745</td>
<td>16.745</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3625</td>
<td>3.3625</td>
</tr>
<tr>
<td>LL</td>
<td>11.93</td>
<td>0</td>
</tr>
</tbody>
</table>

**due to Transverse moment**

\[
A1 = 0 \times 1.16 = 0 \\
B1 = 47.72 \times 1.16 = 55.355
\]

\[
A1 = \frac{0 \times 3.38}{25.31} = 0 \text{ t} \\
B1 = \frac{55.355 \times 3.38}{25.31} = 7.3807 \text{ t}
\]

---

**Summary of loads from superstructure:**

<table>
<thead>
<tr>
<th>Loads on Outer bearings</th>
<th>Left span</th>
<th>Right span</th>
</tr>
</thead>
<tbody>
<tr>
<td>DL</td>
<td>16.745</td>
<td>16.745</td>
</tr>
<tr>
<td>SIDL</td>
<td>3.3625</td>
<td>3.3625</td>
</tr>
<tr>
<td>LL</td>
<td>7.3807</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>11.93</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td><strong>39.418</strong></td>
<td><strong>20.107</strong></td>
</tr>
</tbody>
</table>

---

Selfweight of piercap upto section through outerbearing:

\[
= \frac{0.3 + 1.50 \times 3.00 \times 1 \times 2.4}{2.00} = 6.48 \text{ t}
\]

Distance of c.g from section = 1.1667 m

**Calculation of Cantilevermoment at bearing section**

Moment at the face due to load from outerbearing

\[
= 59.525 \times 2.125 = 126.48997 \text{ t-m}
\]

Due to selfweight of pier cap

\[
= 6.48 \times 1.1667 = 7.56 \text{ t-m}
\]

Total moment at the face of support

\[
= 134.05 \text{ t-m}
\]

Torsion at outerbearing section:

\[
= 39.418 - 20.107 = 19.311
\]

Eccentricity = 0.4

Moment = 7.7243 t-m
Longitudinal Reinforcement:

\[ M_e = M + M_t \]

\[ M_t = \frac{T \times (1 + D/b)}{1.7} \]

\[ = 6.8155 \text{ t-m} \]

\[ M_e = 134.05 + 6.8155 = 140.87 \text{ t-m} \]

For, \( M - 25 \) & \( Fe - 415 \)

\[ \sigma_{cbc} = 8.33 \text{ N/mm}^2 \]

\[ m = 10 \]

\[ \sigma_{st} = 200 \text{ N/mm}^2 \]

\[ K = 0.294 \]

\[ j = 0.902 \]

\[ R = 1.105 \]

Effective depth required

\[ = \frac{140.87 \times 10^7}{1.105 \times 1000} \]

\[ = 1128.9 < 1425 \text{ O.K} \]

\[ A_d \text{ required} = \frac{140.87}{200 \times 0.902 \times 1425} \]

\[ = 5479.8941 \text{ mm}^2 \]

Provide 7 nos 25 mm dia bars in 2 layers 6872.2 mm²

This reinforcement will provided in full length of pier cap.

Check for Shear:

At bearing section

Available Effective depth at root = 1425 mm

Downward Load from Superstructure = 39.418 + 20.107

= 59.525 t

Downward Load due to self wt of pier cap = 6.48 t

Total downward Load = 66.005 t

After shear correction, \( S.F \)

\[ = \frac{V + M \tan \beta}{d} \]

\[ = 66.005 + \frac{134.05 \times 0.4}{1.425} \]

\[ = 28.376631 \text{ t} \]

Equivalent shear IRC:21-2000,cl:304.2.3.1

\[ b = \frac{1 + 1}{2} = 1 \text{ m} \]

\[ V_e = V + 1.6 \times \frac{T}{b} = 40.735 \text{ t} \]

Shear stress

\[ = \frac{40.735 \times 10000}{1000 \times 1425} = 0.2859 \text{ N/mm}^2 \]

Area of tension reinforcement

\[ \tau_c = 0.4823 \% \]

\[ = 0.2966 \text{ N/mm}^2 \]
INPUT FILE: 9.2 M SPAN CLASS A 70R.STD

1. STAAD FLOOR
2. START JOB INFORMATION
3. ENGINEER DATE 09-JUN-06
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
   9. 1 0.0 0.0 0.0; 2 0.02 0.0 0.0; 30.4 0.0 0.0 10 8.4 0.0 0.0; 11 8.8 0.0 0.0; 12 9.18 0.0 0.0; 13 9.22 0.0 0.0; 14 9.6 0.0 0.0 22 18 0.0 0.0; 23 18.38 0.0 0.0; 24 18.4 0.0 0.0
10. MEMBER INCIDENCES
11. CONSTANTS
   12. E 2.7386E+006
   13. POISSON 0.15
   14. DENSITY 2.4 ALL
   15. MEMBER PROPERTY INDIAN
      16. 1 TO 23 PRIS YD 1 ZD 1
17. SUPPORTS
   18. 3 22 FIXED BUT FX MZ
   19. 11 14 PINNED
   20. MEMBER RELEASE
      21. 1 START FX FY MZ
      22. 12 START FX FY MZ
      23. 23 END FX FY MZ
24. DEFINE MOVING LOAD
25. TYPE 1 LOAD 8 12 12 17 17 17 17
26. DIST 3.96 1.52 2.13 1.37 3.05 1.37
27. ** CLASS 70 RW
28. LOAD GENERATION 82
29. TYPE 1 -13.4 0 0 XINC 0.5
30. PERFORM ANALYSIS
31. ** MAX REA
32. LOAD LIST 29
33. PRINT SUPPORT REACTION
   34. SUPPORT REACTION
      35. JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM-Z
         36. --------------------------
         37. 3 29 0.00 18.95 0.00 0.00 0.00 0.00
         38. 22 29 0.00 15.00 0.00 0.00 0.00 0.00
         39. 11 29 0.00 30.05 0.00 0.00 0.00 0.00
         40. 14 29 0.00 36.00 0.00 0.00 0.00 0.00
         41. ** MAX MOMENT
         42. LOAD LIST 19
         43. PRINT SUPPORT REACTION
         44. 3 19 0.00 32.85 0.00 0.00 0.00 0.00
         45. 22 19 0.00 0.00 0.00 0.00 0.00 0.00
         46. 11 19 0.00 47.15 0.00 0.00 0.00 0.00
         47. 14 19 0.00 0.00 0.00 0.00 0.00 0.00
         48. FINISH
DESIGN OF SUPERSTRUCTURE
**DESIGN OF SOLID SLAB SUPERSTRUCTURE**

**DESIGN DATA**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (c/c of exp. joint)</td>
<td>9.20 m</td>
</tr>
<tr>
<td>Eff. Span (c/c brgs.)</td>
<td>8.80 m</td>
</tr>
<tr>
<td>Overall Width of deck slab</td>
<td>8.50 m</td>
</tr>
<tr>
<td>Width of carriageway</td>
<td>7.50 m</td>
</tr>
<tr>
<td>Width of crash barrier</td>
<td>0.50 m</td>
</tr>
<tr>
<td>Thickness of deck slab</td>
<td>0.66 m</td>
</tr>
<tr>
<td>Thickness of wearing coat</td>
<td>0.056 m</td>
</tr>
<tr>
<td>Unit wt of concrete</td>
<td>2.40 t/m³</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M 25</td>
</tr>
<tr>
<td>Grade of Reinforcement</td>
<td>Fe 415 (HYSD)</td>
</tr>
<tr>
<td>Live Load</td>
<td>One Lane of 70R Wheeled/ Tracked &amp; One Lane of Class A</td>
</tr>
<tr>
<td>Permissible Compressive stress in Concrete</td>
<td>850 t/m²</td>
</tr>
<tr>
<td>Permissible Tensile stress in Steel</td>
<td>20400 t/m²</td>
</tr>
<tr>
<td>Modular ratio, ( m )</td>
<td>10</td>
</tr>
<tr>
<td>Lever arm factor, ( j )</td>
<td>0.902</td>
</tr>
<tr>
<td>Moment of resistance coefficient, ( Q )</td>
<td>112.75 t/m²</td>
</tr>
</tbody>
</table>

**CALCULATION OF BENDING MOMENTS AT MID SPAN:**

**DUE TO DEAD LOAD (Per metre width):**

- Deck slab:
  \[ 0.660 \times 2.40 = 1.584 \text{ t/m} \]
- BM at mid span:
  \[ 1.584 \times 8.80 \times \frac{2}{8} = 15.333 \text{ t/m} \]

**DUE TO SUPERIMPOSED DEAD LOAD (Per metre width):**

- Wearing coat ( @ 0.2t/m²):
  \[ 0.200 \text{ t/m} \]
- Crash barrier ( @ 0.9 t/m on each side):
  \[ 2 \times 0.90 / 8.50 = 0.212 \text{ t/m} \]

\[ \text{Total BM at mid span = 0.412 t/m} \]

**DUE TO LIVE LOAD:**

The Superstructure has been analysed for Live load-Class 70R wheeled, class 70R tracked and Class A loading. Load positions for maximum bending moments are evaluated at (i) mid span (L/2) and (ii) at the section 1.5m from support.
CALCULATION OF BENDING MOMENT AT MID SPAN (L/2):

**Due to Class 70R Wheeled**  Impact factor = 1.25

![Diagram showing calculations for Class 70R Wheeled loadings]

**Due to Class 70R Tracked**  Impact factor = 1.108

![Diagram showing calculations for Class 70R Tracked loadings]

Since bending moment due to Class A is having lesser value as compared to Class 70R loadings. Hence the superstructure is checked for Class 70R wheeled and Class 70R tracked loading.
**EFFECTIVE WIDTH OF SLAB**

According to IRC: 21-2000 cl. 305.16.2 (iii) if the effective width of slab for a load overlaps with effective width of slab for an adjacent load, the resultant effective width for two loads equals to sum of respective effective width for each load minus the width of overlap.

Firstly effective width as per IRC:21 is evaluated then actual available width is compared with that value and corresponding load intensity per metre width is evaluated for each axle.

Effective width = \( b_{\text{eff}} = \alpha \cdot a \cdot (1-a/l_0) + b_1 \)

Where

- \( L_0 \) = the effective span
- \( a \) = the distance of centre of gravity of the concentrated load from the nearer support
- \( \alpha \) = A constant depending upon the ratio \( b/l_0 \), where \( b \) is the width of the slab

\( b/l_0 = 0.966 \) Hence as per cl. 305.16.2 of IRC:21, \( \alpha = 2.439 \)

\( b_1 = \) the breadth of concentrated load over the deck slab after 45\(^\circ\) dispersion through wearing coat

\[
\begin{align*}
0.81 + 2\times 0.065 &= 0.922 \text{ m (for all axles of class 70R)} \\
0.84 + 2\times 0.065 &= 0.952 \text{ m (for class 70R Tracked)}
\end{align*}
\]

**CALCULATION OF EFFECTIVE WIDTH AND LOAD INTENSITY**

<table>
<thead>
<tr>
<th>LIVE LOAD</th>
<th>( a ) (m)</th>
<th>( b_1 ) (m)</th>
<th>( \alpha )</th>
<th>( b_{\text{eff}} ) (m)</th>
<th>( b_{\text{eff}}/2 ) (m)</th>
<th>Max. available width (m)</th>
<th>Load intensity (t/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FOR MAXIMUM BM AT MID SPAN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class 70R Wheeled</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>2.990</td>
<td>0.922</td>
<td>2.439</td>
<td>5.737</td>
<td>2.869</td>
<td>6.929</td>
<td>2.454</td>
</tr>
<tr>
<td>17</td>
<td>4.360</td>
<td>0.922</td>
<td>2.439</td>
<td>6.288</td>
<td>3.144</td>
<td>7.204</td>
<td>2.360</td>
</tr>
<tr>
<td>12</td>
<td>2.310</td>
<td>0.922</td>
<td>2.439</td>
<td>5.077</td>
<td>2.539</td>
<td>6.599</td>
<td>1.819</td>
</tr>
<tr>
<td>12</td>
<td>0.790</td>
<td>0.922</td>
<td>2.439</td>
<td>2.676</td>
<td>1.338</td>
<td>4.606</td>
<td>2.605</td>
</tr>
<tr>
<td>Class 70R Tracked</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>4.400</td>
<td>0.952</td>
<td>2.439</td>
<td>6.318</td>
<td>3.159</td>
<td>7.339</td>
<td>9.538</td>
</tr>
</tbody>
</table>
TRANSVERSE POSITION OF LIVE LOAD (CLASS 70 R WHEELED LOAD)

\[ a = 0.263 \]
\[ d = 1 \]
\[ L_0 \]

TRANSVERSE POSITION OF LIVE LOAD (CLASS 70 R TRACKED LOAD)

\[ a = 4.57 \]
\[ B_{\text{eff}}/2 = 2.13 \]

Effective width available

\[ B_{\text{eff}}/2 = 2.06 \]

Effective width available
CALCULATION OF LIVE LOAD MOMENTS (Per metre width)

Class 70R (Wheeled)

Impact factor = 1.25

<table>
<thead>
<tr>
<th></th>
<th>2.454</th>
<th>2.360</th>
<th>1.819</th>
<th>2.605</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.37</td>
<td>2.13</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td>Reaction (R_A)</td>
<td>= 3.52 t</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BM at mid span</td>
<td>= 12.04 tm/m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BM at mid span (with impact factor)</td>
<td>= 15.05 tm/m</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Class 70R (Tracked)

Impact factor = 1.108

<table>
<thead>
<tr>
<th></th>
<th>1.515</th>
<th>5.77</th>
<th>1.515</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.653</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reaction (R_A)</td>
<td>= 4.77 t</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BM at mid span</td>
<td>= 14.10 tm/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BM at mid span (with impact factor)</td>
<td>= 15.62 tm/m, Say 15.62 tm/m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Here 70R Tracked load governs the design

Total Design BM (Due to DL, SIDL & LL )

= 19.32 + 15.62 = 34.94 tm/m

Moment of Resistance (M_R)

lever arm factor, j = 0.902
moment of resistance coeff., Q = 113 t/m^2
effective cover = 50 mm

Eff. available depth = 600 mm

M_R = 40.59 tm/m > 34.94 tm/m
Hence O.K.

Design Reinforcement:

Ast. Reqd. = 31.65 cm^2/m

Hence, Provide 25 130 c/c, Ast. = 37.76 cm^2/m
**CALCULATION OF BM AT Dist.X = 1.50 m FROM THE SUPPORT**

BM AT DISTANCE X, \((BM)_X = \frac{w \cdot L \cdot x}{2 \cdot (1-x/L)}\)

<table>
<thead>
<tr>
<th>DUE TO DL, ((BM)_X)</th>
<th>1.584</th>
<th>5.475</th>
<th>8.67 tm/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>DUE TO SIDL, ((BM)_X)</td>
<td>0.412</td>
<td>5.475</td>
<td>2.25 tm/m</td>
</tr>
<tr>
<td>TOTAL BM</td>
<td>8.67 + 2.25 = 10.93 tm/m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DUE TO LIVE LOAD**

*Class 70R (Wheeled)*

Impact factor = 1.25

![Diagram of Class 70R (Wheeled) live load](image)

*Class 70R (Tracked)*

Impact factor = 1.108

![Diagram of Class 70R (Tracked) live load](image)

*Class A*

Impact factor = 1.304

![Diagram of Class A live load](image)

Since bending moment due to Class A is having lesser value as compared to Class 70R loadings.
Hence the superstructure is checked for Class 70R wheeled and Class 70R tracked loading.
CALCULATION OF EFFECTIVE WIDTH AND LOAD INTENSITY

<table>
<thead>
<tr>
<th>LIVE LOAD</th>
<th>a (m)</th>
<th>b1 (m)</th>
<th>( \alpha )</th>
<th>b(_{\text{eff}} ) (m)</th>
<th>b(_{\text{eff}}/2 ) (m)</th>
<th>Max. available width (m)</th>
<th>Load intensity (t/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 70R Wheeled</td>
<td>17</td>
<td>1.500</td>
<td>0.922</td>
<td>2.626</td>
<td>4.190</td>
<td>2.095</td>
<td>6.120</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>2.870</td>
<td>0.922</td>
<td>2.626</td>
<td>6.001</td>
<td>3.001</td>
<td>7.061</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>3.800</td>
<td>0.922</td>
<td>2.626</td>
<td>6.593</td>
<td>3.296</td>
<td>7.356</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>2.280</td>
<td>0.922</td>
<td>2.626</td>
<td>5.359</td>
<td>2.679</td>
<td>6.739</td>
</tr>
<tr>
<td>Class 70R Tracked</td>
<td>70</td>
<td>3.006</td>
<td>0.952</td>
<td>2.626</td>
<td>6.150</td>
<td>3.075</td>
<td>7.255</td>
</tr>
</tbody>
</table>

CALCULATION OF LIVE LOAD MOMENTS (Per metre width)

**Class 70R (Wheeled)**
Impact factor = 1.25

<table>
<thead>
<tr>
<th></th>
<th>2.778</th>
<th>2.408</th>
<th>1.631</th>
<th>1.52</th>
<th>1.781</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>1.50</td>
<td>1.37</td>
<td>2.13</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Class 70R (Tracked)**
Impact factor = 1.108

<table>
<thead>
<tr>
<th></th>
<th>0.779</th>
<th>2.111</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>0.721</td>
<td>4.57</td>
</tr>
</tbody>
</table>

Reaction \( R_A \) = 5.09 t
BM at the section = 7.64 tm/m
BM at the section (with impact factor) = 9.55 tm/m

Here 70R Tracked load governs the design

Total Design BM (Due to DL, SIDL & LL)

\[ = 10.93 + 10.00 = 20.93 \text{ tm/m} \]

**Design Reinforcement:**

Ast. Reqd. = 18.64 cm²/m

Hence, Provide \( \phi \) 20 @ 130 c/c, Ast. = 24.15 cm²/m
BRIDGE AT CH:54+600
DESIGN OF SUBSTRUCTURE (ABUTMENT)
**DESIGN DATA**

Formation Level = 241.909 m  
Ground Level = 236.941 m  
Lowest Water Level = 232.000 m  
Highest Flood Level = 240.009 m  
Founding Level = 232.000 m  
Thickness of bearing & pedestal = 0.000 m  
Width of abutment = 12.000 m  
Bouyancy factor = 1.0  
Safe Bearing Capacity = 37.600 t/sqm  
Dry density of earth = 1.800 t/cum  
Submerged density of earth = 1.0 t/cum  
Saturated density of earth = 2.000 t/cum  
Coefficient of base friction = 0.55  
Span (c/c of exp. joint) = 10.800 m  
Overall Width of deck slab = 12.000 m  
Width of carriageway = 11.000 m  
Width of crash barrier = 0.500 m  
Depth of Superstructure at mid = 0.925 m  
Depth of Superstructure at end = 0.775 m  
Thickness of wearing coat = 0.056 m  
Grade of Concrete - M 25  
Grade of Reinforcement - Fe 415 (HYSD)  
Live Load - One Lane of 70R Wheeled + Class A  
- 3 lanes of Class A  
Permissible Compressive stress in Concrete = 850 t/m²  
Permissible Tensile stress in Steel = 20400 t/m²  
Modular ratio, m = 10  

factor, k = 0.294  
Lever arm factor, j = 0.902  
Moment of Resistance = 113 t/m²  
Thickness of returnwall = 0.5 m

**COEFFICIENT OF ACTIVE EARTH PRESSURE**

As per Coulomb's Theory, Coefficient of Active Earth Pressure is:

\[
K_a = \sin \phi \frac{\sin \alpha \cdot \sin(\alpha - \delta)}{\sin^2(\phi + \delta)} \cdot \frac{\sin(\alpha - \delta) \cdot \sin(\phi + \delta)}{1 \pm \sqrt{\frac{\sin^2(\alpha + \phi) \cdot \sin(\phi - \delta)}{\sin^2(\alpha - \delta) \cdot \sin(\phi + \delta)}}}
\]

Where:
- \(\phi\) = Angle of internal friction of earth
- \(\alpha\) = Angle of inclination of back of wall
- \(\delta\) = Angle of internal friction between wall & earth
- \(\iota\) = Angle of inclination of backfill

Here:
- \(\phi = 30^\circ = 0.524\) Radian
- \(\alpha = 90^\circ = 1.571\) Radian
- \(\delta = 20^\circ = 0.349\) Radian
- \(\iota = 0^\circ = 0\) Radian

\[
K_a = 0.2973
\]

Therefore, Horizontal coefficient of Active pressure = \(K_a \cos \phi = K_{ha} = 0.2794\)
**HEIGHT OF ABUTMENT**

Total height of abutment = Formation Level - Founding Level = 9.909 m
For DESIGN purpose, the height of abutment is considered as, say, = 9.910 m

**CALCULATION OF ACTIVE EARTH PRESSURE**

![Diagram of active earth pressure calculation]

**DRY. condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>79.73</td>
<td>4.955</td>
<td>395.02</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>9.909</td>
<td>4.98</td>
<td>296.27</td>
<td>4.162</td>
<td>1233.00</td>
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<tr>
<td>3</td>
<td>SubmgEarth</td>
<td>1.0</td>
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<td>4.98</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
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<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>8.009</td>
<td>2.24</td>
<td>107.52</td>
<td>2.670</td>
<td>287.06</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>376.00</td>
<td></td>
<td>1628.02</td>
</tr>
</tbody>
</table>

**b) Span Dislodge Condition**

Net force = 376.00 - 79.73 = 296.27 t
Net moment = 1628.02 - 395.02 = 1233.00 tm

**H.F.L. condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>79.73</td>
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<td>395.02</td>
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<td>Dry Earth</td>
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<td>SubmgEarth</td>
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<td>107.52</td>
<td>2.670</td>
<td>287.06</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.5</td>
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<td></td>
<td>107.52</td>
<td>2.670</td>
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<tr>
<td>TOTAL</td>
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<td></td>
<td></td>
<td></td>
<td>301.39</td>
<td></td>
<td>1197.26</td>
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</table>

**b) Span Dislodge Condition**

Net force = 301.39 - 79.73 = 221.66 t
Net moment = 1197.26 - 395.02 = 802.24 tm
## Forces & moments due to Abutment (Concrete) components

### DRY Case:

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment about toe</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Toe Slab</td>
<td>1.0</td>
<td>2.000</td>
<td>12.00</td>
<td>0.500</td>
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<td>28.80</td>
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<td>28.80</td>
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<tr>
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<td>2.000</td>
<td>12.00</td>
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<td>20.16</td>
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<td>50.40</td>
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<td>2.850</td>
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<tr>
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<tr>
<td>8a</td>
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### H.F.L. Case:

<table>
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<th>Element No.</th>
<th>Component</th>
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<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Toe Slab</td>
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<td>2.000</td>
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<td>20.58</td>
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<td>80.07</td>
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<td>28.60</td>
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<td>66.73</td>
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<td>1.200</td>
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<td>0.300</td>
<td>2.40</td>
<td>6.05</td>
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<tr>
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<td>0.300</td>
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<td>2.40</td>
<td>15.96</td>
<td>4.950</td>
<td>79.00</td>
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<td></td>
<td></td>
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<td>290.88</td>
<td>934.20</td>
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</table>
## Forces & moments due to Earth and LL surcharge

### DRY Case : Self weight of Earth

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<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>DRY EARTH</td>
<td>0.5</td>
<td>3.500</td>
<td>11.00</td>
<td>0.700</td>
<td>1.8</td>
<td>24.26</td>
<td>5.533</td>
<td>134.21</td>
</tr>
<tr>
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<td>1.0</td>
<td>3.500</td>
<td>11.00</td>
<td>8.709</td>
<td>1.800</td>
<td>603.53</td>
<td>4.950</td>
<td>2987.49</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>627.79</td>
<td>3121.70</td>
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### H.F.L Case :

<table>
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<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>SATURATED SOIL</td>
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<td>3.500</td>
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<td>0.7</td>
<td>1.000</td>
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<td>74.56</td>
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<tr>
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<td>3.500</td>
<td>11.00</td>
<td>6.809</td>
<td>1.000</td>
<td>262.15</td>
<td>4.950</td>
<td>1297.63</td>
</tr>
<tr>
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<td>1.0</td>
<td>3.500</td>
<td>11.00</td>
<td>1.9</td>
<td>1.800</td>
<td>131.67</td>
<td>4.950</td>
<td>651.77</td>
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### L.L.SURCHARGE

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
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<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m$^3$)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe</th>
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</thead>
<tbody>
<tr>
<td>12</td>
<td>DRY EARTH</td>
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<td>3.500</td>
<td>11.00</td>
<td>1.20</td>
<td>1.800</td>
<td>0.00</td>
<td>4.950</td>
<td>0.00</td>
</tr>
</tbody>
</table>

### SUMMARY OF FORCES AND MOMENTS:

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>Case. L.W.L.</th>
<th>Case. H.F.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical load from superstructure including LL</td>
<td>232.07</td>
<td>232.07</td>
</tr>
<tr>
<td>Vertical load from substructure (b)</td>
<td>1133.44</td>
<td>698.17</td>
</tr>
<tr>
<td>Total Vertical Load</td>
<td>1365.51</td>
<td>698.17</td>
</tr>
<tr>
<td>Total Horizontal Force</td>
<td>383.51</td>
<td>221.66</td>
</tr>
<tr>
<td>Moment @ toe due to (a)</td>
<td>661.40</td>
<td>661.40</td>
</tr>
<tr>
<td>Moment @ toe due to (b)</td>
<td>4727.95</td>
<td>2958.15</td>
</tr>
<tr>
<td>Total Moment @ toe (M)</td>
<td>5389.35</td>
<td>2958.15</td>
</tr>
<tr>
<td>Dist. of C.G. of V from toe Z = M/V</td>
<td>3.947</td>
<td>3.947</td>
</tr>
<tr>
<td>eccentricity (e = Z -b/2)</td>
<td>0.597</td>
<td>0.597</td>
</tr>
<tr>
<td>Relieving Moment @ c/l base (M1)</td>
<td>814.89</td>
<td>619.28</td>
</tr>
<tr>
<td>overturning moment due to</td>
<td>69.73</td>
<td>69.73</td>
</tr>
<tr>
<td>Horz. braking force</td>
<td>69.73</td>
<td>69.73</td>
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<tr>
<td>Earth Pressure</td>
<td>1628.02</td>
<td>802.24</td>
</tr>
<tr>
<td>Total overturning Moment (M2)</td>
<td>1697.75</td>
<td>802.24</td>
</tr>
<tr>
<td>Net moment (M2-M1) = M</td>
<td>882.86</td>
<td>182.96</td>
</tr>
</tbody>
</table>

| Factor of Safety | Against overturning (M / M2) | 3.17 | 3.69 |
| Against sliding (μ x V / H) | 1.958 | 1.732 |

---

I.R.C 78-2000:cl 706.3.4

Safe against overturning

Safe against sliding
Area of base (A) = 6.700 x 12.00 = 74.40 m²

\[ Z_L = 89.78 \text{ m}^3 \]
\[ Z_T = 160.80 \text{ m}^3 \]

**CHECK FOR BASE PRESSURE:**

<table>
<thead>
<tr>
<th>Base Pressure</th>
<th>LWL CASE</th>
<th>HFL CASE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service Cond.</td>
<td>Span dislodged</td>
</tr>
<tr>
<td>( P/A )</td>
<td>18.35</td>
<td>15.23</td>
</tr>
<tr>
<td>( M_L/Z_L )</td>
<td>9.83</td>
<td>3.36</td>
</tr>
<tr>
<td>( M_T/Z_T )</td>
<td>0.86</td>
<td>0.00</td>
</tr>
<tr>
<td>(A) ( (P/A + M_L/Z_L + M_T/Z_T) )</td>
<td>29.04</td>
<td>18.60</td>
</tr>
<tr>
<td>(B) ( (P/A + M_L/Z_L - M_T/Z_T) )</td>
<td>27.33</td>
<td>18.60</td>
</tr>
<tr>
<td>(C) ( (P/A - M_L/Z_L + M_T/Z_T) )</td>
<td>9.38</td>
<td>11.87</td>
</tr>
<tr>
<td>(D) ( (P/A - M_L/Z_L - M_T/Z_T) )</td>
<td>7.662</td>
<td>11.87</td>
</tr>
</tbody>
</table>

Max. Base Pressure = 29.04 t/m² < 37.60 Hence O.K.
Min. Base Pressure = 3.14 t/m² > 0 Hence O.K.

**DESIGN OF TOE SLAB**
BENDING MOMENT AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
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<tbody>
<tr>
<td>Downward Loads</td>
<td>1</td>
<td>1.0</td>
<td>2.400</td>
<td>1.000</td>
<td>2.400</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>1.880</td>
<td>0.667</td>
<td>1.120</td>
</tr>
<tr>
<td>Upward Base pressure</td>
<td>RecL</td>
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<td>-46.348</td>
<td>1.000</td>
<td>-46.348</td>
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<td></td>
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<td>0.5</td>
<td>-5.871</td>
<td>1.333</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>-48.139</td>
<td></td>
<td>-50.656</td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 50.656 tm/m

Effective depth required 0.670 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 2469 mm²

Minimum steel required 0.15% 1673 mm² I.R.C 78-2000 Clause:707.2.7

Distribution steel 617 mm²/m

mainsteel 2469 mm²

Hence provide, 12 φ , @ 150 C/C 753.982

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC:78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinforcement as per above clause 669 mm²/m

provide 12 φ , @ 150 C/C 753.982

Check for Shear

SHEAR FORCE AT “d” FROM FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1.0</td>
<td>1.062</td>
<td>0.443</td>
<td>0.470</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>0.743</td>
<td>0.295</td>
<td>0.219</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>RecL</td>
<td>1.0</td>
<td>-24.624</td>
<td>0.443</td>
<td>-10.896</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-0.540</td>
<td>0.590</td>
<td>-0.319</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>-23.359</td>
<td></td>
<td>-10.526</td>
</tr>
</tbody>
</table>

Effective depth (d’) at distance d 0.725 m

Shear force at critical section 23.4 t

Bending Moment at critical section 10.53 tm

\[ \tan \beta = 0.36 \]

Net shear force \( S \cdot M \cdot \tan \beta / d' \) 18.10 t

Hence, Shear stress 24.97 t/m²

% of reinforcement 0.39

Permissible shear stress 27.92 t/m² Hence O.K.
DESIGN OF HEEL SLAB

BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads.</td>
<td>3</td>
<td>1</td>
<td>4.200</td>
<td>1.750</td>
<td>7.350</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>2.940</td>
<td>1.167</td>
<td>3.430</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.5</td>
<td>4.410</td>
<td>2.333</td>
<td>10.290</td>
</tr>
<tr>
<td></td>
<td>8a</td>
<td>1</td>
<td>54.867</td>
<td>1.750</td>
<td>96.017</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>4.410</td>
<td>2.333</td>
<td>5.145</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>6.831</td>
<td>1.750</td>
<td>11.955</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>Rect.</td>
<td>1</td>
<td>-26.819</td>
<td>1.750</td>
<td>-46.932</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-17.979</td>
<td>1.167</td>
<td>-20.976</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>32.860</td>
<td></td>
<td>66.278</td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 66.278 tm

Effective depth required 0.767 m
Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3232.56 mm²
Minimum steel 0.15% 1672.50 mm² I.R.C 78-2000 Clause:707.2.7

Distribution steel 808.14 mm²/m
mainsteel 3232.56 mm²

Hence provide, 25 φ @ 150 C/C 753.9822
0 φ @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.
Minimum steel reinforcement as per above clause provide 16 φ , @ 150 C/C 1340.41
**Check for Shear**  
( Critical section at face of stem )

Shear force at face of stem  
\[ 32.86 \text{ t} \]

\[ \tan \beta = 0.200 \]

Bending moment at face of stem  
\[ 66.278 \text{ tm} \]

Net shear force \( S-M\tan \beta /d \)  
\[ 20.97 \text{ t} \]

Hence, Shear stress  
\[ 18.81 \text{ t/m}^2 \]

% of reinforcement  
\[ 0.29 \]

Permissible shear stress  
\[ 24.75 \text{ t/m}^2 \]

Hence O.K.

**DESIGN OF STEM WALL**

![Diagram of stem wall with dimensions](image)

**SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT**

R.L. OF SECTION = 233.200 m

**DRY Condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Area factor</th>
<th>Height of E.P. diagram</th>
<th>Earth Pressure</th>
<th>Force</th>
<th>L.A.</th>
<th>Moment tm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>8.359</td>
<td>0.603</td>
<td>60.5</td>
<td>4.180</td>
<td>253.00</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>8.359</td>
<td>4.204</td>
<td>210.8</td>
<td>3.511</td>
<td>740.18</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td>278.88</td>
<td>7.728</td>
<td>1051.256</td>
</tr>
</tbody>
</table>

Total Vertical Load  
\[ = \text{Stem} + \text{dirt wall} + \text{cap} + \text{Load from superstructure} \]
\[ = 256.712 + 5.45 + 6.05 + 232.07 \]
\[ = 500.282 \text{ t} \]

Longitudinal Moment  
\[ = 1051.256 \text{ t-m} \]

Transverse Moment  
\[ = 137.896 \text{ t-m} \]
b) Span Dislodged Condition

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

\[
= 256.712 + 5.45 + 6.05 + 0.00 \\
= 268.212 \text{ t}
\]

Longitudinal Moment = 993.176 t-m

Transverse Moment = 0.000 t-m

H.F.L. condition

a) Service Condition


<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (t-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.603</td>
<td>60.53</td>
<td>4.180</td>
<td>253.00</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>1.494</td>
<td>0.83</td>
<td>7.48</td>
<td>7.307</td>
<td>54.68</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>1.0</td>
<td>6.809</td>
<td>0.83</td>
<td>68.21</td>
<td>3.405</td>
<td>232.22</td>
</tr>
<tr>
<td>4</td>
<td>SubmgEarth</td>
<td>0.5</td>
<td>6.809</td>
<td>1.90</td>
<td>77.72</td>
<td>2.270</td>
<td>176.39</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>213.94 716.289</td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

\[
= 121.151 + 5.45 + 6.05 + 232.07 \\
= 364.721 \text{ t}
\]

Longitudinal Moment = 774.368 t-m

Transverse Moment = 137.896 t-m

b) Span Dislodged Condition

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

\[
= 121.151 + 5.45 + 6.05 + 0.00 \\
= 132.651 \text{ t}
\]

Longitudinal Moment = 716.289 t-m

Transverse Moment = 0.000 t-m

Cross Sectional area = 0.95 m² 2375.000

For horizontal reinforcement area of steel required for the stem at the section/metre = 185.4639 mm²

Providing 12 @ 47.62 c/c say, 150 C/C as horizontal reinforcement
As per clause 214.2 of IRC:6, horizontal braking force $F_h$, for each span is:

**For Class A Single lane**
$$F_h = \left( 0.2 \times 26.6 \right) = 5.320 \text{ t}$$

**For class 70R Wheeled**
$$F_h = \left( 0.2 \times 55.2 \right) = 11.032 \text{ t}$$

**For class A 3 lane**
$$F_h = 0.2 \times 26.6 + 0.05 \times 26.6 = 6.65 \text{ t}$$

**For class 70R W + class A 1 lane**
$$F_h = 0.2 \times 55.2 + 0.05 \times 26.6 = 12.362 \text{ t}$$

\[
\mu R_g = 7.5155 \text{ t}
\]
\[
\frac{F_h}{2} = 6.181
\]

### Summary of Longitudinal Forces:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70RT+class A 1 lane</td>
<td>7.52</td>
</tr>
</tbody>
</table>

**Dead Load**

- dead load of slab $= 24.48 \text{ t/m}$
- Total reaction $= 132.19 \text{ t}$
- wearing coat $= 1.36 \text{ t/m} = 7.3181 \text{ t}$
- crashbarrier $= 1.00 \text{ t/m} = 10.8 \text{ t}$

**Transverse eccentricity**

Class 70RW+ Class A

<table>
<thead>
<tr>
<th>55.16</th>
<th>3.095</th>
<th>6.84</th>
<th>26.6</th>
</tr>
</thead>
</table>

\[e = 1.6866\]

Class A 3 lane

<table>
<thead>
<tr>
<th>5.3</th>
<th>0.7</th>
</tr>
</thead>
</table>

\[e = 0.7\]
### Summary of Dead load & Live loads from Superstructure. (STAAD Pro)

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load Reaction:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>132.19</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td>SIDL</td>
<td>18.12</td>
<td>t</td>
<td></td>
</tr>
<tr>
<td><strong>L.L Max Reaction Case</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70RT + Class A</td>
<td>81.76</td>
<td>t</td>
<td>137.90</td>
</tr>
<tr>
<td>Class A 3 Lane</td>
<td>79.8</td>
<td>t</td>
<td>55.86</td>
</tr>
<tr>
<td></td>
<td><strong>232.07</strong></td>
<td></td>
<td><strong>137.90</strong></td>
</tr>
</tbody>
</table>
Summary of Loads at Abutment Shaft bottom:

1 DRY condition  

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>500.28</td>
<td>1051.26</td>
<td>137.90</td>
</tr>
</tbody>
</table>

Check for Cracked/Uncracked Section

- Length of section = 12000 mm
- Width of section = 1200 mm
- Gross Area of section \( A_g \) = 14400000 mm²
- Gross M.O.I of section \( I_{gx} \) = \( 1.728 \times 10^{12} \) mm⁴
- Gross M.O.I of section \( I_{gy} \) = \( 1.728 \times 10^{14} \) mm⁴

Transformed sectional properties of section:

- Adopting Modular ratio \( m = 10 \)
- Cover 72.5 68
- Dia of Bars = 25 16
- No of bars in tension face (longer) = 120
- No of bars in compression face = 60
- No of bars in shorter direction = 8
- Total bars in section = 196

Reinforcement provided = 74185.569 mm²
Steel Area $A_s = 74186 \text{ mm}^2$

% of Steel $= 0.5152 \%$

$A_{xx} = 58905 \text{ mm}^2$

$A_{yy} = 1608.5 \text{ mm}^2$

Area of concrete $A_c = A_y - A_s = 14325814 \text{ mm}^2$

C.G of Steel placed on longer face $= 527.5 \text{ mm}$

C.G of Steel placed on shorter face $= 5932 \text{ mm}$

Transformed Area of Section $A_{tfm} = 15067670 \text{ mm}^2$

Transformed M.I $I_{txx} = I_{gxx} + 2 \left[ m - \frac{I}{A_c} \right] A_c a x^2$

$= 2.02303E+12 \text{ mm}^4$

$Z_{xx} = \frac{M.I_{txx}}{d/2} = 3.372E+09 \text{ mm}^3$

Transformed M.I $I_{tyy} = I_{gyy} + 2 \left[ m - \frac{I}{A_c} \right] A_c a y^2$

$= 1.73819E+14 \text{ mm}^4$

$Z_{yy} = \frac{M.I_{tyy}}{d/2} = 2.897E+10 \text{ mm}^3$

**Permissible stresses**

Minimum Gross Moment of inertia $I_{min} = 1.728E+12 \text{ mm}^4$

Area of section $r = 14400000 \text{ mm}^2$

**Effective length of Abutment shaft** (IRC:21-2000 cl: 306.2.1)

Abutment shaft height $L = 9.426 \text{ m}$

Effective length $L_{eff} = 11.311 \text{ m}$

Slenderness ratio $= 32.653 \leq 50$

Type of member $= 1$

**Stress reduction coefficient** (IRC:21-2000 cl: 306.4.2,3)

$\beta = 1$

**Permissible stresses**

$\sigma_{bc} = 8.3333 \text{ N/mm}^2$

$\sigma_{co} = 6.25 \text{ N/mm}^2$

Tensile stress $= 0.61 \text{ N/mm}^2$

**Permissible stresses**

$\sigma_{st} = 200 \text{ N/mm}^2$
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>DRY Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P</td>
<td>500.28 t</td>
</tr>
<tr>
<td>2</td>
<td>M_t</td>
<td>1051.26 t-m</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>137.90 t-m</td>
</tr>
</tbody>
</table>

**Loads and Moments With L.L**

- **Actual(calculated) Stresses**
  - $\sigma_{co,cal} = \frac{P}{A_{flm}}$
  - $\sigma_{cbc,cal} = \frac{M_t}{Z_{xx}}$
  - $\sigma_{cbc,cal} = \frac{M_T}{Z_{yy}}$
  - $\sigma_{cbc,cal} = 5 + 6$

**Permissible Stresses**

- $\sigma_{cbc}$
- $\sigma_{co}$

**Check for Minimum steel area mm$^2$**

- Conc.Area Required for directstress
  - $(1)/(9)$
  - 800450.56
- 0.8% of area required
  - 6403.6045
- 0.3% of $A_g$
  - 43200
- Governing steel mm$^2$
  - 43200

**Provided Steel area mm$^2$**

- 74185.569

**Check for safety of section**

- $\frac{\sigma_{co,cal} + \sigma_{cbc,cal}}{\sigma_{co}} < 1$

- 0.4329792

**Check for Cracked /Uncracked section**

- $\sigma_{co,cal} - \sigma_{cbc,cal}$
  - -2.833439

**Permissible Basic tensile stress in concrete**

- -0.61

**Section to be designed as**

- **Cracked**
Width of Solid return wall \((a)\) = 3.50
Width of Cantilever return wall = 4.00
Avg Height of Solid return wall \((b)\) = 9.059
Height of Cantilever return at Tip = 0.75
Height of Cantilever return at Root = 2.666667
Thickness of Solid Return at farther end = 0.5
Thickness of Solid Return at Root = 0.5
Thickness of Solid Return at bottom = 0.5
Thickness of Solid Return at top = 0.5
Thickness of Cantilever return = 0.5
Unit wt of Soil = 1.8 \(t/m^3\)
Grade of concrete = M 30
\(\sigma_{cbc} = 1020 \ t/m^2\)
m = 10
\(\sigma_{st} = 20400 \ t/m^2\)
k = 0.333333
j = 0.888889
R = 151.1111 \(t/m^2\)

Case (1) For uniformly distributed load over entire plate

<table>
<thead>
<tr>
<th>(a/b)</th>
<th>For (a/b = 0.375)</th>
<th>(\beta_1)</th>
<th>(\beta_2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3863561</td>
<td>0.375</td>
<td>0.353</td>
<td>0.398</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>0.631</td>
<td>0.632</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(a/b)</th>
<th>(\beta_1)</th>
<th>(\beta_2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3863561</td>
<td>0.378256</td>
<td>0.419259</td>
</tr>
</tbody>
</table>
Live Load Surcharge:

\[ q = 0.2794 \times 1.8 \times 1.2 = 0.603504 \, \text{t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.378256 \times 0.603504 \times 82.07}{0.25} = 74.9353 \, \text{t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \, \text{mm}^3 \]

\[ = 0.041667 \, \text{m}^3 \]

Hence Moment /m width along Y direction

\[ M_Y /\text{m width} = 74.935304 \times 0.041667 = 3.122304 \, \text{t-m/m} \]

\[ \sigma_{a_{\text{max}}} = \frac{0.4192586 \times 0.603504 \times 82.07}{0.25} = 83.05823 \, \text{t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \, \text{mm}^3 \]

\[ = 0.041667 \, \text{m}^3 \]

Hence Moment /m width along X direction

\[ M_X /\text{m width} = 83.058232 \times 0.041667 = 3.46076 \, \text{t-m/m} \]

Case (2) For Triangular loading due to earth pressure

\[ a/b = 0.3863561 \quad \text{For a/b} = 0.375 \quad \beta_1 = 0.212 \quad \beta_2 = 0.148 \]

\[ \text{For a/b} = 0.5 \quad \beta_1 = 0.328 \quad \beta_2 = 0.200 \]

\[ a/b = 0.3863561 \quad \beta_1 = 0.222538 \quad \beta_2 = 0.152724 \]

Earth pressure:

\[ q = 0.2794 \times 1.8 \times 9.059 = 4.555952 \, \text{t/m}^2 \]

\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]

\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]

\[ \sigma_{b_{\text{max}}} = \frac{0.2225385 \times 4.555952 \times 82.07}{0.25} = 332.8164 \, \text{t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \, \text{mm}^3 \]

\[ = 0.041667 \, \text{m}^3 \]
Hence Moment /m width along Y direction

\[ M_Y /m \text{ width} = 332.81644 \times 0.041667 = 13.86735 \text{ t-m/m} \]
\[ \sigma_{\text{max}} = 0.1527241 \times 4.555952 \times 82.07 = 228.4059 \text{ t/m}^2 \]

For 1000 mm of width
\[ Z = 1000 \times 250000 \]
\[ = 4166667 \text{ mm}^3 \]
\[ = 0.04167 \text{ m}^3 \]

Hence Moment /m width along X direction

\[ M_X /m \text{ width} = 228.40593 \times 0.041667 = 9.516914 \text{ t-m/m} \]

Total Moment in Solid Return /m height
\[ = 12.97767 \text{ t-m/m} \]
Along X-direction

Total Moment in Solid Return /m width
\[ = 16.98966 \text{ t-m/m} \]
Along Y-direction

Moment due to Cantilever Return:

Moment due to earth pressure at face A - A'

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 \]
\[ + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 \]
\[ + 0.5 \times 0.2794 \times 1.8 \times 0.444444 \times X^2 \times dx \times \left[ 4.00 - X \right] \]
\[ + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left[ 4.00 - X \right] \]
\[ = 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.66667 \]
\[ = 14.11063 \text{ t-m} \]

Design of cantilever Return:
Assuming 50 mm cover and 12 mm dia bars.
Effective depth available
\[ = 500 - 50 - 20 - 6 = 424 \text{ mm} \]
\[ M = R \times b \times d^2 \]
\[ = 151.1111 \times 2.666667 \times 0.179776 = 72.44307 \text{ t-m} \]
\[ A_{st} = 14.11063 \times 10^6 \]
\[ = 1835.283 \text{ mm}^2 \]
\[ A_{se}/m = \frac{20400 \times 0.888889 \times 0.424}{688.231} = 688.231 \text{ mm}^2/m \]

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm² on earth face.
Provide 10 mm dia @ 150 mm c/c providing 523.5988 mm² on other face.

Along Horizontal direction.
Design of Solid Return:

Moment due to Cantilever Return:

\[
M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 5.50 \\
+ 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 5.50 \\
+ 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times X^2 \times dx \times \left[ 7.50 - X \right] \\
+ 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left[ 7.50 - X \right]
\]

\[
= 9.957816 + 3.1118175 + 0.11176 \times 96 + 0.653796 \times 38.66667 \\
= 49.07871 \text{ t-m}
\]

Moment in Solid Return /m height

\[
= 12.97767 + \frac{49.07871}{9.059} = 18.39535 \text{ t-m/m}
\]

Moment in Solid Return /m width

\[
= 16.98966 \text{ t-m/m}
\]

Design of face B-B

Moment in Solid Return /m height

\[
= 18.39535 \text{ t-m/m}
\]

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 20 - 13 = 418 mm

\[
M = R \times b \times d^2 \\
= 151.1111 \times 1 \times 0.174306 = 26.33961 \text{ t-m}
\]

\[
A_{st} = \frac{18.39535 \times 10^6}{20400 \times 0.888889 \times 0.4175} \\
= 2429.819 \text{ mm}^2
\]

\[
A_{as/m} = 2429.819 \text{ mm}^2/m
\]

Provide 20 mm dia @ 125 mm c/c providing 2513.274 mm$^2$ on earth face.
Provide 12 mm dia @ 125 mm c/c providing 904.7787 mm$^2$ on other face.

Along Horizontal direction.

Design of face A'-B'

Moment in Solid Return /m width

\[
= 16.98966 \text{ t-m/m}
\]

Assuming 50 mm cover and 25 mm dia bars.
Effective depth available = 500 - 50 - 0 - 13 = 438 mm

\[
M = R \times b \times d^2 \\
= 151.1111 \times 1 \times 0.191406 = 28.92361 \text{ t-m}
\]

\[
A_{st} = \frac{16.98966 \times 10^6}{20400 \times 0.888889 \times 0.4375} \\
= 2141.553 \text{ mm}^2
\]

\[
A_{as/m} = 2141.553 \text{ mm}^2/m
\]

Provide 20 mm dia @ 125 mm c/c providing 2513.274 mm$^2$ on earth face.
Provide 12 mm dia @ 125 mm c/c providing 904.7787 mm$^2$ on other face.

Along Vertical direction.
1. **DESIGN FOR FORCES IN LONGITUDINAL DIRECTION**

Active earth pressure  $K_a = 0.279384$

FOR NORMAL CASE, $F_a = 1.0$

unit wt of soil  $\gamma = 1.8$

width of return wall  $= 0.5$

Avg. cover to reinforcement. (FOR 2-3 LAYERS)  $= 0.150\ m$

H (From formation level)  $= 8.709\ m$

### DESIGN FOR BENDING MOMENT

<table>
<thead>
<tr>
<th>S.NO.</th>
<th>UNITS</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (FROM TOP)</td>
<td>m</td>
<td>8.709</td>
</tr>
<tr>
<td>WIDTH OF RETURN AT THIS LEVEL</td>
<td>m</td>
<td>3.500</td>
</tr>
<tr>
<td>FORCE DUE TO EARTH PRESSURE</td>
<td>t</td>
<td>107.527</td>
</tr>
<tr>
<td>MOMENT DUE TO EARTH PRESSURE</td>
<td>tm</td>
<td>393.310</td>
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<tr>
<td>FORCE DUE TO L.L. SURCHARGE</td>
<td>t</td>
<td>29.632</td>
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<tr>
<td>MOMENT DUE TO L.L. SURCHARGE</td>
<td>tm</td>
<td>129.032</td>
</tr>
<tr>
<td>TOTAL MOMENT</td>
<td>tm</td>
<td>522.342</td>
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<tr>
<td>DESIGN MOMENT</td>
<td>tm</td>
<td>522.342</td>
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<tr>
<td>REQUIRED EFFECTIVE DEPTH</td>
<td>m</td>
<td>2.940</td>
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<tr>
<td>EFF. DEPTH AVAILABLE</td>
<td>m</td>
<td>3.350</td>
</tr>
<tr>
<td>AREA OF STEEL REQUIRED</td>
<td>cm$^2$</td>
<td>85.976</td>
</tr>
<tr>
<td>DIAMETER OF BAR PROVIDED</td>
<td>mm</td>
<td>32</td>
</tr>
<tr>
<td>TOTAL NO. OF BARS</td>
<td>no.</td>
<td>12</td>
</tr>
<tr>
<td>AREA OF STEEL PROVIDED</td>
<td>cm$^2$</td>
<td>96.51</td>
</tr>
</tbody>
</table>

**CHECK FOR SHEAR STRESS**

| SHEAR FORCE | t | 142.159 |
| SHEAR STRESS | t/m$^2$ | 84.871 |
| Ast/bd x 100 | % | 0.513 |
| PERMISSIBLE SHEAR STRESS | MPa | 0.31 |
| PERMISSIBLE SHEAR STRESS | t/m$^2$ | 31.95 |
| Asv/sv | cm$^2$/m | 7.744 |
Design of Abutment Cap:

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl: 710.8.2 of IRC:78-2000 is 200 mm.

However the thickness of abutment cap is = 300 mm
Assuming a cap thickness of = 300 mm

Volume of Abutment cap = 0.3 x 0.70 x 12
= 2.52 m³

Quantity of steel = 1 % of volume
= 1 x 2.52
= 0.0252 m³

Quantity of steel to be provided at top = 0.0126 m³
Quantity of steel to be provided at bottom = 0.0126 m³

Top & bottom face:

Quantity of steel to be provided in Longitudinal dir = 0.0063 m³
Assuming a clear cover of = 50 mm
Length of bar 12.00 - 0.100 = 11.9 m

Area of steel required in Longitudinal direction
= 0.0063
11.9
= 529.412 mm²

Provide 8 nos of bars 12 mm dia at top & bottom face.
= 904.779 mm²

Transverse steel:

Quantity of steel to be provided in Longitudinal dir = 0.0063 m³
Assuming a clear cover of = 50 mm
Assuming a dia of bar = 12 mm
Length of bar 0.70 - 0.100 = 0.6 m

Volume of each stirrup = 6.8E-05 m³

no of stirrups required for m/length = 8 nos
Required Spacing = 1000
= 8
= 125 mm

Provide 12 mm dia bar 125 mm c/c stirrups through in length of abutment cap.
904.779 mm²
Design of Dirt wall:
Dirt wall designed as a vertical cantilever.

Intensity for rectangular portion = \( 0.2794 \times 2.00 \times 1.2 = 0.67056 \) t/m

\[ F_1 \]

\[ F_2 = 0.548183 \times 12.00 \times 0.98 = 3.226604 \text{ t} \]

\[ M_1 = 0.2794 \times 2.00 \times 0.98 = 0.548183 \text{ t-m} \]

\[ M_2 = 3.226604 \times 0.41202 = 1.329425 \text{ t-m} \]

\[ M_1 + M_2 = 5.20135 \text{ t-m} \]

Total moment at base of dirt wall /m length = \( 0.433446 \text{ t-m/m} \)

Thickness of dirt wall = 0.3 m

Assuming a clear cover on either face = 50 mm

**Vertical steel on earth face:**

dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 6 = 244 mm

Effective depth req = \( \frac{0.433446 \times 1.51 \times 1000}{200 \times 0.889 \times 244} = 53.57707 \text{ mm} \)

Ast req = \( \frac{0.433446 \times 1.51 \times 1000}{200 \times 0.889 \times 244} = 99.91099 \text{ mm} \)

Minimum steel = \( 0.12 \times 300 \times 1000 = 360 \text{ mm}^2/m \)

Provide 12 mm dia bar 200 mm c/c as vertical steel at earth face.

**Distribution steel on earth face:**

dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 12 = 238 mm

\[ 0.3M = 0.3 \times 0.433446 = 0.130034 \text{ t-m/m} \]

\[ \text{Ast req} = \frac{0.130034 \times 200 \times 0.889 \times 238}{200 \times 0.889 \times 238} = 30.72893 \text{ mm}^2/m \]

Minimum steel as per IRC:21-200 cl:305.10 = 250 mm²/m

Governing steel at earth face = 250 mm²/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

**Vertical steel on earth face**

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm²/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

**Distribution steel:**

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm²/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.
INPUT FILE: 70r.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
9. 1 0.00 0 0;2 0.4 0 0;3 10.4 0 0;4 10.8 0 0
10. MEMBER INCIDENCES
11. 1 1 2 3
12. MEMBER PROPERTY CANADIAN
13. 1 TO 3 PRI YD 1.0 ZD 1.0
14. CONSTANT
15. E CONCRETE ALL
16. DENSITY CONCRETE ALL
17. POISSON CONCRETE ALL
18. SUPPORT
19. 2 3 PINNED
20. DEFINE MOVING LOAD
21. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
22. LOAD GENERATION 175
23. TYPE 1 -13.4 0. 0. XINC .2
24. PERFORM ANALYSIS
25. LOAD LIST 55
26. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METER STRUCTURE TYPE = PLANE
-----------------------------
<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>55</td>
<td>0.00</td>
<td>36.84</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>55</td>
<td>0.00</td>
<td>55.16</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

*************** END OF LATEST ANALYSIS RESULT ***************

27. FINISH
INPUT FILE: CLAAS A.STD
3. STAAD PLANE
4. INPUT WIDTH 72
5. UNIT METER MTON
6. PAGE LENGTH 1000
7. UNIT METER MTON
8. JOINT COORDINATES
   9. 1 0.00 0 0; 2 0.4 0 0; 3 10.4 0 0; 4 10.8 0 0
10. MEMBER INCIDENCES
11. 1 2 3
12. MEMBER PROPERTY CANADIAN
13. 1 TO 3 PRI YD 1.0 ZD 1.0
14. CONSTANT
15. E CONCRETE ALL
16. DENSITY CONCRETE ALL
17. POISSON CONCRETE ALL
18. SUPPORT
19. 2 3 PINNED
20. DEFINE MOVING LOAD
21. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
22. LOAD GENERATION 202
23. TYPE 1 -18.8 0. 0. XINC .2
24. PERFORM ANALYSIS
25. LOAD LIST 74
26. PRINT SUPPORT REACTION
   SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = PLANE
---------------------
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM Z
2 74 0.00 26.60 0.00 0.00 0.00 0.00
3 74 0.00 9.80 0.00 0.00 0.00 0.00

************** END OF LATEST ANALYSIS RESULT **************

27. FINISH
Abutment SHAFT ..DRY NORMAL

Depth of Section = 1.200 m  
Width of Section = 12.000 m

along width-compression face- no of bar: 70  
tension face- no of bar: 140
Dia (mm) = 16  
Cover (cm) = 7.5  
Dia (mm) = 25  
Cover (cm) = 10.5
along depth-compression face- no of bar: 8  
tension face- no of bar: 8
Dia (mm) = 16  
Cover (cm) = 7.50  
Dia (mm) = 16  
Cover (cm) = 7.5
Modular Ratio : Compression = 10.0  
Modular Ratio : Tension = 10.0  
Allowable Concrete Stress = 85.00 Kg/cm^2  
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 500.282 T  
Mxx = 1051.256 Tm  
Myy = 137.896 Tm

Intercept of Neutral axis : X axis : = 273.656 m  
: y axis : = .369 m
Concrete Stress Governs Design

Stress in Concrete due to Loads = 60.05 Kg/cm^2  
Stress in Steel due to Loads = 1206.23 Kg/cm^2  
Percentage of Steel = .60 %

Abutment SHAFT ..DRY SPAN DISLODGED

Depth of Section = 1.200 m  
Width of Section = 12.000 m
Modular Ratio : Compression = 10.0  
Modular Ratio : Tension = 10.0  
Allowable Concrete Stress = 85.00 Kg/cm^2  
Allowable Steel Stress = 2040.00 Kg/cm^2
Axial Load = 268.212 T  
Mxx = 993.176 Tm  
Myy = .010 Tm

Intercept of Neutral axis : X axis : = ******* m  
: y axis : = .332 m
Concrete Stress Governs Design

Stress in Concrete due to Loads = 54.70 Kg/cm^2  
Stress in Steel due to Loads = 1255.23 Kg/cm^2
Percentage of Steel = 0.60%

Abutment SHAFT ..HFL NORMAL

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 364.721 T
Mxx = 774.368 Tm
Myy = 137.896 Tm

Intercept of Neutral axis : X axis : = 202.764 m
: y axis : = 0.371 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 44.57 Kg/cm^2
Stress in Steel due to Loads = 894.35 Kg/cm^2
Percentage of Steel = 0.60%

Abutment SHAFT ..HFL SPAN DISLODGED

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 132.289 T
Mxx = 716.289 Tm
Myy = 0.010 Tm

Intercept of Neutral axis : X axis : = ******* m
: y axis : = 0.321 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 39.20 Kg/cm^2
Stress in Steel due to Loads = 945.83 Kg/cm^2
Percentage of Steel = 0.60%
DESIGN OF SUBSTRUCTURE (PIER)
**Design Data:**
For design purposes, following parameters have been considered.

Grade of concrete = M - 25
Pier Cap = M - 25
Grade of reinforcement steel = Fe - 415
Centre to Centre distance of A / Expansion joints = 10.800 m
Centre to Centre distance of Bearing = 10.400 m
Depth of superstructure = 925 mm
Thickness of wearing coat = 56.00 mm
Formation level along C of carriage way = 241.909 m
Soffit level = 240.928
Pedestal top level = 240.928
Height of bearing and Pedestal = 0.000 m
L.W.L./Bed level = 236.941 m
H.F.L = 240.009 m
M.S.L = 234.384 m
Founding Level = 233.441 m
Pier cap top level = 240.928 m
S.B.C of Soil = 37.600 t/m²

Live Load (a) Class A three Lane
(b) Class 70R wheeled/Tracked + Class A

Maximum surface velocity of water = \( 2 \times 1.200 \) m/sec = 1.7 m/sec
Say, 1.70 m/sec

The following codes are used for the design of substructure:
1 IRC : 6 - 2000
2 IRC : 21 - 2000
3 IRC : 78 - 2000
Dead Load

dead load of slab = 24.28 t/m
Total reaction = 262.22 = 262.22 t
wearing coat = 1.36 t/m = 14.636 t
crashbarrier = 1.00 t/m = 21.6 t

As per clause 214.2 of IRC:6, horizontal braking force \( F_h \), for each span is:

For Class A 3 lane:
\[ F_h = 0.2 \times 31.5 + 0.05 \times 31.5 = 7.883 \text{ t} \]

class 70R wheeled + A:
\[ F_h = 0.2 \times 71.5 + 0.05 \times 31.5 = 15.883 \text{ t} \]

\( \mu R_g = 7.4615 \text{ t} \)
\( F_h/2 = 7.9413 \)

Summary of Longitudinal Forces:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>class 70R + A</td>
<td>7.94</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Summary sheet 54-600

<table>
<thead>
<tr>
<th>70RW</th>
<th>( R_R )</th>
<th>( R_C )</th>
<th>( M_L )</th>
<th>( R )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>max rea</td>
<td>32.11</td>
<td>39.42</td>
<td>2.924</td>
<td>71.53</td>
<td>208</td>
</tr>
<tr>
<td>max long</td>
<td>53.32</td>
<td>0</td>
<td>21.328</td>
<td>53.32</td>
<td>155</td>
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<tr>
<td>70RT</td>
<td>max rea</td>
<td>32.7</td>
<td>32.1</td>
<td>0.24</td>
<td>64.8</td>
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<td>max long</td>
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<td>58.19</td>
<td>23.276</td>
<td>58.19</td>
<td>165</td>
</tr>
<tr>
<td>CLASS A</td>
<td>max rea</td>
<td>25.0</td>
<td>6.53</td>
<td>7.388</td>
<td>31.53</td>
</tr>
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<td>max long</td>
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<td>CLASS3A</td>
<td>max rea</td>
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<td>19.59</td>
<td>22.164</td>
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<td>max long</td>
<td>10.92</td>
<td>79.8</td>
<td>27.552</td>
<td>90.72</td>
<td>63.5</td>
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</tbody>
</table>
### H.F.L Condition with L.L

#### a) Vertical load and their moments about C/L of Foundation base.

<table>
<thead>
<tr>
<th></th>
<th>D.L Reaction</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>a Left span</td>
<td>132.19</td>
<td>-0.4</td>
<td>-52.876</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>b Right span</td>
<td>132.19</td>
<td>0.4</td>
<td>52.876</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>a Left span</td>
<td>18.12</td>
<td>-0.4</td>
<td>-7.248</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>b Right span</td>
<td>18.12</td>
<td>0.4</td>
<td>7.248</td>
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<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>300.62</td>
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<td></td>
<td>0</td>
<td>0</td>
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</table>

#### 4 L.L Max Reaction case

<table>
<thead>
<tr>
<th></th>
<th>Left span</th>
<th>70R Wheeled + class A</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70R Wheeled + class A</td>
<td>57.978</td>
<td>-0.4</td>
<td>-23.191</td>
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<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>67.5</td>
<td>-0.4</td>
<td>-27</td>
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<tr>
<td></td>
<td>70R Wheeled + class A</td>
<td>35.307</td>
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<tr>
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<td>Class A 3 Lane</td>
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<td>0.4</td>
<td>7.0524</td>
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</table>

#### 4 L.L Max Long. Moment case

<table>
<thead>
<tr>
<th></th>
<th>Left span</th>
<th>70R Wheeled + class A</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70R Wheeled + class A</td>
<td>71.928</td>
<td>-0.4</td>
<td>-28.771</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>9.828</td>
<td>-0.4</td>
<td>-3.9312</td>
<td></td>
</tr>
<tr>
<td></td>
<td>70R Wheeled + class A</td>
<td>3.276</td>
<td>0.4</td>
<td>1.3104</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Class A 3 Lane</td>
<td>71.82</td>
<td>0.4</td>
<td>28.728</td>
<td></td>
</tr>
</tbody>
</table>

Max Reaction case: 385.75 19.948 164.18
Max Long. moment case: 375.82 27.461 132.36

### II Substructure

<table>
<thead>
<tr>
<th></th>
<th>V</th>
<th>η</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>b Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Pier cap R</td>
<td>3.69</td>
<td>2.4</td>
<td>8.856</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
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<td>2.4</td>
<td>0</td>
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</tr>
<tr>
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<td>0</td>
<td>0</td>
</tr>
<tr>
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<td>52.688</td>
<td>1.4</td>
<td>73.763</td>
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<td>0</td>
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<tr>
<td>4</td>
<td>Footing</td>
<td>35.67</td>
<td>1.4</td>
<td>49.938</td>
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<td>5</td>
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<tr>
<td>6</td>
<td>Water current</td>
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<td>0</td>
<td>7.0012</td>
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<td>19.236</td>
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</tbody>
</table>

223.91 7.0012 19.236
b) Horizontal Forces and Moments with respect to Base

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>H&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<tr>
<td>7.94</td>
</tr>
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<td>7.94</td>
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</tbody>
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**Summary**

Max Reaction case

- \( P = 609.66 \text{ t} \)
- \( M_L = 86.405 \text{ t-m} \)
- \( M_T = 183.42 \text{ t-m} \)

Max Long moment case

- \( P = 599.73 \text{ t} \)
- \( 93.918 \text{ t-m} \)
- \( 151.59 \text{ t-m} \)

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{609.66}{43.05} + \frac{86.405}{88.253} + \frac{183.42}{25.113} = 22.444 \text{ t/m}^2
\]

< 37.6 \text{ t/m}^2  
O.K

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{609.66}{43.05} - \frac{86.405}{88.253} - \frac{183.42}{25.113} = 5.8787 \text{ t/m}^2
\]

> 0.00  
O.K.NO.Tension

Max Long

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{599.73}{43.05} + \frac{93.918}{88.253} + \frac{151.59}{25.113} = 21.032 \text{ t/m}^2
\]

< 37.6 \text{ t/m}^2  
O.K

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{599.73}{43.05} - \frac{93.918}{88.253} - \frac{151.59}{25.113} = 6.8302 \text{ t/m}^2
\]

> 0.00  
O.K.NO.Tension
### H.F.L Condition with L.L

#### a) Vertical load and their moments about Pier Shaft bottom

<table>
<thead>
<tr>
<th>Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
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<td></td>
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<tr>
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<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b Right span</td>
<td>0</td>
<td>2.4</td>
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<td>0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
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<td>3.69</td>
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<td>8.856</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>up to H.F.L</td>
<td>0</td>
<td>1.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>52.688</td>
<td>1.4</td>
<td>73.763</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4 Water current</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19.223</td>
<td>6.9967</td>
<td></td>
</tr>
</tbody>
</table>

#### b) Horizontal Forces and Moments with respect to Pier Shaft bottom

1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.94</td>
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<td>6.487</td>
<td>51.515</td>
<td>6.487</td>
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</table>

<table>
<thead>
<tr>
<th>H_L</th>
<th>H_T</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.94</td>
<td>0</td>
<td></td>
<td>51.515</td>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

**Summary for design of Pier**

<table>
<thead>
<tr>
<th>P</th>
<th>Max Reaction case</th>
<th>Max Long.moment case</th>
</tr>
</thead>
<tbody>
<tr>
<td>P</td>
<td>468.37 t</td>
<td>458.44 t</td>
</tr>
<tr>
<td>M_L</td>
<td>90.686 t-m</td>
<td>98.199 t-m</td>
</tr>
<tr>
<td>M_T</td>
<td>171.18 t-m</td>
<td>139.36 t-m</td>
</tr>
</tbody>
</table>

### H.F.L Condition One Span dislodged

#### a) Vertical load and their moments about C/L of Foundation base.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e_L</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
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<tr>
<td>1 Pedestal</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a Left span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b Right span</td>
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<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>3.69</td>
<td>2.4</td>
<td>8.856</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pier cap Tr</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>up to H.F.L</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>below H.F.L</td>
<td>52.688</td>
<td>1.4</td>
<td>73.763</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4 Footing</td>
<td>35.67</td>
<td>1.4</td>
<td>49.938</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5 Earth above footing</td>
<td>91.348</td>
<td>1</td>
<td>91.348</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6 Water current</td>
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<td>0</td>
<td>7.0012</td>
<td>0</td>
<td>19.236</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| 223.91 | 7.0012 | 19.236 |
Summary

\[
P = t \\
M_L = t \cdot m \\
M_T = t \cdot m
\]

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P + M_L + M_T}{A + Z_L + Z_T}
\]

\[
= \frac{524.53 + 7.0012 + 19.236}{43.05 + 88.253 + 25.113} = \frac{13.029}{37.6} \text{ t/m}^2
\]

O.K

\[
P_{\text{min}} = \frac{P - M_L - M_T}{A + Z_L + Z_T}
\]

\[
= \frac{524.53 - 7.0012 - 19.236}{43.05 + 88.253 + 25.113} = \frac{11.339}{0.00} \text{ t/m}^2
\]

O.K. NO Tension

H.F.L. Condition One Span dislodged

a) Vertical load and their moments about Abutment Shaft bottom.

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e_l</th>
<th>M_L</th>
<th>e_T</th>
<th>M_T</th>
</tr>
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<tbody>
<tr>
<td>1 Pedestal</td>
<td></td>
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<td></td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>a Left span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>3.69</td>
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<td>8.856</td>
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<td>0</td>
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<tr>
<td>3 Pier shaft</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4 Water current</td>
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<td>1.4</td>
<td>73.763</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19.223</td>
<td>6.9967</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Summary for design of Pier

\[
P = 383.24 \text{ t} \\
M_L = 19.223 \text{ t} \cdot \text{m} \\
M_T = 6.9967 \text{ t} \cdot \text{m}
\]
### Dry Condition with L.L

#### a) Vertical load and their moments about C/L of Foundation base.

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>e₁</th>
<th>M₁</th>
<th>e₂</th>
<th>M₂</th>
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<td></td>
</tr>
<tr>
<td>a</td>
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<td>-0.4</td>
<td>-52.876</td>
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</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>132.19</td>
<td>0.4</td>
<td>52.876</td>
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</table>

#### 2 S.I.D.L

<table>
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<th>M₁</th>
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<tbody>
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<td>18.12</td>
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#### 4 L.L Max Reaction case

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<th>M₁</th>
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#### 4 L.L Max Longitudinal Moment case

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<th>e₁</th>
<th>M₁</th>
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<td>0.4</td>
<td>1.3104</td>
<td>1.76</td>
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#### Max Reaction case

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<th>e₁</th>
<th>M₁</th>
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<td>1.3104</td>
<td>1.76</td>
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#### Max Longitudinal Moment

<table>
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<th>M₁</th>
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### II Substructure

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<th>M₁</th>
<th>e₂</th>
<th>M₂</th>
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<td>-0.4</td>
<td>0</td>
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<tr>
<td>b</td>
<td>Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
</tr>
</tbody>
</table>

#### 2 Pier cap R

| Pier cap Tr | 3.69 | 2.4 | 8.856 | 0  | 0  | 0  |

#### 3 Pier shaft

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<th>34.888808</th>
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</table>

#### 4 Footing

|     | 35.67 | 2.4 | 85.608 | 0  | 0  | 0  |

#### 5 Earth above footing

|     | 91.348363 | 1.8 | 164.43 | 0  | 0  | 0  |

<table>
<thead>
<tr>
<th>V</th>
<th>ρ</th>
<th>P</th>
<th>e₁</th>
<th>M₁</th>
<th>e₂</th>
<th>M₂</th>
</tr>
</thead>
<tbody>
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<td>a</td>
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<td>2.4</td>
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<td>-0.4</td>
<td>0</td>
</tr>
<tr>
<td>b</td>
<td>Right span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
</tr>
</tbody>
</table>

| 399.4 | 0 | 0 |

### b) Horizontal Forces and Moments with respect to Base

#### 1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>H₁</th>
<th>H₂</th>
<th>e₁</th>
<th>M₁</th>
<th>e₂</th>
<th>M₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.94</td>
<td>0</td>
<td>7.487</td>
<td>59.456</td>
<td>7.487</td>
<td>0</td>
</tr>
</tbody>
</table>

| 7.94 | 0  | 59.456 | 0  |
Summary

Max Reaction   Max Longitudinal

\[ P = 785.15 \text{ t} \]
\[ M_L = 79.404 \text{ t-m} \]
\[ M_T = 164.18 \text{ t-m} \]


\[ A = 43.05 \]
\[ Z_L = 88.253 \]
\[ Z_T = 25.113 \]

Max reaction case:

Check for Maximum Allowable Base Pressure:

\[ P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} \]

\[ = \frac{785.1512}{43.05} + \frac{79.404}{88.253} + \frac{164.18}{25.113} \]

\[ = 25.676 \text{ t/m}^2 \]

\[ < 37.6 \text{ t/m}^2 \]

O.K.

\[ P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} \]

\[ = \frac{785.1512}{43.05} - \frac{79.404}{88.253} - \frac{164.18}{25.113} \]

\[ = 10.801 \text{ t/m}^2 \]

\[ > 0.00 \text{ t/m}^2 \]

O.K. NO. Tension

Max Longmoment case:

\[ P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T} \]

\[ = \frac{775.22412}{43.05} + \frac{86.917}{88.253} + \frac{132.36}{25.113} \]

\[ = 24.263 \text{ t/m}^2 \]

\[ < 37.6 \text{ t/m}^2 \]

O.K.

\[ P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T} \]

\[ = \frac{775.22412}{43.05} - \frac{86.917}{88.253} - \frac{132.36}{25.113} \]

\[ = 11.752 \text{ t/m}^2 \]

\[ > 0.00 \text{ t/m}^2 \]

O.K. NO. Tension
Design of Footing in Transverse direction

Bending moment at the face of pier $b-b = 211.91$ t-m

Footing is checked for shear at distance of $d$ from face pier

Effective depth $d$ of footing = 0.915 m

Total depth of section at distance $d$ from pier = 0.6611 m

Effective depth at distance $d$ from face of pier = 0.5701 m

Bending moment at a distance of $d$ from face of pier = 4.2466 t-m

Shear force at a distance $d$ from face of pier = 101.8 t

Net shear force at a distance $d$ from face of pier = 99.04 t

(after correction)

Design for Flexure:

\[
M = 30 \quad F_e = 415
\]

Permissible compressive stress

\[
\sigma_{c混} = 10 \text{ N/mm}^2
\]

Permissible tensile stress

\[
\sigma_{st} = 200 \text{ N/mm}^2
\]

$\text{width } b = 12.30$

Effective depth required $d = 341.02$ mm

Effective depth provided $= 915$ mm

$A_{st}$ Required $= 1059.1 \text{ mm}^2/m$

Provide 4 nos 20 dia bars /m width
### Check for shear

Shear stress at effective depth from pier  
\[ = 14.124 \text{ t/m}^2, \quad 0.1385 \]

% of steel provided  
\[ = 0.13 \]

Permissible shear stress  
\[ = 19.38 \text{ t/m}^2 \]

### Dry Condition with L.L.

#### a) Vertical load and their moments about Pier Shaft bottom

<table>
<thead>
<tr>
<th>II Substructure</th>
<th>V</th>
<th>( \rho )</th>
<th>P</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Pedestal</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>-0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>a Left span</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>b Right span</td>
<td>3.69</td>
<td>2.4</td>
<td>8.856</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2 Pier cap R</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier cap Tr</td>
<td>0</td>
<td>2.4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3 Pier shaft</td>
<td>34.88808</td>
<td>2.4</td>
<td>83.733</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>up to G.L.</td>
<td>23.656637</td>
<td>2.4</td>
<td>56.776</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>below G.L.</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

#### b) Horizontal Forces and Moments with respect to Pier Shaft bottom

<table>
<thead>
<tr>
<th>1 Longitudinal Forces at bearing level</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_L )</td>
</tr>
<tr>
<td>7.94</td>
</tr>
<tr>
<td>7.94</td>
</tr>
</tbody>
</table>

### Summary for design of Pier

<table>
<thead>
<tr>
<th>Max Reaction case</th>
<th>Max Longitudinal moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P )</td>
<td>535.12 \text{ t}</td>
</tr>
<tr>
<td>( M_L )</td>
<td>71.462 \text{ t-m}</td>
</tr>
<tr>
<td>( M_T )</td>
<td>164.18 \text{ t-m}</td>
</tr>
</tbody>
</table>
**Dry Condition One Span dislodged**

*a) Vertical load and their moments about C/L of Foundation base.*

### 1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>( H_L )</th>
<th>( H_T )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0</td>
<td>7.487</td>
<td>0</td>
<td>7.487</td>
<td>0</td>
</tr>
<tr>
<td>0.00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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</tbody>
</table>

**Summary**

<table>
<thead>
<tr>
<th>( P )</th>
<th>( M_L )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>=</td>
<td>700.02</td>
<td>t</td>
</tr>
<tr>
<td>=</td>
<td>0</td>
<td>t-m</td>
</tr>
<tr>
<td>=</td>
<td>0</td>
<td>t-m</td>
</tr>
</tbody>
</table>

Check for Maximum Allowable Base Pressure:

\[
P_{\text{max}} = \frac{P}{A} + \frac{M_L}{Z_L} + \frac{M_T}{Z_T}
\]

\[
= \frac{700.02}{43.05} + \frac{0}{88.253} + \frac{0}{25.113} = 16.261 \text{ t/m}^2
\]

< 37.6 \text{ t/m}^2

O.K

\[
P_{\text{min}} = \frac{P}{A} - \frac{M_L}{Z_L} - \frac{M_T}{Z_T}
\]

\[
= \frac{700.02}{43.05} - \frac{0}{88.253} - \frac{0}{25.113} = 16.261 \text{ t/m}^2
\]

> 0.00

O.K. NO Tension

**Dry Condition One Span dislodged**

*a) Vertical load and their moments about Abutment Shaft bottom.*

### 1 Longitudinal Forces at bearing level

<table>
<thead>
<tr>
<th>( H_L )</th>
<th>( H_T )</th>
<th>( e_L )</th>
<th>( M_L )</th>
<th>( e_T )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0</td>
<td>6.487</td>
<td>0</td>
<td>6.487</td>
<td>0</td>
</tr>
<tr>
<td>0.00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

**Summary for design of Pier**

<table>
<thead>
<tr>
<th>( P )</th>
<th>( M_L )</th>
<th>( M_T )</th>
</tr>
</thead>
<tbody>
<tr>
<td>=</td>
<td>449.99</td>
<td>t</td>
</tr>
<tr>
<td>=</td>
<td>0</td>
<td>t-m</td>
</tr>
<tr>
<td>=</td>
<td>0</td>
<td>t-m</td>
</tr>
</tbody>
</table>
Summary of Loads at Pier Shaft bottom:

<table>
<thead>
<tr>
<th>1 DRY condition</th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Reaction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With L.L.</td>
<td>535.12</td>
<td>71.46</td>
<td>164.18</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>449.99</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Max Long</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With L.L.</td>
<td>525.19</td>
<td>78.98</td>
<td>132.36</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>449.99</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2 H.F.L.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Reaction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With L.L.</td>
<td>468.37</td>
<td>90.69</td>
<td>171.18</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>383.24</td>
<td>19.22</td>
<td>7.00</td>
</tr>
<tr>
<td>Max Long</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With L.L.</td>
<td>458.44</td>
<td>98.20</td>
<td>139.36</td>
</tr>
<tr>
<td>Span dislodged</td>
<td>383.24</td>
<td>19.22</td>
<td>7.00</td>
</tr>
</tbody>
</table>

Check for Cracked/Uncracked Section

- Length of section = 11908.982 mm
- Width of section = 800 mm
- Gross Area of section \( A_g \) = 9527185.2 mm^2
- Gross M.O.I of section \( I_{gx} \) = 5.081E+11 mm^4
- Gross M.O.I of section \( I_{gy} \) = 1.126E+14 mm^4

Transformed sectional properties of section:

Adopting
- Modular ratio \( m \) = 10
- Cover = 70
- Dia of Bars = 20
- No of bars in tension face (longer) = 45
- No of bars in compression face = 45
No of bars in shorter direction = 6
Total bars in section = 102

Steel Area $A_s$ = 30687 mm$^2$
% of Steel = 0.3221%

Area of concrete $A_c = A_g - A_s = 9496498.2$ mm$^2$
C.G of Steel placed on longer face = 330 mm
C.G of Steel placed on shorter face = 5884.5 mm

Transformed Area of Section $A_{fin} = 9803368.9$ mm$^2$

Transformed M.I$_{xx} = I_{gxx} + 2 \frac{m}{1} A_s ax^2$
$= 5.35828E+11$ mm$^4$

$A_{sx} = 14137$ mm$^2$
$A_{sy} = 1885$ mm$^2$

Transformed M.I$_{yy} = I_{gyy} + 2 \frac{m}{1} A_s ay^2$
$= 1.13773E+14$ mm$^4$

$Z_{sx} = \frac{M_{I_{xx}}}{d/2}$
$= 1.34E+09$ mm$^3$

$Z_{sy} = \frac{M_{I_{yy}}}{d/2}$
$= 1.911E+10$ mm$^3$

**Permissible stresses**
Minimum Gross Moment of inertia $I_{min} = 5.081E+11$ mm$^4$
Area of section = 9527185.2 mm$^2$
$r = 230.94011$ mm

**Effective length of Abutment shaft** (IRC:21-2000 cl: 306.2.1)
Abutment shaft height $L = 6.187$ m
Effective length $L_{eff} = 7.4244$ m
Slenderness ratio = 32.149 < 50
Type of member = 1 Short Column

**Stress reduction coefficient** (IRC:21-2000 cl: 306.4.2,3)
$\beta = 1$

**Permissible stresses**

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{cbc}$</td>
<td>$\sigma_{st}$</td>
</tr>
<tr>
<td>8.3333 N/mm$^2$</td>
<td>200 N/mm$^2$</td>
</tr>
<tr>
<td>$\sigma_{co}$</td>
<td></td>
</tr>
<tr>
<td>6.25 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>Tensile stress</td>
<td></td>
</tr>
<tr>
<td>0.61 N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>S.No</td>
<td>Item</td>
</tr>
<tr>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>P</td>
</tr>
<tr>
<td>2</td>
<td>$M_L$</td>
</tr>
<tr>
<td>3</td>
<td>$M_T$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>$\sigma_{co,cal}$</td>
</tr>
<tr>
<td>5</td>
<td>$\sigma_{cbc,cal}$</td>
</tr>
<tr>
<td>6</td>
<td>$\sigma_{cbc,cal}$</td>
</tr>
<tr>
<td>7</td>
<td>$\sigma_{cbc,cal} = 5 + 6$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>$\sigma_{cbc}$</td>
</tr>
<tr>
<td>9</td>
<td>$\sigma_{co}$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Conc.Area Required for directstress</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of $A_g$</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel $\text{mm}^2$</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area $\text{mm}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma_{co,cal} + \sigma_{cbc,cal}$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{co}$ $\sigma_{cbc}$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma_{co,cal} - \sigma_{cbc,cal}$</td>
</tr>
<tr>
<td></td>
<td>Permissible Basic tensile stress in concrete</td>
</tr>
<tr>
<td></td>
<td>Section to be designed as</td>
</tr>
<tr>
<td>S.No</td>
<td>Item</td>
</tr>
<tr>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>P</td>
</tr>
<tr>
<td>2</td>
<td>M_L</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

| 4    | \( \sigma_{co,cal} \) | P/A_{min} | 0.477764624 | 0.3909261 |
| 5    | \( \sigma_{cbc,cal} \) | M_L/Z_{xx} | 0.676977088 | 0.143504  |
| 6    | \( \sigma_{cbc,cal} \) | M_T/Z_{yy} | 0.089588606 | 0.0036618 |
| 7    | \( \sigma_{cbc,cal} = 5 + 6 \) | 0.766565695 | 0.1471658 |

**Permissible Stresses**

| 8    | \( \sigma_{cbc} \) | 8.33333333 | 8.3333    |
| 9    | \( \sigma_{co} \) | 6.25       | 6.25      |

**Check for Minimum steel area mm²**

| 10   | Conc.Area Required for directstress \( (1)/(9) \) | 749392.459 | 613183   |
| 11   | 0.8% of area required | 5995.13967 | 4905.5   |
| 12   | 0.3% of \( A_g \) | 28581.5557 | 28582    |
| 13   | Governing steel mm² | 28581.5557 | 28582    |
| 14   | Provided Steel area mm² | 30687.077 | 30687    |

**Check for safety of section**

\[
\frac{\sigma_{co,cal}}{\sigma_{co}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}} < 1
\]

**Check for Cracked /Uncracked section**

| \( \sigma_{co,cal} - \sigma_{cbc,cal} \) | -0.2888011 | 0.2438 |

Permissible Basic tensile stress in concrete -0.61 -0.61

Section to be designed as | Uncracked | Uncracked |
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>DRY Case (Max Longitudinal moment)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loads and Moments</td>
</tr>
<tr>
<td>1</td>
<td>P</td>
<td>525.19 t</td>
</tr>
<tr>
<td>2</td>
<td>M_L</td>
<td>78.98 t-m</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>132.36 t-m</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{\text{cal}}$</th>
<th>P/A</th>
<th>$\sigma_{\text{cal}}$</th>
<th>$M_L/Z_{xx}$</th>
<th>$\sigma_{\text{cal}}$</th>
<th>$M_T/Z_{yy}$</th>
<th>$\sigma_{\text{cal}}$ = $\sigma_{\text{cal}}$ + $\sigma_{\text{cal}}$</th>
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</thead>
<tbody>
<tr>
<td>4</td>
<td>$\sigma_{\text{cal}}$</td>
<td>P/A</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_L/Z_{xx}$</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_T/Z_{yy}$</td>
<td>$\sigma_{\text{cal}}$ = $\sigma_{\text{cal}}$ + $\sigma_{\text{cal}}$</td>
</tr>
<tr>
<td>5</td>
<td>$\sigma_{\text{cal}}$</td>
<td>P/A</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_L/Z_{xx}$</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_T/Z_{yy}$</td>
<td>$\sigma_{\text{cal}}$ = $\sigma_{\text{cal}}$ + $\sigma_{\text{cal}}$</td>
</tr>
<tr>
<td>6</td>
<td>$\sigma_{\text{cal}}$</td>
<td>P/A</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_L/Z_{xx}$</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_T/Z_{yy}$</td>
<td>$\sigma_{\text{cal}}$ = $\sigma_{\text{cal}}$ + $\sigma_{\text{cal}}$</td>
</tr>
<tr>
<td>7</td>
<td>$\sigma_{\text{cal}}$</td>
<td>P/A</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_L/Z_{xx}$</td>
<td>$\sigma_{\text{cal}}$</td>
<td>$M_T/Z_{yy}$</td>
<td>$\sigma_{\text{cal}}$ = $\sigma_{\text{cal}}$ + $\sigma_{\text{cal}}$</td>
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</tbody>
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**Permissible Stresses**

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{\text{cal}}$</th>
<th>P/A</th>
<th>$\sigma_{\text{cal}}$</th>
<th>P/A</th>
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</thead>
<tbody>
<tr>
<td>8</td>
<td>$\sigma_{\text{bc}}$</td>
<td>8.3333333</td>
<td>8.3333333</td>
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</tr>
<tr>
<td>9</td>
<td>$\sigma_{\text{bc}}$</td>
<td>6.25</td>
<td>6.25</td>
<td></td>
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</tbody>
</table>

**Check for Minimum steel area mm²**

<table>
<thead>
<tr>
<th></th>
<th>1) Concentration required for direct stress</th>
<th>(1)/(9)</th>
<th>719976.11</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.8% of area required</td>
<td>6722.4201</td>
<td>5759.8089</td>
</tr>
<tr>
<td>11</td>
<td>0.3% of healthy</td>
<td>28581.556</td>
<td>28581.556</td>
</tr>
<tr>
<td>12</td>
<td>Governing steel</td>
<td>28581.556</td>
<td>28581.556</td>
</tr>
<tr>
<td>13</td>
<td>Provided Steel area</td>
<td>30687.077</td>
<td>30687.077</td>
</tr>
</tbody>
</table>

**Check for safety of section**

\[
\frac{\sigma_{\text{cal}} + \sigma_{\text{cal}}}{\sigma_{\text{bc}}} < 1 \quad \frac{\sigma_{\text{cal}} + \sigma_{\text{cal}}}{\sigma_{\text{bc}}} < 1
\]

**Check for Cracked /Uncracked section**

<table>
<thead>
<tr>
<th></th>
<th>$\sigma_{\text{cal}}$</th>
<th>$\sigma_{\text{cal}}$</th>
<th>Permissible Basic tensile stress in concrete</th>
<th>Section to be designed as</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>0.1647755</td>
<td>0.0734417</td>
<td>$\sigma_{\text{cal}}$ - $\sigma_{\text{cal}}$</td>
<td>Uncracked</td>
</tr>
<tr>
<td>15</td>
<td>-0.123109</td>
<td>0.4590106</td>
<td>$\sigma_{\text{cal}}$ - $\sigma_{\text{cal}}$</td>
<td>Uncracked</td>
</tr>
</tbody>
</table>

Permissible Basic tensile stress in concrete -0.61 -0.61

Section to be designed as Uncracked Uncracked
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>H.F.L Case (Max Longitudinal)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loads and Moments</td>
</tr>
<tr>
<td>1</td>
<td>P</td>
<td>458.44 t</td>
</tr>
<tr>
<td>2</td>
<td>M_L</td>
<td>98.20 t-m</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>139.36 t-m</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>( \sigma_{\text{co,cal}} ) P/A_{\text{flm}}</td>
<td>0.467638513</td>
</tr>
<tr>
<td>5</td>
<td>( \sigma_{\text{cbc,cal}} ) M_L/Z_{xx}</td>
<td>0.73306372</td>
</tr>
<tr>
<td>6</td>
<td>( \sigma_{\text{cbc,cal}} ) M_T/Z_{yy}</td>
<td>0.072933819</td>
</tr>
<tr>
<td>7</td>
<td>( \sigma_{\text{cbc,cal}} = 5 + 6 )</td>
<td>0.805997538</td>
</tr>
</tbody>
</table>

**Permissible Stresses**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
<td>8</td>
<td>( \sigma_{\text{cbc}} )</td>
<td>8.3333333</td>
</tr>
<tr>
<td>9</td>
<td>( \sigma_{\text{co}} )</td>
<td>6.25</td>
</tr>
</tbody>
</table>

**Check for Minimum steel area mm\(^2\)**

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Conc.Area Required for directstress ((1)/(9))</td>
<td>733509.26</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
<td>5868.0741</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of A_g</td>
<td>28581.556</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel mm(^2)</td>
<td>28581.556</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area mm(^2)</td>
<td>30687.077</td>
</tr>
</tbody>
</table>

**Check for safety of section**

\[
\frac{\sigma_{\text{co,cal}}}{\sigma_{\text{co}}} + \frac{\sigma_{\text{cbc,cal}}}{\sigma_{\text{cbc}}} < 1
\]

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Check for Cracked /Uncracked section**

\[
\sigma_{\text{co,cal}} - \sigma_{\text{cbc,cal}}
\]

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Permissible Basic tensile stress in concrete: -0.61, -0.61

Section to be designed as: Uncracked, Uncracked
0.3
0.000

0.8

0.5

3.5

1.000

0.000
Dimensions of Substructure & Foundation

1 Pedestal

Length = 0.7
Width = 0.55  Volume = 0
Height = 0

2 Pier cap
a  Top uniform portion

Width = 1.00
Depth = 0.3  Volume = 3.69 m³
Length = 12.30

b  Bottom trapezoidal portion

Width = 1
Depth = 0  Volume = 0 m³
Length = 12.3

Area at level 240.628 m = 12.3 x 1 = 12.3 m²
Area at level 240.628 m = 1 x 12 = 12 m²  = 3.69 m³
3 Pier shaft
Dimension of pier shaft in longitudinal direction = 11.20
Dimension of pier shaft in Transverse direction
\[ R = 0.00 \quad \text{Area} = 9.46 \text{ m}^2 \]
\[ C = 0.80 \]
Height of pier shaft = 6.19
Height above Ground level = 3.687 m Volume = 34.8888
Height below Ground level = 2.500 m Volume = 23.6566
\[ \text{Volume} = 58.5454 \text{ m}^3 \]

H.F.L Case:
Height above H.F.L = 0.000 m Volume = 0
Height below H.F.L = 5.57 m Volume = 52.6881
\[ \text{Volume} = 52.6881 \text{ m}^3 \]

4 Pedestal at footing top Width = 12.3
<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>12.30</td>
</tr>
<tr>
<td>Volume</td>
<td>0 m$^3$</td>
</tr>
<tr>
<td>Height</td>
<td>0</td>
</tr>
</tbody>
</table>

5 Footing

at foundation level

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>12.30 m</td>
</tr>
<tr>
<td>Length</td>
<td>3.50 m</td>
</tr>
<tr>
<td>Thickness at Root</td>
<td>1 m</td>
</tr>
<tr>
<td>Thickness at Tip</td>
<td>0.5 m</td>
</tr>
</tbody>
</table>

Volume = 35.67 m$^3$
Volume of Overburdened earth below ground level

Total volume \( 12.300 \times 3.500 \times 3.500 \) = 150.675 \( m^3 \)

Net volume below Ground level = 91.3484 \( m^3 \)

Sectional Properties of Footing

\[ A = 12.300 \times 3.500 = 43.050 \, m^2 \]
\[ Z_L = 3.500 \times 25.215 = 88.253 \, m^3 \]
\[ Z_T = 12.300 \times 2.04167 = 25.113 \, m^3 \]

Forces and Moments due to Watercurrent Force

Intensity of water current pressure = 52 \( k \, V^2 \) \( kg/m^3 \)

Assuming 20 degree variation in water current direction

\[
\begin{array}{c}
\text{H.F.L} \\
4.00 \\
5.625 \\
\text{Top of footing} \\
0.04053 \\
\text{Foundation level} \\
0.0001 \\
\text{Deepest scour} \\
0
\end{array}
\]

Water current Pressure in Transverse direction

<table>
<thead>
<tr>
<th>R.L</th>
<th>k</th>
<th>Pressure = 52 ( k , V^2 ) COS 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>H.F.L</td>
<td>240.009</td>
<td>0.66</td>
</tr>
<tr>
<td>Pier Shaft bottom</td>
<td>234.441</td>
<td>0.66</td>
</tr>
<tr>
<td>Top of footing</td>
<td>234.441</td>
<td>1.5</td>
</tr>
<tr>
<td>Bottom of footing</td>
<td>233.441</td>
<td>1.5</td>
</tr>
</tbody>
</table>
### Water current Force in Transverse direction

<table>
<thead>
<tr>
<th>Component</th>
<th>Length m</th>
<th>Height m</th>
<th>Force t</th>
<th>HT of force</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier</td>
<td>11.20</td>
<td>5.568</td>
<td>4.06311</td>
<td>4.73119</td>
<td>19.2234 t-m</td>
</tr>
<tr>
<td>Top of footing</td>
<td>12.300</td>
<td>1.000</td>
<td>0.01832</td>
<td>0.66585</td>
<td>0.0122 t-m</td>
</tr>
</tbody>
</table>
Water Current Pressure in Longitudinal direction

<table>
<thead>
<tr>
<th>R.L</th>
<th>k</th>
<th>Pressure = 52 k V^2 SIN 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>H.F.L</td>
<td>240.009</td>
<td>0.66</td>
</tr>
<tr>
<td>Pier Shaft bottom</td>
<td>234.441</td>
<td>0.66</td>
</tr>
<tr>
<td>Top of footing</td>
<td>234.441</td>
<td>1.5</td>
</tr>
<tr>
<td>Bottom of footing</td>
<td>233.441</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Water current Force in Longitudinal direction

<table>
<thead>
<tr>
<th>Component</th>
<th>Length m</th>
<th>Height m</th>
<th>Force t</th>
<th>HT of force t</th>
<th>Moment t-m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier</td>
<td>11.20</td>
<td>5.568</td>
<td>1.47885</td>
<td>4.73119</td>
<td>6.99673</td>
</tr>
<tr>
<td>Top of footing</td>
<td>12.300</td>
<td>1.000</td>
<td>0.00667</td>
<td>0.66585</td>
<td>0.00444</td>
</tr>
</tbody>
</table>

Summary at Founding level

<table>
<thead>
<tr>
<th>F</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal direction</td>
<td>4.08143</td>
</tr>
<tr>
<td>Transverse direction</td>
<td>1.48552</td>
</tr>
</tbody>
</table>

Summary at PierShaf bottom level

<table>
<thead>
<tr>
<th>F</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal direction</td>
<td>4.06311</td>
</tr>
<tr>
<td>Transverse direction</td>
<td>1.47885</td>
</tr>
</tbody>
</table>
DESIGN OF PIER CAP

The pier cap is supported by pier shaft on all sides. Therefore pier cap has been designed as per clause 716.2.1, I.R.C:78-1987

The pier cap shall be reinforced with a total minimum of 1% steel in both directions as per clause 716.2.1, I.R.C:78-1987

Length of pier cap = 12 m
width of pier cap = 1
Depth of pier cap = 0.3

Reinforcement in the direction of length of pier

Area of steel required (mm$^2$) = 1500 mm$^2$ in each face
Providing steel by distributing equally at top & bottom = 8 16 dia

Area of steel provided (mm$^2$) = 1608.495 mm$^2$
Therefore providing 16 dia 8 nos

Reinforcement in the direction of width of pier

Therefore providing 12 dia stirrups @ 150 mm c/c 8 nos 16 dia
REINFORCEMENT DETAILS OF PIER SHAFT

Design of pier wall

The pier wall on a conservative side has been analysed as a rectangular section (neglecting the semicircular cut & ease water on both end) subjected to combined axial load and biaxial moments with the help of computer programme. The grade of concrete for pier shall be M30. The section of pier is checked conservatively at top of foundation. The minimum area of steel provided is 0.3% of gross cross-sectional area as per IRC:21-2000

Minimum Longitudinal Reinforcement required in pier shaft:

0.3% of the gross cross sectional area of pier shaft = 28800 mm²

Provided reinforcement = 30687.08 mm²

Transverse Reinforcement provided in pier shaft:
(refer clause 306.3 IRC:21-2000)

Thick of pier (least lateral dimension) = 800 mm
Dia of largest longitudinal bar = 20 mm
dia of lateral ties required = 5

but >= 8 mm

The pitch of transverse reinforcement shall not exceed the lesser of the following

1 Least lateral dimension of pier = 800
2 twelve times dia of smallest longitudinal reinforcement in pier shaft = 240
3 = 300

Therefore allowable pitch of lateral ties = 240

Provide transverse reinforcement

8 dia of lateral ties 150 mm c/c
SLENDERNESS EFFECT OF PIER WALL

Calculation of slenderness effect of pierwall:

height of pier shaft = h = 6.187

effective length of pier wall under service condition = 1.2 h = 7.4244
(refer clause 306.1 IRC:21-2000)

Thickness of pier wall = 0.8

Radius of gyration = r = \frac{d}{\sqrt{12}} = 0.23094

Slenderness ratio of pier wall = \frac{1.2 \times h}{r} = 32.1486 < 50

short column effect predominant

Since the pier shaft is acting as a short column, The reduction coefficient is 1

\beta = 1.5 - \frac{l}{100r} = 1.031166

Reduced value of permissible stresses for steel and concrete are:

Grade of concrete = M30

concrete = 1051.79 t/m²

steel = 21035.79 t/m²

FOR ONE SPAN DISLODGED CONDITION:

Effective length of pier wall under service condition is = 1.75*h = 10.82725

Slenderness ratio = \frac{46.88337}{50} < 50

short column effect predominant

Since the pier shaft is acting as a short column, The reduction coefficient is 1

\beta = 1.5 - \frac{l}{100r} = 1.031166

Reduced value of permissible stresses for steel and concrete are:

Grade of concrete = M30

concrete = 1051.79 t/m²

steel = 21035.79 t/m²
INPUT FILE: 70RW.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
   7. 1 0.02 0 0; 2 0.4 0 0; 3 10.4 0 0; 4 10.78 0 0; 5 10.82 0 0
   8. 6 11.2 0 0; 7 21.2 0 0; 8 21.58 0 0
9. MEMBER INCIDENCES
10. 1 1 2 7
11. MEMBER PROPERTY CANADIAN
12. 1 TO 7 PRI YD 1.0 ZD 1.0
13. CONSTANT
14. E CONCRETE ALL
15. DENSITY CONCRETE ALL
16. POISSON CONCRETE ALL
17. SUPPORT
18. 2 3 6 7 PINNED
19. MEMBER RELEASE
20. 4 START FX FY FZ MX MY MZ
21. 4 END FX FY FZ MX MY MZ
22. DEFINE MOVING LOAD
23. TYPE 1 LOAD 8.0 $2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
24. LOAD GENERATION 175
25. TYPE 1 -13.4 0 0 XINC .2
26. PERFORM ANALYSIS
27. ***MAX REA
28. LOAD LIST 78
29. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE
-----------------
<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM Z</th>
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</thead>
<tbody>
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<td>2</td>
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<tr>
<td>3</td>
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<td>0.00</td>
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<td>11.58</td>
<td>0.00</td>
<td>0.00</td>
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</table>
30. ***MAX MOM
31. LOAD LIST 54
32. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE
-----------------
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<thead>
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<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
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<td>54</td>
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</tbody>
</table>
33. FINISH
INPUT FILE: CLASS A.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
   7. 1 0.02 0 0;2 0.4 0 0;3 10.4 0 0;4 10.78 0 0;5 10.82 0 0
   8. 6 11.2 0 0;7 21.2 0 0;8 21.58 0 0
9. MEMBER INCIDENCES
10. 1 1 2 7
11. MEMBER PROPERTY CANADIAN
12. 1 TO 7 PRI YD 1.0 ZD 1.0
13. CONSTANT
14. E CONCRETE ALL
15. DENSITY CONCRETE ALL
16. POISSON CONCRETE ALL
17. SUPPORT
18. 2 3 6 7 PINNED
19. MEMBER RELEASE
20. 4 START FX FY FZ MX MY MZ
21. 4 END FX FY FZ MX MY MZ
22. DEFINE MOVING LOAD
23. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
24. LOAD GENERATION 202
25. TYPE 1 -18.8 0. 0. XINC .2
26. PERFORM ANALYSIS
27. ***MAX REA
28. LOAD LIST 121
29. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
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<td>121</td>
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<td>6.53</td>
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<td>0.00</td>
<td>0.00</td>
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<tr>
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<td>121</td>
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<td>13.87</td>
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<td>0.00</td>
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<td>0.00</td>
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</tbody>
</table>

30. ***MAX MOMENT
31. LOAD LIST 128
32. PRINT SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE

<table>
<thead>
<tr>
<th>JOINT</th>
<th>LOAD</th>
<th>FORCE-X</th>
<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
<th>MOM-Z</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.75</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
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<td>128</td>
<td>0.00</td>
<td>3.64</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
<td>6</td>
<td>128</td>
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<td>128</td>
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<td>9.80</td>
<td>0.00</td>
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</tr>
</tbody>
</table>

33. FINISH
DESIGN OF SUPERSTRUCTURE
For Design of superstructure of solid slab of 10.0 m refer MOST STANDARD Drawing titled “STANDARD PLANS FOR 3.0 m TO 10.0 M SPAN REINFORCED CEMENT CONCRETE (Solid slab superstructure With & without footpaths)) FOR HIGHWAYS” Drg. No. SD/114.
BRIDGE AT CH:58+900
DESIGN OF SUBSTRUCTURE
**DESIGN DATA**

Formation Level = 245.451 m  
Ground Level = 239.785 m  
Lowest Water Level = 235.785 m  
Highest Flood Level = 242.295 m  
Founding Level = 235.785 m  
Thickness of bearing & pedestal = 0.300 m  
Width of abutment = 12.000 m  
Bouyancy factor = 1.0  
Gross Safe Bearing Capacity = 32.000 t/sqm  
Dry density of earth = 1.800 t/cum  
Submerged density of earth = 1.0 t/cum  
Saturated density of earth = 2.000 t/cum  
Coefficient of base friction = 0.5  
Span (c/c of exp. joint) = 21.600 m  
Overall Width of deck slab = 12.000 m  
Width of carriageway = 11.000 m  
Width of crash barrier = 0.500 m  
Depth of Superstructure = 2.100 m  
Thickness of wearing coat = 0.056 m  
Unit wt of concrete = 2.400 t/m$^3$  
no. of elastomeric bearing = 4  
size of elastomer brgs. = 500 x 250 x 50 mm  
Grade of Concrete = M 25 (HYSD)  
Grade of Reinforcement = Fe 415  
Live Load = One Lane of 70R Wheeled + Class A  
- 3 lanes of Class A  
Permissible Compressive stress in Concrete = 850 t/m$^2$  
Permissible Tensile stress in Steel = 20400 t/m$^2$  
Modular ratio, m = 10  
factor, k = 0.294  
Lever arm factor, j = 0.902  
Moment of Resistance = 113 t/m$^2$  
Thickness of returnwall = 0.5 m

**COEFFICIENT OF ACTIVE EARTH PRESSURE**

As per Coulomb's theory, coefficient of active earth pressure is 

\[
K_a = \frac{\sin^2(\alpha - \phi)}{1 + \sqrt{\frac{\sin(\alpha + \varphi)\sin(\alpha - \delta)}{\sin(\alpha - \delta)\sin(\varphi + \iota)}}} 
\]

Where  

- $\phi$ = angle of internal friction of earth  
- $\alpha$ = angle of inclination of back of wall  
- $\delta$ = angle of internal friction between wall & earth  
- $\iota$ = angle of inclination of backfill  

Here  

<table>
<thead>
<tr>
<th>Angle (°)</th>
<th>Radian</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$</td>
<td>30°</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>90°</td>
</tr>
<tr>
<td>$\delta$</td>
<td>20°</td>
</tr>
<tr>
<td>$\iota$</td>
<td>0°</td>
</tr>
</tbody>
</table>

$K_a = 0.2973$  

Therefore, horizontal coefficient of active earth pressure = $K_a \cos \phi = K_{ha} = 0.2794$
HEIGHT OF ABUTMENT

Total height of abutment = Formation Level - Founding Level = 9.666 m
For DESIGN purpose, the height of abutment is considered as, say, = 9.670 m

CALCULATION OF ACTIVE EARTH PRESSURE

DRY. condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>77.78</td>
<td>4.833</td>
<td>375.89</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>9.666</td>
<td>4.86</td>
<td>281.92</td>
<td>4.060</td>
<td>1144.50</td>
</tr>
<tr>
<td>3</td>
<td>SubmgEarth</td>
<td>1.0</td>
<td>0.000</td>
<td>4.86</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>0.000</td>
<td>4.86</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>359.69</td>
<td>1520.38</td>
<td></td>
</tr>
</tbody>
</table>

b) Span Dislodge Condition

Net force = 359.69 - 77.78 = 281.92 t
Net moment = 1520.38 - 375.89 = 1144.50 tm

H.F.L. condition

a) Service Condition

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.67</td>
<td>77.78</td>
<td>4.833</td>
<td>375.89</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>3.156</td>
<td>1.76</td>
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<td>7.836</td>
<td>261.65</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>1.0</td>
<td>6.510</td>
<td>1.76</td>
<td>137.76</td>
<td>3.255</td>
<td>448.42</td>
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<tr>
<td>4</td>
<td>SubmgEarth</td>
<td>0.5</td>
<td>6.510</td>
<td>1.82</td>
<td>71.04</td>
<td>2.170</td>
<td>154.16</td>
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<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>319.97</td>
<td>1240.12</td>
<td></td>
</tr>
</tbody>
</table>

b) Span Dislodge Condition

Net force = 319.97 - 77.78 = 242.20 t
Net moment = 1240.12 - 375.89 = 864.23 tm
**Forces & moments due to Abutment (Concrete) components**

**DRY Case :**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment about toe (t/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Toe Slab</td>
<td>1.0</td>
<td>2.200</td>
<td>12.00</td>
<td>0.500</td>
<td>2.40</td>
<td>31.68</td>
<td>1.100</td>
<td>34.85</td>
</tr>
<tr>
<td>2</td>
<td>Heel Slab</td>
<td>0.5</td>
<td>2.200</td>
<td>12.00</td>
<td>0.700</td>
<td>2.40</td>
<td>22.18</td>
<td>1.467</td>
<td>32.52</td>
</tr>
<tr>
<td>3</td>
<td>Stem Wall</td>
<td>1.0</td>
<td>3.600</td>
<td>12.00</td>
<td>0.700</td>
<td>2.40</td>
<td>51.84</td>
<td>5.200</td>
<td>269.57</td>
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<td>4</td>
<td>Stem Wall</td>
<td>0.5</td>
<td>3.600</td>
<td>12.00</td>
<td>0.700</td>
<td>2.40</td>
<td>238.81</td>
<td>4.600</td>
<td>166.92</td>
</tr>
<tr>
<td>5</td>
<td>Heel Slab</td>
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<td>1.200</td>
<td>12.00</td>
<td>6.910</td>
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<td>41.47</td>
<td>2.800</td>
<td>116.12</td>
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<td>6</td>
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<td>0.000</td>
<td>12.00</td>
<td>6.910</td>
<td>2.40</td>
<td>0.00</td>
<td>2.200</td>
<td>0.00</td>
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<tr>
<td>7</td>
<td>stem rect</td>
<td>1.0</td>
<td>1.200</td>
<td>12.00</td>
<td>1.200</td>
<td>2.40</td>
<td>10.37</td>
<td>2.800</td>
<td>29.03</td>
</tr>
<tr>
<td>10</td>
<td>Cap</td>
<td>1.0</td>
<td>1.200</td>
<td>12.00</td>
<td>0.300</td>
<td>2.40</td>
<td>18.48</td>
<td>1.100</td>
<td>20.33</td>
</tr>
<tr>
<td>11</td>
<td>Dirt Wall</td>
<td>1.0</td>
<td>0.300</td>
<td>12.00</td>
<td>2.106</td>
<td>2.40</td>
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<tr>
<td>8</td>
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<td>0.5</td>
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<td>1.00</td>
<td>0.700</td>
<td>2.40</td>
<td>30.24</td>
<td>5.200</td>
<td>157.25</td>
</tr>
<tr>
<td>8a</td>
<td>stem rect</td>
<td>1.0</td>
<td>3.600</td>
<td>1.00</td>
<td>9.166</td>
<td>2.40</td>
<td>238.81</td>
<td>4.600</td>
<td>166.92</td>
</tr>
<tr>
<td>TOTAL</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**H.F.L. Case :**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>C.G. from toe (m)</th>
<th>Moment @ Toe (t/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Toe Slab</td>
<td>1.0</td>
<td>2.200</td>
<td>12.00</td>
<td>0.500</td>
<td>1.40</td>
<td>18.48</td>
<td>1.100</td>
<td>20.33</td>
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<td>2</td>
<td>Heel Slab</td>
<td>0.5</td>
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<td>12.00</td>
<td>0.700</td>
<td>1.40</td>
<td>12.94</td>
<td>1.467</td>
<td>18.97</td>
</tr>
<tr>
<td>3</td>
<td>Stem Wall</td>
<td>1.0</td>
<td>3.600</td>
<td>12.00</td>
<td>0.500</td>
<td>1.40</td>
<td>30.24</td>
<td>5.200</td>
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</tr>
<tr>
<td>4</td>
<td>Stem Wall</td>
<td>0.5</td>
<td>3.600</td>
<td>12.00</td>
<td>0.700</td>
<td>1.40</td>
<td>21.17</td>
<td>4.600</td>
<td>97.37</td>
</tr>
<tr>
<td>5</td>
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<td>0.000</td>
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<td>299.74</td>
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<td>0.000</td>
<td>12.00</td>
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<td>1.40</td>
<td>107.05</td>
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<td>299.74</td>
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<td>1.200</td>
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<td>Cap</td>
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<td>0.300</td>
<td>2.40</td>
<td>10.37</td>
<td>2.800</td>
<td>29.03</td>
</tr>
<tr>
<td>11</td>
<td>Dirt Wall</td>
<td>1.0</td>
<td>0.300</td>
<td>12.00</td>
<td>2.11</td>
<td>2.40</td>
<td>18.20</td>
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<td>1.00</td>
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<td>238.81</td>
<td>4.600</td>
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<tr>
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<td>3.600</td>
<td>1.00</td>
<td>9.166</td>
<td>2.40</td>
<td>238.81</td>
<td>4.600</td>
<td>166.92</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
# Forces & moments due to Earth and LL surcharge

## DRY Case: Self weight of Earth

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>Moment @ Toe (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>DRY EARTH</td>
<td>0.5</td>
<td>3.600</td>
<td>11.00</td>
<td>0.700</td>
<td>1.8</td>
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<tr>
<td>TOTAL</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>628.40</td>
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</table>

## H.F.L Case:

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>Moment @ Toe (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>SATURATED SOIL</td>
<td>0.5</td>
<td>3.600</td>
<td>11.00</td>
<td>0.7</td>
<td>1.000</td>
<td>13.86</td>
<td>5.800</td>
</tr>
<tr>
<td>8c</td>
<td></td>
<td>1.0</td>
<td>3.600</td>
<td>11.00</td>
<td>5.31</td>
<td>1.000</td>
<td>210.28</td>
<td>5.2</td>
</tr>
<tr>
<td>8d</td>
<td>DRY</td>
<td>1.0</td>
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<td>3.156</td>
<td>1.800</td>
<td>224.96</td>
<td>5.2</td>
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<tr>
<td>TOTAL</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>449.10</td>
<td>2343.61</td>
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</table>

## L.L.SURCHARGE

<table>
<thead>
<tr>
<th>Element No</th>
<th>Component</th>
<th>Area Factor</th>
<th>Length (m)</th>
<th>Width (m)</th>
<th>Height (m)</th>
<th>Density (t/m³)</th>
<th>Weight (t)</th>
<th>Moment @ Toe (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>DRY EARTH</td>
<td>0.00</td>
<td>3.600</td>
<td>11.00</td>
<td>1.20</td>
<td>1.80</td>
<td>0.00</td>
<td>5.200</td>
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</tbody>
</table>

## SUMMARY OF FORCES AND MOMENTS:

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>Case. L.W.L.</th>
<th>Case. H.F.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service Cond.</td>
<td>Span dislodged</td>
<td>Service Cond.</td>
</tr>
<tr>
<td>Vertical load from superstructure including LL</td>
<td>334.81</td>
<td>0.00</td>
</tr>
<tr>
<td>Vertical load from substructure (b)</td>
<td>1161.45</td>
<td>1161.45</td>
</tr>
<tr>
<td>Total Vertical Load V = (a) + (b)</td>
<td>1496.27</td>
<td>1161.45</td>
</tr>
<tr>
<td>Total Horizontal Force H =</td>
<td>377.69</td>
<td>281.92</td>
</tr>
<tr>
<td>Moment @ toe due to (a)</td>
<td>937.48</td>
<td>0.00</td>
</tr>
<tr>
<td>Moment @ toe due to (b)</td>
<td>5088.84</td>
<td>5088.84</td>
</tr>
<tr>
<td>Total Moment @ toe (M)</td>
<td>6026.32</td>
<td>5088.84</td>
</tr>
<tr>
<td>Dist. of C.G. of V from toe Z = M/V</td>
<td>4.028</td>
<td>4.38</td>
</tr>
<tr>
<td>eccentricity (e = Z -b/2)</td>
<td>0.528</td>
<td>0.881</td>
</tr>
<tr>
<td>Relieving Moment @ c/l base (M1)</td>
<td>789.39</td>
<td>1023.76</td>
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<tr>
<td>overturning moment due to</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horz. braking force</td>
<td>141.48</td>
<td>0.00</td>
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<td>Earth Pressure</td>
<td>1520.38</td>
<td>1144.50</td>
</tr>
<tr>
<td>Total overturning Moment (M2)</td>
<td>1661.86</td>
<td>1144.50</td>
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<tr>
<td>Net moment (M2-M1) = M</td>
<td>872.47</td>
<td>120.74</td>
</tr>
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<td>Factor of Safety</td>
<td>3.63</td>
<td>4.45</td>
</tr>
<tr>
<td>Against overturning (M / M2)</td>
<td>0.528</td>
<td>0.881</td>
</tr>
<tr>
<td>Against sliding (μ x V / H)</td>
<td>1.981</td>
<td>2.060</td>
</tr>
<tr>
<td>Safe against overturning</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Safe against sliding</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

I.R.C 78-2000:cl 706.3.4
Area of base (A) = 7.000 \times 12.00 = 84.00 \text{ m}^2

Z_L = 98.00 \text{ m}

Z_T = 168.00 \text{ m}

CHECK FOR BASE PRESSURE:

<table>
<thead>
<tr>
<th>Base Pressure</th>
<th>LWL CASE</th>
<th>HFL CASE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Service Cond.</td>
<td>Span dislodged</td>
</tr>
<tr>
<td>P/A</td>
<td>17.81</td>
<td>13.83</td>
</tr>
<tr>
<td>M_L/Z_L</td>
<td>8.90</td>
<td>1.23</td>
</tr>
<tr>
<td>M_T/Z_T</td>
<td>1.15</td>
<td>0.00</td>
</tr>
<tr>
<td>(A) (P/A + M_L/Z_L + M_T/Z_T)</td>
<td>27.86</td>
<td>15.06</td>
</tr>
<tr>
<td>(B) (P/A + M_L/Z_L - M_T/Z_T)</td>
<td>25.57</td>
<td>15.06</td>
</tr>
<tr>
<td>(C) (P/A - M_L/Z_L + M_T/Z_T)</td>
<td>10.06</td>
<td>12.59</td>
</tr>
<tr>
<td>(D) (P/A - M_L/Z_L - M_T/Z_T)</td>
<td>7.762</td>
<td>12.59</td>
</tr>
</tbody>
</table>

Max. Base Pressure = 27.86 \text{ t/m}^2 < 32.00 \text{ Hence O.K.}

Min. Base Pressure = 3.15 \text{ t/m}^2 > 0 \text{ Hence O.K.}

DESIGN OF TOE SLAB
### BENDING MOMENT AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads</td>
<td>1</td>
<td>1.0</td>
<td>2.640</td>
<td>1.100</td>
<td>2.904</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>1.848</td>
<td>0.733</td>
<td>1.355</td>
</tr>
<tr>
<td>Upward Base pressure</td>
<td>Rect.</td>
<td>1.0</td>
<td>48.988</td>
<td>1.100</td>
<td>-53.886</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-6.156</td>
<td>1.467</td>
<td>-9.028</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>-50.655</td>
<td></td>
<td>-58.655</td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 58.655 t/m

Effective depth required 0.721 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 2859 mm²

Minimum Steel required 0.15% 1673 mm² I.R.C 78-2000 Clause:707.2.7

Distribution steel 0.06% 669 mm²/m

Mainsteel 2859 mm² 12 ø , @ 150 C/C 753.9822

Hence provide, 25 ø , @ 150 C/C

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum Steel reinforcement as per above clause 250 mm²/m provide 12 ø , @ 150 C/C 753.9822

### Check for Shear

### SHEAR FORCE AT "d" FROM FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward loads</td>
<td>1</td>
<td>1.0</td>
<td>1.302</td>
<td>0.543</td>
<td>0.706</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.5</td>
<td>0.911</td>
<td>0.362</td>
<td>0.330</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>Rect.</td>
<td>1.0</td>
<td>-27.237</td>
<td>0.543</td>
<td>-14.776</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-1.497</td>
<td>0.723</td>
<td>-1.083</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>-26.521</td>
<td></td>
<td>-14.823</td>
</tr>
</tbody>
</table>

Effective depth (d') at distance d 0.760 m

Shear force at critical section 26.5 t

Bending Moment at critical section 14.82 tm

\[ \tan \beta = 0.35 \]

Net shear force \( S \cdot M \cdot \tan \beta / d' \) 19.78 t

Hence, shear stress 26.02 t/m²

% of reinforcement 0.43

Permissible shear stress 29.33 t/m² Hence O.K.
### DESIGN OF HEEL SLAB

![Diagram of bridge design](image)

### BENDING MOMENT AND SHEAR FORCE AT FACE OF STEM

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Element</th>
<th>Area fact.</th>
<th>force</th>
<th>L.A.</th>
<th>moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward Loads.</td>
<td>3</td>
<td>1</td>
<td>4.320</td>
<td>1.800</td>
<td>7.776</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.5</td>
<td>3.024</td>
<td>1.200</td>
<td>3.629</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.5</td>
<td>4.536</td>
<td>2.400</td>
<td>10.886</td>
</tr>
<tr>
<td></td>
<td>8a</td>
<td>1</td>
<td>54.860</td>
<td>1.800</td>
<td>98.747</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5</td>
<td>4.536</td>
<td>2.400</td>
<td>5.443</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>6.852</td>
<td>1.800</td>
<td>12.333</td>
</tr>
<tr>
<td>Upward base pressure</td>
<td>Rect.</td>
<td>1</td>
<td>-27.944</td>
<td>1.800</td>
<td>-50.300</td>
</tr>
<tr>
<td></td>
<td>Trian.</td>
<td>0.5</td>
<td>-16.483</td>
<td>1.200</td>
<td>-19.779</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>33.700</td>
<td></td>
<td>68.736</td>
</tr>
</tbody>
</table>

Bending Moment at face of stem 68.736 tm

Effective depth required 0.781 m

Effective depth provided at face of stem 1.115 m

Area of Reinforcement required 3352.42 mm²

Minimum steel 0.15% 1672.50 mm² I.R.C 78-2000 Clause:707.2.7

Distribution steel 0.06% 669.00 mm³/m

Mainsteel 3352.42 mm²

Hence provide, 12 φ , @ 150 C/C 753.9822

There is no tension below foundation, hence foundation will not have negative moment at top. However in reference to clause 707.2.8 of IRC: 78-2000, the requirement of reinforcement at top is follows.

Minimum steel reinforcement as per above clause 250 mm³/m

Provide 12 φ , @ 150 C/C 753.9822
**Check for Shear**

(Critical section at face of stem)

Shear force at face of stem 33.70 t

\[ \tan \beta = 0.194 \]

Bending moment at face of stem 68.736 tm

Net shear force \( S \cdot M \cdot \tan \beta / d \) 21.71 t

Hence, Shear stress 19.47 t/m²

% of reinforcement 0.30

Permissible shear stress 25.10 t/m²  
Hence O.K.

**DESIGN OF STEM WALL**

![Diagram of stem wall](image)

**SUMMARY OF FORCES AND MOMENTS IN ABUTMENT SHAFT**

R.L. OF SECTION = 236.985 m

**DRY Condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element No.</th>
<th>Area factor</th>
<th>Height of E.P. diagram</th>
<th>Earth Pressure</th>
<th>Force</th>
<th>L.A.</th>
<th>Moment tm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>8.116</td>
<td>0.603</td>
<td>58.8</td>
<td>4.058</td>
<td>238.50</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>8.116</td>
<td>4.081</td>
<td>198.8</td>
<td>3.409</td>
<td>677.49</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>18.00</td>
<td>14.600</td>
<td>262.80</td>
<td></td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

= 197.338 + 18.20 + 10.37 + 334.81
= 560.716 t

Longitudinal Moment = 1178.787 t-m

Transverse Moment = 192.801 t-m

**b) Span Dislodged case**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

= 197.338 + 18.20 + 10.37 + 0.00
= 225.901 t

Longitudinal Moment = 915.987 t-m

Transverse Moment = 0.000 t-m
**H.F.L. condition**

**a) Service Condition**

<table>
<thead>
<tr>
<th>Element no.</th>
<th>Component</th>
<th>Area factor</th>
<th>Height (m)</th>
<th>Pressure (t/m²)</th>
<th>Force (t)</th>
<th>C.G. from base (m)</th>
<th>Moment (tm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LL Surcharge</td>
<td>1.0</td>
<td>1.200</td>
<td>0.603</td>
<td>61.31</td>
<td>4.058</td>
<td>248.79</td>
</tr>
<tr>
<td>2</td>
<td>Dry Earth</td>
<td>0.5</td>
<td>2.750</td>
<td>1.54</td>
<td>25.35</td>
<td>6.227</td>
<td>157.87</td>
</tr>
<tr>
<td>3</td>
<td>SubmgEarth</td>
<td>1.0</td>
<td>5.310</td>
<td>1.54</td>
<td>97.91</td>
<td>2.655</td>
<td>259.96</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.5</td>
<td>5.310</td>
<td>1.48</td>
<td>47.27</td>
<td>1.770</td>
<td>83.66</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>231.84</td>
<td>750.275</td>
</tr>
</tbody>
</table>

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

\[ = 120.874 + 18.20 + 10.37 + 334.81 \]

\[ = 484.252 \text{ t} \]

Longitudinal Moment = 1013.075 t·m

Transverse Moment = 192.801 t·m

**b) Span Dislodged case**

Total Vertical Load = Stem + dirt wall + cap + Load from superstructure

\[ = 120.874 + 18.20 + 10.37 + 0.00 \]

\[ = 149.437 \text{ t} \]

Longitudinal Moment = 750.275 t·m

Transverse Moment = 0.000 t·m

Cross Sectional area = 1.2 m²

For horizontal reinforcement area of steel required for the stem at the section/metre = 194.6531 mm²

Providing 12 @ 581.02 c/c say, 150 c/c as horizontal reinforcement
**Movement of Deck:**

Total Length of Bridge = 21.6 m

\[ \text{Size of bearing} = 500 \times 250 \times 50 \text{ mm} \]

\[ \text{Strain in bearing} = \frac{5.400}{50} = 0.108 \]

Shear modulus = 1.0 Mpa

Shear force per Bearing = 0.108 \times 1.0 \times 500 \times 250 = 13500 N

\[ \text{Total shear force} = \frac{13500}{1.376} \times 4 \times 1.05 = 5.780 \text{ t} \]

Refer IRC : 6 clause 214.5.1.5;
10 \% increase for variation in movement of span

Total shear force = \( 1.1 \times 5.780 = 6.358 \text{ t} \)

As per clause 214.2 of IRC:6, horizontal braking force \( F_h \) for each span is:

**For Class A Single lane** : \[ F_h = \left( 0.2 \times 55.4 \right) = 11.080 \text{ t} \]

**For class 70R wheeled** : \[ F_h = \left( 0.2 \times 100 \right) = 20.000 \text{ t} \]

**For class A 3 lane** : \[ F_h = 0.2 \times 55.4 + 0.05 \times 55.4 = 13.85 \text{ t} \]

**For class 70R wheeled +class A 1 lane** : \[ F_h = 0.2 \times 100 + 0.05 \times 55.4 = 22.77 \text{ t} \]

**For class A 3 lane**

Longitudinal horizontal force = \( \frac{13.850 + 6.358}{2} = 13.28 \text{ t} \) Say, 14.0 t

**For class 70R wheeled +class A 1 lane**

Longitudinal horizontal force = \( \frac{22.770 + 6.358}{2} = 17.7 \text{ t} \) Say, 18.0 t

**Span dislodged condition**

Longitudinal horizontal force = \( \frac{0.000 + 6.358}{2} = 6.358 \text{ t} \) Say, 7.0 t

**Summary of Longitudinal Forces**:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Longitudinal horizontal force (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A 3 lane</td>
<td>14.00</td>
</tr>
<tr>
<td>70R+class A 1 lane</td>
<td>18.00</td>
</tr>
<tr>
<td>Span dislodged condition</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Transverse eccentricity**

Class 70RW+ Class A

\[ e = 1.6927 \]
**Class A 3 lane**

\[
\begin{array}{c|c|c}
5.3 & 0.7 & e = 0.7 \\
\end{array}
\]

**Summary of Dead load & Live loads from Superstructure. (STAAD Pro)**

<table>
<thead>
<tr>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load Reaction:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SIDL</td>
<td>184.71</td>
<td></td>
</tr>
<tr>
<td>L.L  Max Reaction Case</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70R + Class A</td>
<td>113.9</td>
<td>192.80</td>
</tr>
<tr>
<td>Class A 3 Lane</td>
<td>110.61</td>
<td>77.43</td>
</tr>
<tr>
<td></td>
<td>334.81</td>
<td>192.80</td>
</tr>
</tbody>
</table>
Summary of Loads at Abutment Shaft bottom:

<table>
<thead>
<tr>
<th>1 DRY condition</th>
<th>P</th>
<th>M_L</th>
<th>M_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>With L.L</td>
<td>560.72</td>
<td>1178.79</td>
<td>192.80</td>
</tr>
</tbody>
</table>

Check for Cracked/Uncracked Section

Length of section = 12000 mm
Width of section = 1200 mm
Gross Area of section \( A_g \) = 14400000 mm²
Gross M.O.I of section \( I_{gxx} \) = 1.728E12 mm⁴
Gross M.O.I of section \( I_{gyy} \) = 1.728E14 mm⁴

Transformed sectional properties of section:

Adopting

- Modular ratio \( m \) = 10
- Cover = 70
- Dia of Bars = 20
- No of bars in tension face (longer) = 180
- No of bars in compression face = 90
- No of bars in shorter direction = 8
- Total bars in section = 286

% Reinforcement provided = 77861.232 mm²
Steel Area \( A_s = 77861 \text{ mm}^2 \)
% of Steel \( = 0.5407 \% \)

\[
\begin{align*}
A_{sx} &= 56549 \text{ mm}^2 \\
A_{sy} &= 1608.5 \text{ mm}^2 \\
\text{Area of concrete} \quad A_c &= A_g - A_s = 14322139 \text{ mm}^2 \\
\text{C.G of Steel placed on longer face} &= 530 \text{ mm} \\
\text{C.G of Steel placed on shorter face} &= 5932 \text{ mm} \\
\text{Transformed Area of Section} \quad A_{\text{fin}} &= 15100751 \text{ mm}^2 \\
\text{Transformed M.I}_{xx} &= I_{gxx} + 2 \left[ m - \frac{1}{2} \right] A_c ax^2 \\
&= 2.01392E+12 \text{ mm}^4 \\
Z_{xx} &= \frac{M.I_{xx}}{d/2} = 3.357E+09 \text{ mm}^3 \\
\text{Transformed M.I}_{yy} &= I_{gyy} + 2 \left[ m - \frac{1}{2} \right] A_c ay^2 \\
&= 1.73819E+14 \text{ mm}^4 \\
Z_{yy} &= \frac{M.I_{yy}}{d/2} = 2.897E+10 \text{ mm}^3 \\
\end{align*}
\]

**Permissible stresses**

Minimum Gross Moment of inertia \( I_{\text{min}} = 1.728E+12 \text{ mm}^4 \)
Area of section \( = 14400000 \text{ mm}^2 \)
r \( = 346.41016 \text{ mm} \)

**Effective length of Abutment shaft** (IRC:21-2000 cl: 306.2.1)

Abutment shaft height \( L = 9.426 \text{ m} \)
Effective length \( I_{\text{eff}} = 11.311 \text{ m} \)
Slenderness ratio \( = 32.653 < 50 \)
Type of member \( = 1 \)

**Stress reduction coefficient** (IRC:21-2000 cl: 306.4.2,3)
\( \beta = 1 \)

**Permissible stresses**

- **concrete**
  \( \sigma_{\text{cbc}} = 8.3333 \text{ N/mm}^2 \)
  \( \sigma_{\text{co}} = 6.25 \text{ N/mm}^2 \)
  Tensile stress \( = 0.67 \text{ N/mm}^2 \)

- **Steel**
  \( \sigma_{st} = 200 \text{ N/mm}^2 \)
<table>
<thead>
<tr>
<th>S.No</th>
<th>Item</th>
<th>DRY Case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Loads and Moments</strong></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>P</td>
<td>560.72 t</td>
</tr>
<tr>
<td>2</td>
<td>M_l</td>
<td>1178.79 t-m</td>
</tr>
<tr>
<td>3</td>
<td>M_T</td>
<td>192.80 t-m</td>
</tr>
</tbody>
</table>

**Actual(calculated) Stresses**

<table>
<thead>
<tr>
<th></th>
<th>S_{co,cal} P/\Lambda_{thi}</th>
<th>0.37131642</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>S_{cbc,cal} M_l/Z_{xx}</td>
<td>3.511914483</td>
</tr>
<tr>
<td>6</td>
<td>S_{cbc,cal} M_T/Z_{yy}</td>
<td>0.066552525</td>
</tr>
<tr>
<td>7</td>
<td>S_{cbc,cal} = 5 + 6</td>
<td>3.578467008</td>
</tr>
</tbody>
</table>

**Permissible Stresses**

<table>
<thead>
<tr>
<th></th>
<th>S_{cbc}</th>
<th>8.3333333</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>S_{co}</td>
<td>6.25</td>
</tr>
</tbody>
</table>

**Check for Minimum steel area mm²**

<table>
<thead>
<tr>
<th></th>
<th>Conc.Area Required for directstress</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>(1)/(9) 897145.09</td>
</tr>
<tr>
<td>11</td>
<td>0.8% of area required</td>
</tr>
<tr>
<td>12</td>
<td>0.3% of A_g</td>
</tr>
<tr>
<td>13</td>
<td>Governing steel mm²</td>
</tr>
<tr>
<td>14</td>
<td>Provided Steel area mm²</td>
</tr>
</tbody>
</table>

**Check for safety of section**

\[
\frac{S_{co,cal}}{S_{co}} + \frac{S_{cbc,cal}}{S_{cbc}} < 1
\]

\[
0.4888267 + 3.207151 < 1
\]

**Check for Cracked /Uncracked section**

\[
S_{co,cal} - S_{cbc,cal} = -3.207151
\]

Permissible Basic tensile stress in concrete -0.67

Section to be designed as **Cracked**
Width of Solid return wall (a) = 3.60
Width of Cantilever return wall = 4.00
Avg Height of Solid return wall (b) = 8.816
Height of Cantilever return at Tip = 0.75
Height of Cantilever return at Root = 2.666667
Thickness of Solid Return at farther end = 0.5
Thickness of Solid Return at Root = 0.5
Thickness of Solid Return at bottom = 0.5
Thickness of Solid Return at top = 0.5
Thickness of Cantilever return = 0.5
Unit wt of Soil = 1.8 t/m³
Grade of concrete = M30

\[
\sigma_{ebc} = \frac{1020}{\text{t/m}^2}
\]

\[
m = 10
\]

\[
\sigma_{st} = \frac{20400}{\text{t/m}^2}
\]

\[
k = 0.333333
\]

\[
j = 0.888889
\]

\[
R = 151.1111 \text{ t/m}^2
\]

Case (1) For uniformly distributed load over entire plate

\[
a/b = 0.4083485 \\
\text{For } a/b = 0.375 \quad \beta_1 = 0.353 \quad \beta_2 = 0.398
\]

\[
\text{For } a/b = 0.5 \quad \beta_1 = 0.631 \quad \beta_2 = 0.632
\]

\[
a/b = 0.4083485 \\
\beta_1 = 0.427167
\]

\[
\beta_2 = 0.460428
\]
Live Load Surcharge:
\[ q = 0.2794 \times 1.8 \times 1.2 = 0.603504 \text{ t/m}^2 \]
\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]
\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]
\[ \sigma_{b_{\text{max}}} = \frac{0.427167 \times 0.603504 \times 77.72}{0.25} = 80.14584 \text{ t/m}^2 \]
For 1000 mm of width
\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]
\[ = 0.041667 \text{ m}^3 \]

Hence Moment /m width along Y direction
\[ M_Y /m \text{ width} = 80.145837 \times 0.041667 = 3.33941 \text{ t-m/m} \]
\[ \sigma_{a_{\text{max}}} = \frac{0.4604283 \times 0.603504 \times 77.72}{0.25} = 86.38639 \text{ t/m}^2 \]
For 1000 mm of width
\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]
\[ = 0.041667 \text{ m}^3 \]

Hence Moment /m width along X direction
\[ M_X /m \text{ width} = 86.386391 \times 0.041667 = 3.599433 \text{ t-m/m} \]

Case (2) For Triangular loading due to earth pressure
\[ \frac{a}{b} = 0.4083485 \quad \text{For } \frac{a}{b} = 0.375 \quad \beta_1 = 0.212 \quad \beta_2 = 0.148 \]
\[ \text{For } \frac{a}{b} = 0.5 \quad \beta_1 = 0.328 \quad \beta_2 = 0.200 \]
\[ \frac{a}{b} = 0.4083485 \quad \beta_1 = 0.242947 \]
\[ \beta_2 = 0.161873 \]

Earth pressure:
\[ q = 0.2794 \times 1.8 \times 8.816 = 4.433743 \text{ t/m}^2 \]
\[ \sigma_{b_{\text{max}}} = \frac{\beta_1 \times q \times b^2}{t^2} \]
\[ \sigma_{a_{\text{max}}} = \frac{\beta_2 \times q \times b^2}{t^2} \]
\[ \sigma_{b_{\text{max}}} = \frac{0.2429474 \times 4.433743 \times 77.72}{0.25} = 334.8774 \text{ t/m}^2 \]
For 1000 mm of width
\[ Z = \frac{1000 \times 250000}{6} = 41666667 \text{ mm}^3 \]
\[ = 0.041667 \text{ m}^3 \]
Hence Moment /m width along Y direction

\[ M_Y \text{/m width} = 334.8774 \times \frac{0.041667}{0.25} = 13.95323 \text{ t-m/m} \]

\[ \sigma_{\text{max}} = \frac{0.161873 \times 4.433743 \times 77.72}{6} = 223.1249 \text{ t/m}^2 \]

For 1000 mm of width

\[ Z = \frac{1000 \times 250000}{6} = 4166667 \text{ mm}^3 \]

\[ = 0.04167 \text{ m}^3 \]

Hence Moment /m width along X direction

\[ M_X \text{/m width} = 223.12485 \times \frac{0.041667}{0.25} = 9.296869 \text{ t-m/m} \]

Total Moment in Solid Return /m height

\[ = 12.8963 \text{ t-m/m} \]

Along X-direction

Total Moment in Solid Return /m width

\[ = 17.29263 \text{ t-m/m} \]

Along Y-direction

**Moment due to Cantilever Return:**

**Moment due to earth pressure at face A - A**

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 2.00 + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 2.00 + 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times X^2 \times dx \times \left[ \frac{4.00 - X}{4.00 - X} \right] + 0.2794 \times 1.8 \times 1.95 \times 0.666667 \times X \times dx \times \left[ \frac{4.00 - X}{4.00 - X} \right] \]

\[ = 3.621024 + 1.13157 + 0.11176 \times 21.33333 + 0.653796 \times 10.66667 \]

\[ = 14.11063 \text{ t-m} \]

**Design of cantilever Return:**

Assuming 50 mm cover and 12 mm dia bars.

Effective depth available = 500 - 50 - 20 - 6 = 424 mm

\[ M = R \times b \times d^2 \]

\[ = 151.1111 \times 2.666667 \times 0.179776 = 72.44307 \text{ t-m} \]

\[ A_{st} = \frac{14.11063 \times 10^6}{20400 \times 0.888889 \times 0.424} = 1835.283 \text{ mm}^2 \]

\[ A_{st/m} = 688.231 \text{ mm}^2/m \]

Provide 12 mm dia @ 150 mm c/c providing 753.9822 mm$^2$ on earth face.

Provide 12 mm dia @ 180 mm c/c providing 628.3185 mm$^2$ on other face.

**Along Horizontal direction.**
Design of Solid Return:

Moment due to Cantilever Return:

\[ M = 0.2794 \times 1.2 \times 1.8 \times 0.75 \times 4.00 \times 5.60 + 0.5 \times 0.2794 \times 1.8 \times 0.5625 \times 4.00 \times 5.60 + 0.5 \times 0.2794 \times 1.8 \times 0.44444 \times X^2 \times dx \times \left[ 7.60 - X \right] + 0.2794 \times 1.8 \times 1.95 \times 0.66667 \times X \times dx \times \left[ 7.60 - X \right] \]

\[ = 10.1387 + 3.168396 + 0.11176 \times 98.1333 + 0.653796 \times 39.4667 \]

\[ = 50.07779 \text{ t-m} \]

Moment in Solid Return /m height = 12.8963 + \frac{50.07779}{8.816} = 18.57663 \text{ t-m/m}

Moment in Solid Return /m width = 17.29263 \text{ t-m/m}

Design of face B-B:

Moment in Solid Return /m height = 18.57663 \text{ t-m/m}

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 20 - 10 = 420 mm

\[ M = R \times b \times d^2 \]
\[ = 151.1111 \times 1 \times 0.1764 = 26.656 \text{ t-m} \]

\[ A_{st} = \frac{18.57663 \times 10^6}{20400 \times 0.888889 \times 0.42} = 2439.159 \text{ mm}^2 \]

\[ A_{sd}/m = 2439.159 \text{ mm}^2/m \]

Provide 20 mm dia @ 120 mm c/c providing 2617.994 mm$^2$ on earth face.

Provide 12 mm dia @ 120 mm c/c providing 942.4778 mm$^2$ on other face.

Along Horizontal direction.

Design of face A-B:

Moment in Solid Return /m width = 17.29263 \text{ t-m/m}

Assuming 50 mm cover and 20 mm dia bars.

Effective depth available = 500 - 50 - 0 - 10 = 440 mm

\[ M = R \times b \times d^2 \]
\[ = 151.1111 \times 1 \times 0.1936 = 29.25511 \text{ t-m} \]

\[ A_{st} = \frac{17.29263 \times 10^6}{20400 \times 0.888889 \times 0.44} = 2167.359 \text{ mm}^2 \]

\[ A_{sd}/m = 2167.359 \text{ mm}^2/m \]

Provide 20 mm dia @ 130 mm c/c providing 2416.61 mm$^2$ on earth face.

Provide 12 mm dia @ 130 mm c/c providing 869.9795 mm$^2$ on other face.

Along Vertical direction.
1. **DESIGN FOR FORCES IN LONGITUDINAL DIRECTION**

Active earth pressure \( K_a = 0.279384 \)

FOR NORMAL CASE, \( FA = 1.0 \)

unit wt of soil \( \gamma = 1.8 \)

\( \delta = 20 \)

clear span between return wall = 11.000 m

width of return wall = 0.5

Avg. cover to reinforcement. (FOR 2-3 LAYERS) = 0.150 m

H (From formation level) = 8.466 m

**DESIGN FOR BENDING MOMENT**

<table>
<thead>
<tr>
<th>S.NO.</th>
<th>UNITS</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (FROM TOP)</td>
<td>m</td>
<td>8.466</td>
</tr>
<tr>
<td>WIDTH OF RETURN AT THIS LEVEL</td>
<td>m</td>
<td>3.600</td>
</tr>
<tr>
<td>FORCE DUE TO EARTH PRESSURE</td>
<td>t</td>
<td>101.610</td>
</tr>
<tr>
<td>MOMENT DUE TO EARTH PRESSURE</td>
<td>tm</td>
<td>361.297</td>
</tr>
<tr>
<td>FORCE DUE TO L.L. SURCHARGE</td>
<td>t</td>
<td>28.805</td>
</tr>
<tr>
<td>MOMENT DUE TO L.L. SURCHARGE</td>
<td>tm</td>
<td>121.932</td>
</tr>
<tr>
<td>TOTAL MOMENT</td>
<td>tm</td>
<td>483.229</td>
</tr>
<tr>
<td>DESIGN MOMENT</td>
<td>tm</td>
<td>483.229</td>
</tr>
<tr>
<td>REQUIRED EFFECTIVE DEPTH</td>
<td>m</td>
<td>2.828</td>
</tr>
<tr>
<td>EFF. DEPTH AVAILABLE</td>
<td>m</td>
<td>3.450</td>
</tr>
<tr>
<td>AREA OF STEEL REQUIRED</td>
<td>cm(^2)</td>
<td>77.233</td>
</tr>
<tr>
<td>DIAMETER OF BAR PROVIDED</td>
<td>mm</td>
<td>32</td>
</tr>
<tr>
<td>TOTAL NO. OF BARS</td>
<td>no.</td>
<td>12</td>
</tr>
<tr>
<td>AREA OF STEEL PROVIDED</td>
<td>cm(^2)</td>
<td>96.51</td>
</tr>
</tbody>
</table>

**CHECK FOR SHEAR STRESS**

| SHEAR FORCE | t | 135.415 |
| SHEAR STRESS | t/m\(^2\) | 78.502 |
| Ast/bd \( \times \) 100 | % | 0.448 |
| PERMISSIBLE SHEAR STRESS | MPa | 0.29 |
| PERMISSIBLE SHEAR STRESS | t/m\(^2\) | 29.91 |
| Asv/sv | cm\(^2\)/m | 6.904 |
**Design of Abutment Cap:**

As the cap is fully supported on the abutment. Minimum thickness of the cap required as per cl: 710.8.2 of IRC:78-2000 is 200 mm.

However the thickness of abutment cap is \(= 300\) mm

Assuming a cap thickness of \(= 300\) mm

Volume of Abutment cap \(= 0.3 \times 1.20 \times 12 = 4.32\) m\(^3\)

Quantity of steel \(= 1\%\) of volume

\[\frac{1}{100} \times 4.32 = 0.0432\) m\(^3\)

Quantity of steel to be provided at top \(= 0.0216\) m\(^3\)

Quantity of steel to be provided at bottom \(= 0.0216\) m\(^3\)

**Top & bottom face:**

Quantity of steel to be provided in Longitudinal direction \(= 0.0108\) m\(^3\)

Assuming a clear cover of \(= 50\) mm

Length of bar \(= 12.00 - 0.100 = 11.9\) m

Area of steel required in Longitudinal direction

\[\frac{0.0108}{11.9} = 907.563\) mm\(^2\)

Provide \(9\) nos of bars \(12\) mm dia at top & bottom face.

\(= 1017.88\) mm\(^2\)

**Transverse steel:**

Quantity of steel to be provided in Longitudinal direction \(= 0.0108\) m\(^3\)

Assuming a clear cover of \(= 50\) mm

Assuming a dia of bar \(= 12\) mm

Length of bar \(= 1.20 - 0.100 = 1.1\) m

Volume of each stirrup \(= 0.00012\) m\(^3\)

\(\text{no of stirrups required for m/length} = \frac{8}{1000} = 0.0012\)

\(\text{Required Spacing} = \frac{8}{125} = 0.064\) mm

Provide \(12\) mm dia bar \(125\) mm c/c stirrups through in length of abutment cap.

\(904.779\) mm\(^2\)
Design of Dirt wall:
Dirt wall designed as a vertical cantilever.

Intensity for rectangular portion = \(0.2794 \times 2.00 \times 1.2\) = 0.67056 \(\text{t/m}^2\)

\[F_1 = 0.67056 \times 12.00 \times 2.11 = 16.94639 \quad \text{t}\]

Intensity for triangular portion = \(0.2794 \times 2.00 \times 2.11\) = 1.176833 \(\text{t/m}^2\)

\[F_2 = 1.176833 \times 12.00 \times 2.11 = 14.87046 \quad \text{t}\]

\[M_1 = 16.94639 \times 1.05 = 17.84455 \quad \text{t-m}\]

\[M_2 = 14.87046 \times 0.88452 = 13.15322 \quad \text{t-m}\]

\[M_{1+2} = 30.99777 \quad \text{t-m}\]

Total moment at base of dirt wall /m length = 2.583147 \(\text{t-m/m}\)

Thickness of dirtwall = 0.3 m

Assuming a clear cover on either face = 50 mm

**Vertical steel on earth face:**

dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 6 = 244 mm

effective depth req = \(2.583147 \times 1.51 \times 1000\) = 130.7935 mm

Ast req = \(2.583147 \times 200 \times 0.889 \times 244\) = 595.4258 mm\(^2\)/m

Minimum steel = \(0.12 \times 300 \times 1000\) = 360 mm\(^2\)/m

Provide 12 mm dia bar 200 mm c/c as vertical steel at earth face.

565.4867 mm\(^2\)/m

**Distribution steel on earth face:**

dia of steel bar = 12 mm

Available effective depth = 300 - 50 - 12 = 238 mm

0.3M = 0.3 \(\times 2.583147\) = 0.774944 \(\text{t-m/m}\)

Ast req = \(0.774944 \times 200 \times 0.889 \times 238\) = 183.1309 mm\(^2\)/m

Minimum steel as per IRC:21-200 cl:305.10 = 250 mm\(^2\)/m

Governing steel at earth face = 250 mm\(^2\)/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm\(^2\)/m

**Vertical steel on earth face**

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm\(^2\)/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm\(^2\)/m

**Distribution steel:**

As per IRC:21-200 cl:305.10 All faces provide minimum steel of = 250 mm\(^2\)/m

Provide 10 mm dia bar 200 mm c/c as vertical steel at earth face.

392.6991 mm\(^2\)/m
INPUT FILE: 70RW.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
7. 1 0.00 0 0; 2 0.3 0 0; 3 21.3 0 0; 4 21.6 0 0
8. MEMBER INCIDENCES
9. 1 2 3
10. MEMBER PROPERTY CANADIAN
11. 1 TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 8.0 2*12 4*17.0 DIS 3.96 1.52 2.13 1.37 3.05 1.37
20. LOAD GENERATION 175
21. TYPE 1 -13.4 0.0 0.0 XINC .2
22. PERFORM ANALYSIS
23. LOAD LIST 109
24. PRINT SUPPORT REACTION
SUPPORT REACTION
SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE
--------------------------
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM Z

2 109 0.00 22.97 0.00 0.00 0.00 0.00
3 109 0.00 77.03 0.00 0.00 0.00 0.00

*************** END OF LATEST ANALYSIS RESULT ***************

25. FINISH

*********** END OF THE STAAD.Pro RUN ***********
INPUT FILE: CLASS A.STD
1. STAAD SPACE
2. INPUT WIDTH 72
3. UNIT METER MTON
4. PAGE LENGTH 1000
5. UNIT METER MTON
6. JOINT COORDINATES
   7. 1 0.00 0 0; 2 0.3 0 0; 3 21.3 0 0; 4 21.6 0 0
8. MEMBER INCIDENCES
   9. 1 1 2 3
10. MEMBER PROPERTY CANADIAN
11. 1 TO 3 PRI YD 1.0 ZD 1.0
12. CONSTANT
13. E CONCRETE ALL
14. DENSITY CONCRETE ALL
15. POISSON CONCRETE ALL
16. SUPPORT
17. 2 3 PINNED
18. DEFINE MOVING LOAD
19. TYPE 1 LOAD 2*2.7 2*11.4 4*6.8 DIS 1.1 3.2 1.2 4.3 3 3 3
20. TYPE 2 LOAD 4*6.8 2*11.4 2*2.7 DIS 3 3 3 4.3 1.2 3.2 1.1
21. *LOAD GENERATION 202
22. *TYPE 1 -18.8 0. 0. XINC .2
23. LOAD GENERATION 202
24. TYPE 2 -18.8 0. 0. XINC .2
25. PERFORM ANALYSIS
26. LOAD LIST 130
27. PRINT SUPPORT REACTION
28. SUPPORT REACTION

SUPPORT REACTIONS -UNIT MTON METE STRUCTURE TYPE = SPACE

---------------------
JOINT LOAD FORCE-X FORCE-Y FORCE-Z MOM-X MOM-Y MOM Z
  2 130 0.00 13.13 0.00 0.00 0.00 0.00
  3 130 0.00 36.87 0.00 0.00 0.00 0.00

28. FINISH
Abutment SHAFT ..DRY NORMAL

Depth of Section = 1.200 m
Width of Section = 12.000 m

along width-compression face- no of bar: 90 tension face- no of bar: 180
Dia (mm) 16 20
Cover (cm) 7.50 10.5

along depth-compression face- no of bar: 8 tension face- no of bar: 8
Dia (mm) 16 16
Cover (cm) 7.50 7.5

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 560.716 T
Mxx = 1178.787 Tm
Myy = 192.801 Tm

Intercept of Neutral axis : X axis : = 214.704 m
: y axis : = .342 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 71.67 Kg/cm^2
Stress in Steel due to Loads = 1614.66 Kg/cm^2
Percentage of Steel = .54 %

Abutment SHAFT ..DRY SPAN DISLODGED

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 225.901 T
Mxx = 915.987 Tm
Myy = .000 Tm

Intercept of Neutral axis : X axis : = ****** m
: y axis : = .302 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 53.55 Kg/cm^2
Stress in Steel due to Loads = 1404.83 Kg/cm^2
Percentage of Steel = .54 %

Abutment SHAFT ..HFL NORMAL

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 484.252 T
Mxx = 1013.075 Tm
Myy = 192.801 Tm

Intercept of Neutral axis : X axis : = 185.513 m
:y axis : = .344 m

Concrete Stress Governs Design

Stress in Concrete due to Loads = 61.88 Kg/cm^2
Stress in Steel due to Loads = 1388.68 Kg/cm^2
Percentage of Steel = .54 %

Abutment SHAFT ..HFL SPAN DISLODGED

Depth of Section = 1.200 m
Width of Section = 12.000 m

Modular Ratio : Compression = 10.0
Modular Ratio : Tension = 10.0
Allowable Concrete Stress = 85.00 Kg/cm^2
Allowable Steel Stress = 2040.00 Kg/cm^2

Axial Load = 149.437 T
Mxx = 750.275 Tm
Myy = .000 Tm

Intercept of Neutral axis : X axis : = ****** m
:y axis : = .296 m

Steel Stress Governs Design

Stress in Concrete due to Loads = 43.71 Kg/cm^2
Stress in Steel due to Loads = 1179.01 Kg/cm^2
Percentage of Steel = .54 %
DESIGN OF SUPERSTRUCTURE
For Design of Superstructure of RCC Girder 21.0 m span refer MOST STANDARD Drawing titled “STANDARD PLANS FOR HIGHWAY BRIDGES (R.C.C T-beam and Slab Superstructure)” Drg. No. SD/220 to SD/226.