



NCR Planning Board  
Asian Development Bank

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

FINAL REPORT

Volume V-A2: DPR for Flyover at Mohan Nagar Junction in  
Ghaziabad

Detailed Designs

July 2010

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## **Contents**

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

## **Appendix D-1: Super Structure Design: Deck Slab**

### **Deck slab design**

#### **General**

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

#### **Live load calculation**

	unit in meters
Total width =	8.5
Cantilever length	0.8
C/C of main beams( $l_0$ ) =	2.2
C/Cof Cross girders( $b$ ) =	10

#### **Effective width of dispersion**

$$\text{Effective width} = k \cdot a \cdot (1 - a/l_0) + b_1$$

$k$  depends on  $b/l_0$  ratio

$a$  = distance of the load from the nearest support

$$b/l_0 = 4.55$$

$$k = 2.6 \text{ Refer cl. 305.16.2 IRC 1-2000}$$

#### **LOADING - CLASS A WHEELED - minimum distance from the ker** **Impact factor**

Refer clause 211.2 IRC 6-2000

$$\text{Impact factor} = \frac{1 + 4.5/(6+L)}{1.5} = 1.55$$

Tyre contact dimensions = 0.5 x 0.25

$$b_1 = \text{Dispersion upto the top of the slab} \\ = 0.25 + 2 \cdot 0.065 = 0.38 \text{ m}$$

#### **Effective dispersion along the span**

$$\text{Dispersion upto the bottom of the deck slab} \\ = \text{wheel dim. along span} + 2 \cdot (0.065 + 0.24) \\ = 1.11 \text{ m}$$

Maximum wheel load =	57 kN
including impact =	85.5 kN
Effective width for L1	
$a$ =	0.1 m
$b_{eff1}$ =	0.63 m
L1/contact area =	
(incl. Impact)	122.62 (kN/m <sup>2</sup> )
Effective width for L2	
$a$ =	0.3 m
$b_{eff}$ =	1.05 m
L2/contact area =	
(incl. Impact)	73.11 (kN/m <sup>2</sup> )

Effective width for L3  
 $a = 0.8 \text{ m}$   
 $b_{eff} = 1.70 \text{ m}$

L2/contact area =  
(incl. Impact)  $45.21 \text{ (kN/m}^2\text{)}$

Effective width for L4  
 $a = 1 \text{ m}$   
 $b_{eff1} = 1.798 \text{ m}$   
L2/contact area =  $85.5/(1.092*0.362)$   
(incl. Impact)  $42.84 \text{ (kN/m}^2\text{)}$

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

Refer clause 211.3 IRC 6-2000

Impact factor = 1.25

tyre contact dimensions .36 x .263

Dispersion perpendicular to span=  $0.263+2*.075$   
0.413 m

Dispersion along span =  $0.36+2*(0.075+0.24)$   
= 0.99 m

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

**Effective width of dispersion**

For L1  
 $a = 0.72 \text{ m}$   
 $b_{eff1} = 1.67 \text{ m}$   
L1/ contact area =  $64.18 \text{ kN/m}^2$   
(Including impact)

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

L2/ contact area =  $58.69 \text{ kN/m}^2$   
(Including impact)

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

For L1 And L2

$a = 0.9 \text{ m}$	L3
$b_{eff} = 1.76 \text{ m}$	0.4
	1.25

L/contact area =  $43.70 \text{ kN/m}^2$   
(including impact)  $60.49 \text{ kN/m}^2$

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

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

Load/contact area =  $58.92 \text{ kN/m}^2$   
(including impact)

**LOADING - CLASS A WHEEL****Impact factor**

Refer clause 211.2 IRC 6-2000

$$\text{Impact factor} = \frac{1+4.5/(6+L)}{1.5} = 1.55$$

$$\begin{aligned} \text{Tyre contact dimensions} &= 0.5 \times 0.25 \\ b_1 &= \text{Dispersion upto the top of the slab } (0.25+2*0.075) \\ &= 0.4 \text{ m} \end{aligned}$$

**Effective dispersion along the span**

$$\begin{aligned} \text{Dispersion upto the bottom of the deck slab} \\ &= \text{wheel dim. along span} + 2*(0.75+0.2) \\ &= 1.13 \text{ m} \end{aligned}$$

**Maximum load at mid span**

$$\begin{aligned} \text{Maximum wheel load} &= 57 \text{ kN} \\ \text{including impact} &= 85.5 \text{ kN} \\ \text{Effective width for L1} \\ a &= 1.1 \text{ m} \\ b_{eff1} &= 1.83 \text{ m} \\ \text{L1/contact area} &= \\ \text{(incl. Impact)} &= 41.35 \text{ (kN/m}^2\text{)} \\ \\ \text{Effective width for L2} \\ a &= 0.7 \text{ m} \\ b_{eff} &= 1.64 \text{ m} \\ \\ \text{L2/contact area} &= \\ \text{(incl. Impact)} &= 46.11 \text{ (kN/m}^2\text{)} \end{aligned}$$

**LOADING - CLASS 70R WHEEL****Maximum load at mid span**

Refer clause 211.3 IRC 6-2000

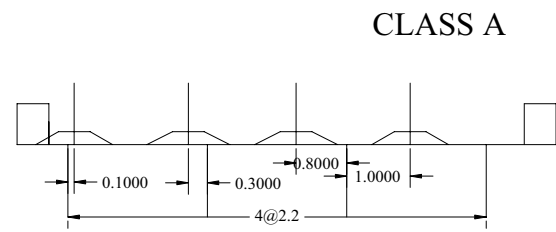
$$\text{Impact factor} = 1.25$$

$$\begin{aligned} \text{tyre contact dimensions} &= .36 \times .263 \\ \text{Dispersion perpendicular to span} &= 0.263+2*.075 \\ &= 0.413 \text{ m} \\ \\ \text{Dispersion along span} &= 0.36+2*(0.075+0.24) \\ &= 0.99 \text{ m} \end{aligned}$$

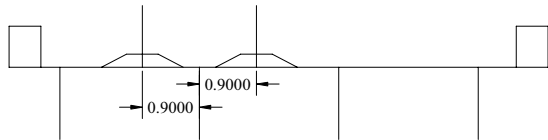
$$\begin{aligned} \text{Maximum wheel load} &= 85 \text{ kN} \\ \text{Load with impact} &= 106.25 \text{ kN} \end{aligned}$$

**Effective width of dispersion**

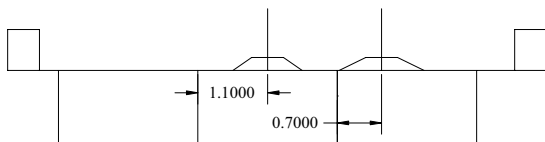
$$\begin{aligned} \text{For L1} \\ a &= 1.1 \text{ m} \\ b_{eff1} &= 1.84 \text{ m} \\ \text{L1/ contact area} &= 58.23 \text{ kN/m}^2 \\ \text{(Including impact)} & \\ \\ \text{For L2} \\ a &= 0.83 \text{ m} \\ b_{eff2} &= 1.76 \text{ m} \\ \\ \text{L2/ contact area} &= 61.09 \text{ kN/m}^2 \\ \text{(Including impact)} & \end{aligned}$$



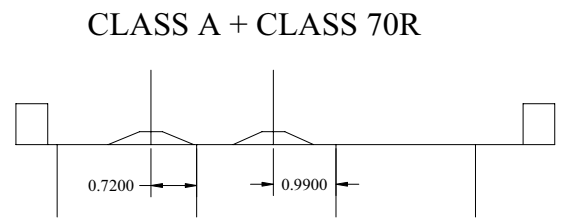
MINIMUM CLEARANCE FROM KERB END



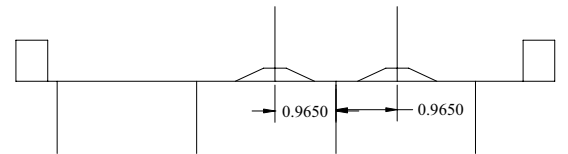
MAXIMUM LOADS EQUIDISTANT FROM SUPPORT



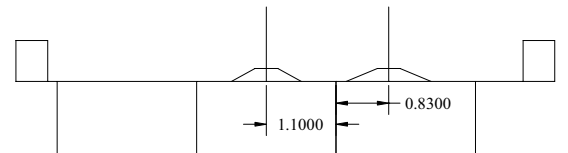
MAXIMUM LOAD AT MID SPAN



MINIMUM CLEARANCE FROM KERB END



MAXIMUM LOADS EQUIDISTANT FROM SUPPORT



MAXIMUM LOAD AT MID SPAN

**FIG. 2 LOAD ARRANGEMENT FOR TRANSVERSE ANALYSIS**

**GIRDER SPACING - 2.2m**



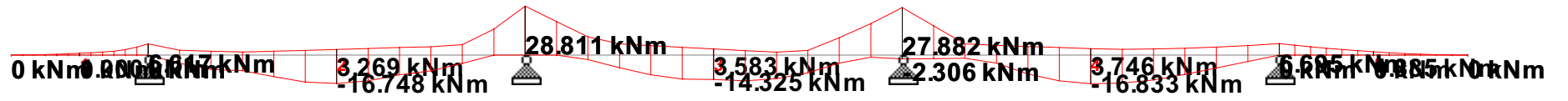
## STAAD INPUT

```
STAAD PLANE TRANSVERSE ANALYSIS OF DECK
UNIT METER KNS
PAGE LENGTH 100
JOINT COORDINATES
1 0 0 0; 2 0.8 0 0; 3 3.0 0 0; 4 5.2 0 0; 5 7.4 0 0
6 8.5 0 0;
MEMBER INCIDENCES
1 1 2 5
MEMBER PROPERTIES
1 TO 5 PRI YD 0.25 ZD 1
CONSTANTS
E CONCRETE ALL
POISSON CONCRETE ALL
SUPPORT
2 3 4 5 PINNED
LOAD 1 DL
MEM LOAD
1 TO 5 UNI GY -6.25
LOAD 2 SIDL
MEM LOAD
CRASH BARRIER
member load
1 UNI GY -6.25 0 0.5
5 UNI GY -6.25 0.6
WEARING COAT
member load
1 UNI GY -1.43 0.5
5 UNI GY -1.43 0 0.6
2 3 4 UNI GY -1.43
LOAD 3 LIVELOAD (CLASS A SINGLE LANE)
MEM LOAD
1 UNI GY -122.62 0.585
2 UNI GY -122.62 0.0 0.415
2 UNI GY -73.11 1.395
3 UNI GY -73.11 0 0.2025
LOAD 4 LIVELOAD (CLASS A DOUBLE LANE)
MEM LOAD
1 UNI GY -122.62 0.585
2 UNI GY -122.62 0.0 0.415
2 UNI GY -73.11 1.395
3 UNI GY -73.11 0 0.2025
3 UNI GY -45.21 0.55 2.055
4 UNI GY -45.21 0.0 0.05
4 UNI GY -42.84 0.5 1.5
LOAD 5 LIVELOAD (CLASS 70 R MAX. SPAN MOMENT)
MEM LOAD
2 UNI GY -64.18 0.805 2.155
3 UNI GY -58.69 0.575 1.845
LOAD 6 LIVELOAD (CLASS 70 R MAX. SUPPORT MOMENT)
MEM LOAD
3 UNI GY -58.92 0.61 1.86
4 UNI GY -58.92 0.34 1.59
LOAD 7 LIVELOAD (CLASS A MAX. SUPPORT MOMENT)
MEM LOAD
2 UNI GY -43.7 0.655 1.945
3 UNI GY -43.7 0.255 1.545
LOAD 8 LIVELOAD (CLASS 70 R MAX LOAD AT MID SPAN)
MEM LOAD
3 UNI GY -58.23 0.55 1.65
4 UNI GY -61.09 0.165 1.505
LOAD 9 LIVELOAD (CLASS A MAX LOAD AT MID SPAN)
```

```
MEM LOAD
3 UNI GY -41.35 0.555 1.645
4 UNI GY -46.11 0.035 1.375
LOAD COMBINATION 10
1 1 2 1 3 1
LOAD COMBINATION 11
1 1 2 1 4 1
LOAD COMBINATION 12
1 1 2 1 5 1
LOAD COMBINATION 13
1 1 2 1 6 1
LOAD COMBINATION 14
1 1 2 1 7 1
LOAD COMBINATION 15
1 1 2 1 8 1
LOAD COMBINATION 16
1 1 2 1 9 1
PERFORM ANALYSIS
PRINT MEMBER FORCES MEMB
PRINT MAX FORCE ENVELOPE
LOAD LIST 10 TO 16
SEC 0.001 0.999 MEM 1 TO 5
PRINT SECTION FORCES
SEC .25 .5 .75 MEM 2 TO 4
PRINT SECTION FORCES
Print section forces
FINISH
```



Member numbers



BMD Envelope

## Design of section

### Material properties and design constants

Concrete	M50	m	10	k	0.455	Depth	250
Steel	Fe415	$\sigma_c$ (MPa)	16.67	j	0.848	Cover	50
		$\sigma_{st}$ (MPa)	200	Q (MPa)	3.21		

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

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

The design moments have been taken from staad Output  
Refer to the staad details attached

### Design

(a) Cantilever support (hogging)

At support

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

**Provided depth enough**

Steel requirement

Ast =	215	mm <sup>2</sup>	
provide 10mm bars at 150mm c/c			
Bar area =	78.5	mm <sup>2</sup>	
Steel provided	524	mm <sup>2</sup>	

### Design

(b) Intermediate support (hogging)

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

**Provided depth enough**

Steel requirement

Ast =	890	mm <sup>2</sup>	
provide 16 mm bars at 150mm c/c			
Bar area =	201	mm <sup>2</sup>	
Steel provided	1340	mm <sup>2</sup>	

Bar area =	113	mm <sup>2</sup>	
Steel provided	754	mm <sup>2</sup>	

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

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

**Provided depth enough**

Steel requirement

Ast =	706	mm <sup>2</sup>	
provide 16 mm bars at 150mm c/c			
Bar area =	201	mm <sup>2</sup>	
Steel provided	1340	mm <sup>2</sup>	

### Distribution steel

Design moment = 0.3*Limoment + 0.2* DL moment			
Maz live load moment	26	kNm	
Maz dead load moment	6	kNm	
Design moment =	9		

Ast required	296.29	mm <sup>2</sup>	
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0.12% of Cross sectional area = 300 mm<sup>2</sup>

Provide 10mm dia bars

Bar Area =	78.5	mm <sup>2</sup>	
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Spacing	261.6667		
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Provide 10mm bars at 175mm c/c

**STRUCTURAL DATA.(Mid Girder)**

Span of Bridge	=	39520 mm	(length of girder)
Centre to centre of bearing	=	38440 mm	
Carriageway Width	=	15 mm	19760
Width of footpath	=	0 mm	19220
Width of crash barrier	=	500 mm	540
O/O of Parapet	=	17000 mm	
Depth of Deck Slab	=	250 mm	
Overall Depth of the Girder	=	2200 mm	
C/C distance between the girders	=	2200 mm	
Cantilever distance beyond end girder	=	800 mm	
Web thickness (Running X <sup>n</sup> )	=	350 mm	
Web thickness (End Block X <sup>n</sup> )	=	700 mm	
Thickness of End Diaphragm	=	800 mm	
Thickness of Intermediate Diaphragm	=	300 mm	
Density of Concrete	=	2.5 T/m <sup>3</sup>	
Grade of Concrete Used	=	<b>M 500</b>	
Characteristic Compressive Strength	=	500 Kg/cm <sup>2</sup>	
Permissible stress in Concrete.	=	165 Kg/cm <sup>2</sup>	
Modulus of Elasticity of Concrete "Ec"	=	353553.3906	5000 x √f <sub>ck</sub> Mpa
Grade of Steel Used	=	<b>Fe-415</b>	
Yield strength of Steel	=	2000 Kg/cm <sup>2</sup>	
Modulus of Elasticity of Steel "Es"	=	2.0E+06 Kg/cm <sup>2</sup>	(Vide Cl:10.1.1 of I.R.C:-18-2000)

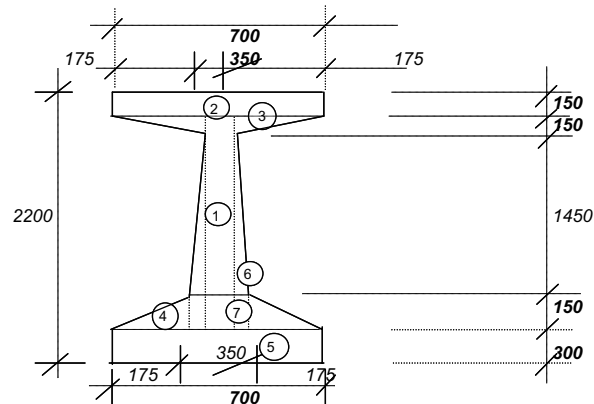
**References :**

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

**SECTION PROPERTIES**

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

a) **Simple Section at Mid span**



1) Moment of Inertia. (I<sub>zz</sub>)

Sl	Description	Nos	Area	x	Ax	Ax <sup>2</sup>	I <sub>self</sub>
1)	35 x 175.0 x 1	1	= 6125.00	117.50	7.2.E+05	8.5.E+07	1.6.E+07
2)	70 x 15.0 x 1	1	= 1050.00	212.50	2.2.E+05	4.7.E+07	2.0.E+04
3)	( 17.5 x 15.0)/2 x 2	2	= 262.50	200.00	5.3.E+04	1.1.E+07	3.3.E+03
4)	( 17.5 x 15 )/2 x 2	2	= 262.50	35.00	9.2.E+03	3.2.E+05	3.3.E+03
5)	70 x 30 x 1	1	= 2100.00	15.00	3.2.E+04	4.7.E+05	1.6.E+05
6)	( 0 x 14 )/ x 2	2	= 0.00	93.33	0.0.E+00	0.0.E+00	0.0.E+00
7)	0 x 15 x 2	2	= 0.00	37.50	0.0.E+00	0.0.E+00	0.0.E+00
<b>Total</b>			<b>9800.00</b>		<b>1.0.E+06</b>	<b>1.4.E+08</b>	<b>1.6.E+07</b>

Distance of N.A from bottom "Vb" =  $\frac{Ax}{A} = \frac{1036000}{9800} = 105.7142857$  cms

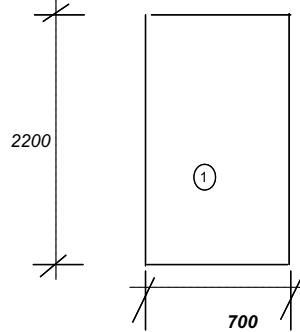
Distance of N.A from top "Vt" =  $220 - 105.714 = 114.2857143$  cms

Moment of Inertia about N.A "I<sub>zz</sub>" =  $I_{self} + Ax^2 - (Ax \times Vb)$   
 =  $15815260.42 + 1.4E+08 - (1036000 \times 105.7142857)$   
 = **4.96E+07 cm<sup>4</sup>**

Section Modulus about top "Z<sub>t</sub>" =  $\frac{I_{self}}{Vt} = \frac{5E+07}{114.286} = 433708.3333$  cm<sup>3</sup>

Section Modulus about bottom "Z<sub>b</sub>" =  $\frac{I_{self}}{Vb} = \frac{5E+07}{105.714} = 468873.8739$  cm<sup>3</sup>

b) Simple section at end Block



1) Moment of Inertia. (I<sub>zz</sub>)

Sl	Description	Nos	Area	x	Ax	Ax <sup>2</sup>	I <sub>self</sub>
1)	70 x 220 x 1	1	= 15400.00	110.00	1.7.E+06	1.9.E+08	6.2.E+07
<b>Total</b>			<b>15400.00</b>		<b>1.7.E+06</b>	<b>1.9.E+08</b>	<b>6.2.E+07</b>

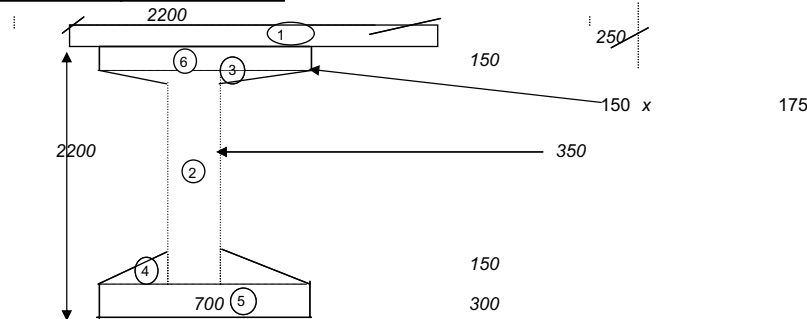
Distance of N.A from bottom "Vb" =  $\frac{Ax}{A} = \frac{1694000}{15400} = 110$  cms

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

Moment of Inertia about N.A "I<sub>zz</sub>" =  $I_{self} + Ax^2 - (Ax \times Vb)$   
 =  $62113333.33 + 1.9E+08 - (1694000 \times 110)$   
 = **62113333.33 cm<sup>4</sup>**

$$\begin{aligned} \text{Section Modulus about top "Zt"} &= \frac{I_{self}}{V_t} = \frac{6.2E+07}{110} = 564666.6667 \text{ cm}^3 \\ \text{Section Modulus about bottom "Zb"} &= \frac{I_{self}}{V_b} = \frac{6.2E+07}{110} = 564666.6667 \text{ cm}^3 \end{aligned}$$

a) **Composite Section at Mid span for end Girders**



1) **Moment of Inertia. ( $I_{zz}$ )**

Sl	Description	Nos	Area	x	Ax	Ax <sup>2</sup>	I <sub>self</sub>
1)	220 x 25.0 x 1	=	5500.00	232.5	1.3.E+06	3.0.E+08	2.9.E+05
2)	35 x 175.0 x 1	=	6125.00	117.5	7.2.E+05	8.5.E+07	1.6.E+07
3)	( 17.5 x 15.0 )/2 x 2	=	262.50	200.0	5.3.E+04	1.1.E+07	3.3.E+03
4)	( 17.5 x 15 )/2 x 2	=	262.50	35.0	9.2.E+03	3.2.E+05	3.3.E+03
5)	70 x 30 x 1	=	2100.00	15.0	3.2.E+04	4.7.E+05	1.6.E+05
6)	( 70 x 15 x 1	=	1050.00	212.5	2.2.E+05	4.7.E+07	2.0.E+04
<b>Total</b>			<b>15300.00</b>		<b>2.3.E+06</b>	<b>4.4.E+08</b>	<b>1.6.E+07</b>

$$\text{Distance of N.A from bottom "Vb"} = \frac{Ax}{A} = \frac{2314750}{15300} = 151.2908497 \text{ cms}$$

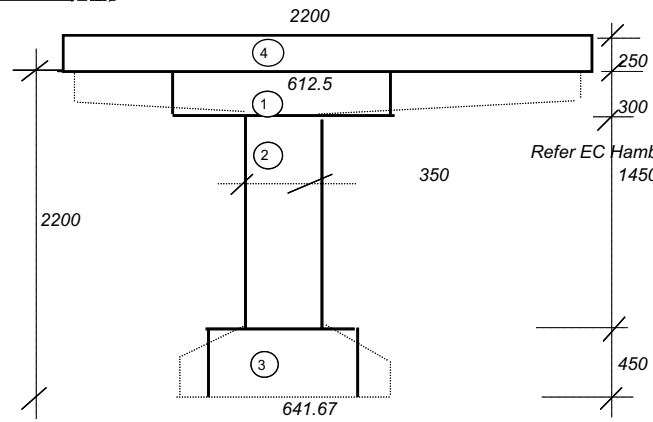
$$\text{Distance of N.A from top "Vt"} = 245 - 151.291 = 93.70915033 \text{ cms}$$

$$\begin{aligned} \text{Moment of Inertia about N.A "Izz"} &= I_{self} + Ax^2 - ( Ax \times Vb ) \\ &= 16101718.75 + 4.4E+08 - ( 2314750 \times 151.2908497 ) \\ &= 106482005.7 \text{ cm}^4 \end{aligned}$$

$$\text{Section Modulus about top "Zt"} = \frac{I_{self}}{V_t} = \frac{1.1E+08}{93.7092} = 1136303.182 \text{ cm}^3$$

$$\text{Section Modulus about bottom "Zb"} = \frac{I_{self}}{V_b} = \frac{1.1E+08}{151.291} = 703823.172 \text{ cm}^3$$

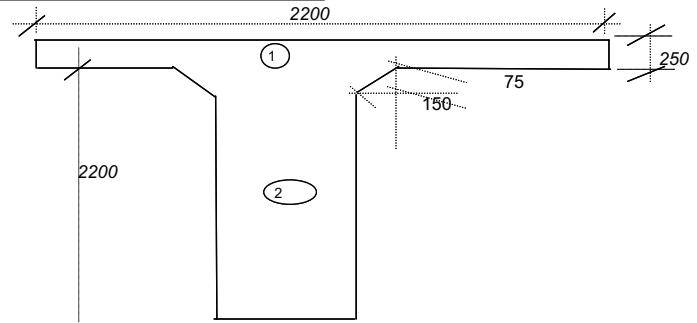
2) Torsional Stiffness ( $I_{xx}$ )



Refer EC Hambly's book on Bridge Deck Behaviour.

Area 1	=	400133.0618
Area 2	=	1762379.284
Area 3	=	1175849.905
Area 4	=	1018103.009
$\Sigma I_{xx}$	=	4356465.259 $cm^4$
	=	<b>0.0436 <math>m^4</math></b>

b) Composite section at end Block for Inner Girders





1) Moment of Inertia. ( $I_{zz}$ )

700

Sl	Description	Nos	Area	x	Ax	Ax <sup>2</sup>	Iself
1)	220 x 25	x 1	= 5500.00	232.5	1.3.E+06	3.0.E+08	2.9.E+05
2)	70 x 220	x 1	= 15400.00	110	1.7.E+06	1.9.E+08	6.2.E+07
3)	15 x 7.5	x 0	= 0.00	217.5	0.0.E+00	0.0.E+00	0.0.E+00
Total			<b>20900.00</b>	-	<b>2972750</b>	<b>4.8.E+08</b>	<b>6.2.E+07</b>

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

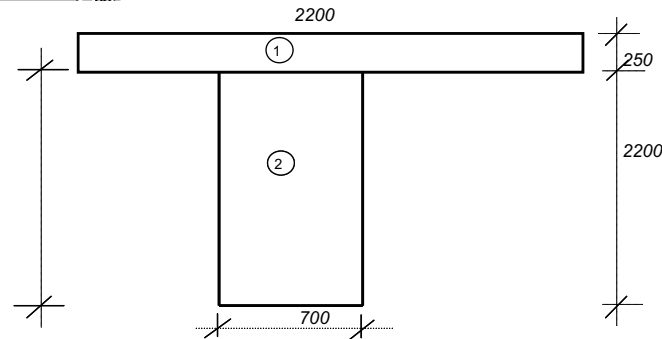
Distance of N.A from top "Vt" =  $245 - 142.237 = 102.7631579$  cms

Moment of Inertia about N.A "Izz" = Iself + Ax<sup>2</sup> - ( Ax x Vb )  
 = 62399791.67 + 4.8E+08 - ( 2972750 x 142.2368421 )  
 = **123214594.3** cm<sup>4</sup>

Section Modulus about top "Zt" =  $\frac{Iself}{Vt} = \frac{1.2E+08}{102.763} = 1199015.258$  cm<sup>3</sup>

Section Modulus about bottom "Zb" =  $\frac{Iself}{Vb} = \frac{1.2E+08}{142.237} = 866263.5677$  cm<sup>3</sup>

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



Refer EC Hambly's book on Bridge Deck Behaviour.

Area 2 = 20556833.02  
 Area 1 = 1018103.009  
 $\Sigma I_{xx} = 21574936.03$  cm<sup>4</sup>  
 = **0.2157** m<sup>4</sup>

c) **End Diaphragm**

Effective flange width =  $b_w + (l_o / 10)$ , as per IRC: 21-2000, for L beams.

$$\frac{x}{x} = 1482.81 \text{ mm}$$

1) **Moment of Inertia. ( $I_{zz}$ )**

Area 1 =  $5.7E+07 \text{ cm}^4$

Area 2 =  $9.7E+07 \text{ cm}^4$

$\Sigma I_{xx}$  =  $1.5E+08 \text{ cm}^4$

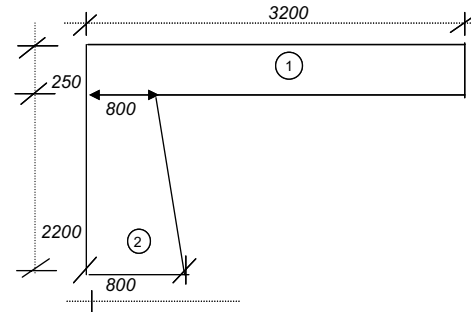
**Torsional Stiffness ( $I_{xx}$ )**

Area 1 = 1490900

Area 2 =  $3E+07$

$\Sigma I_{xx}$  =  $3.1E+07 \text{ cm}^3$

=  **$0.31336 \text{ m}^3$**



d) **Intermediate Diaphragm**

Effective flange width =  $b_w + (l_o / 5)$ , as per IRC: 21-2000, for T beams.

$$\frac{x}{x} = 1482.81 \text{ mm}$$

**Moment of Inertia. ( $I_{zz}$ )**

Area 1 =  $2.1E+07 \text{ cm}^4$

Area 2 =  $3.6E+07 \text{ cm}^4$

$\Sigma I_{zz}$  =  $5.8E+07 \text{ cm}^4$

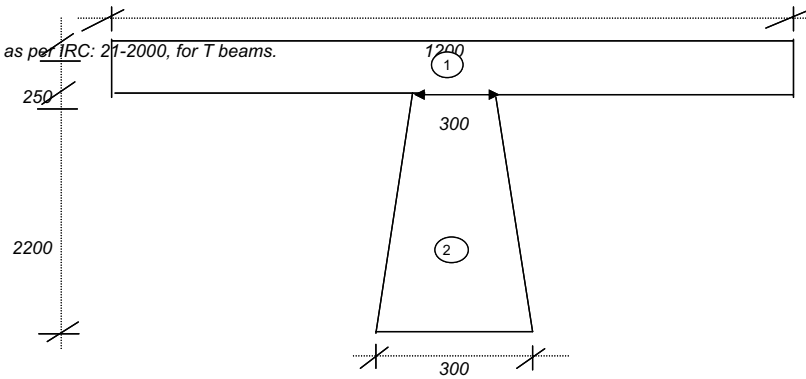
**Torsional Stiffness ( $I_{xx}$ )**

Area 1 = 539101

Area 2 = 1749469

$\Sigma I_{xx}$  =  $2288570 \text{ cm}^3$

=  **$0.02289 \text{ m}^3$**



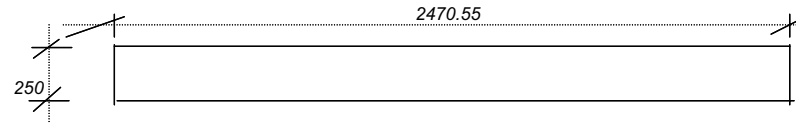
e) **Transverse Members**

Moment of Inertia (I<sub>zz</sub>)

$$\Sigma I_{zz} = 0.00322 \text{ m}^4$$

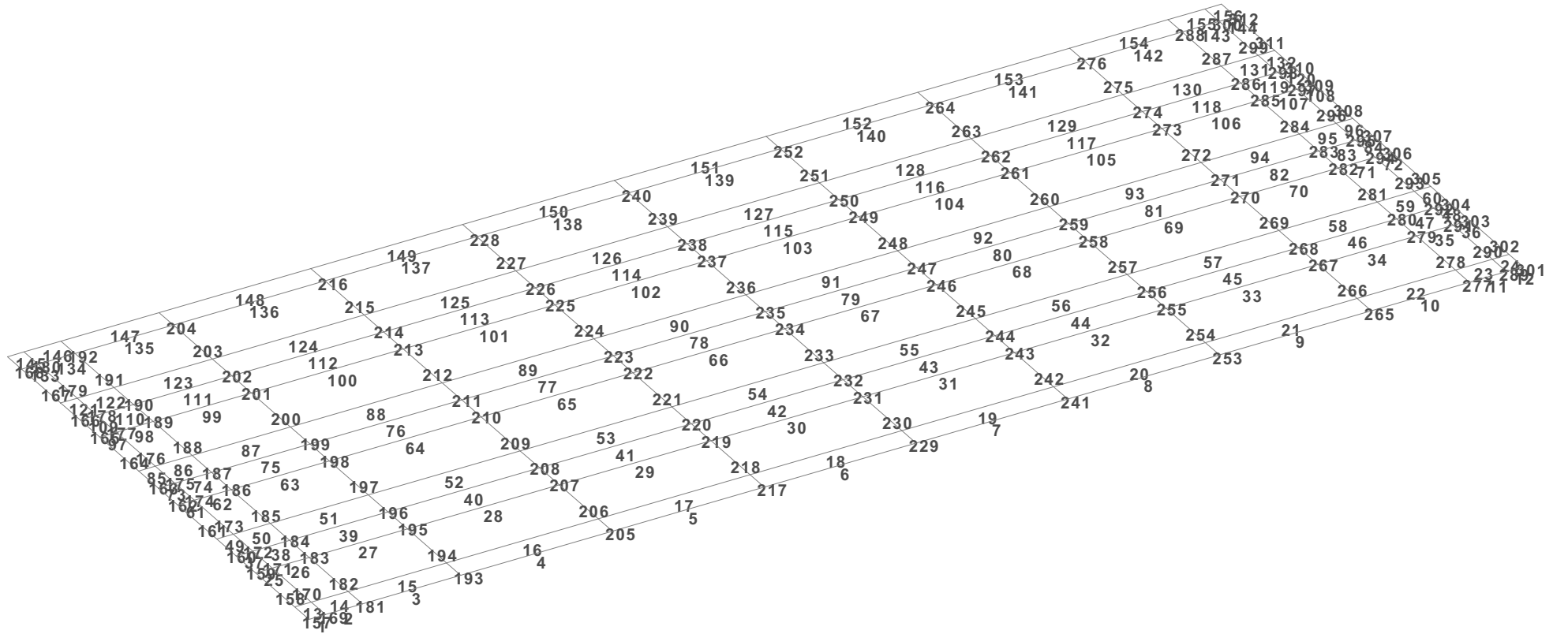
Torsional Stiffness (I<sub>xx</sub>)

$$\Sigma I_{xx} = 0.00643 \text{ cm}^3$$



Summary of Member Properties for Grillage Analysis

Description	Area m <sup>2</sup>	C. G from Bottom m	Moment of Inertia m <sup>4</sup>	Torsional Constant m <sup>4</sup>	Vb in cm	Zt in cm <sup>3</sup>	Zb in cm <sup>3</sup>
<b>Simple</b> Running	0.9800	1.0571	0.4957	0.0000	105.7143	433708.3333	468873.8739
End block	1.5400	1.1000	0.6211	0.0000	110.0000	564666.6667	564666.6667
Average	1.2600	1.0786	0.5584	0.0000	107.8571	499187.5000	516770.2703
<b>Composite</b> Running 2&3	1.5300	1.5129	1.0648	0.0436	151.2908	1136303.1822	703823.1720
End block 2&3	2.0900	1.4224	1.2321	0.2157	142.2368	1199015.2582	866263.5677
<b>Averaged</b> For 2 & 3	1.8100	1.4676	1.1485	0.1297	146.7638	1167659.2202	785043.3699
<b>End Diaphragm</b>	2.5600	1.4828	0.5773	0.3134	-	0.0000	0.0000
<b>Intermediate Diaphragm</b>	0.9600	1.4828	0.5773	0.0229	-	0.0000	0.0000
<b>Transverse Members</b>	0.6176	0.1250	0.0032	0.0064	-	0.0000	0.0000



Model

```

STAAD PLANE stage - 1
START JOB INFORMATION
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
joint coordinates
1 0 0 0; 2 0.54 0 0; 3 1.74 0 0; 4 4.94 0 0 10 34.58 0 0
11 37.78 0 0; 12 38.98 0 0; 13 39.52 0 0
member incidences
1 1 2 12
member property indian
1 2 11 12 pri ax 1.54 ix 0.001 iz 0.62
3 10 pri ax 1.26 ix 0.001 iz 0.558
4 to 9 pri ax 0.98 ix 0.001 iz 0.496
constants
e concrete all
poi concrete all
supports
2 12 pinned
load 1
member load
1 2 11 12 uni gy -38.5
3 10 uni gy -31.5
4 to 9 uni gy -24.5
Load 2 end girder
member load
slab
0.25*1.9*25 = 11.875
1 to 12 uni gy -11.875
pre-cast slab
0.05*1.5*25/2=0.9375
1 to 12 uni gy -0.9375
diaphragm
joint load
1 13 fy -4.9
7 FY -16.5
load 3 mid girder
member load
slab
0.25*2.2*25 = 13.75
1 to 12 uni gy -13.75
pre-cast slab
0.05*1.5*25=1.875
1 to 12 uni gy -1.875
diaphragm
joint load
1 13 fy -9.75
7 FY -33
perform analysis
PRINT MAXFORCES ENVELOPE
print member forces
print support reactions
FINISH

```

```

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 15-Dec-07
END JOB INFORMATION
INPUT WIDTH 79
unit kn meter
joint coordinates
1 0 0 0; 2 0.54 0 0; 3 1.74 0 0; 4 4.94 0 0 10 34.58 0 0
11 37.78 0 0; 12 38.98 0 0; 13 39.52 0 0
repeat all 1 0 0 -0.8
repeat 1 0 0 -2.2
repeat 1 0 0 -1.1
repeat 1 0 0 -1.1
repeat 1 0 0 -2.2
repeat 1 0 0 -1.1
repeat 1 0 0 -1.1
repeat 1 0 0 -2.2
repeat 1 0 0 -1.1
repeat 1 0 0 -1.1
repeat 1 0 0 -2.2
repeat 1 0 0 -0.8
member incidences
1 1 2 12
13 14 15 24
25 27 28 36
37 40 41 48
49 53 54 60
61 66 67 72
73 79 80 84
85 92 93 96
97 105 106 108
109 118 119 120
121 131 132 132
133 144 145 144
145 157 158 156
157 1 14 168 1 13
169 2 15 180 1 13
181 3 16 192 1 13
193 4 17 204 1 13
205 5 18 216 1 13
217 6 19 228 1 13
229 7 20 240 1 13
241 8 21 252 1 13
253 9 22 264 1 13
265 10 23 276 1 13
277 11 24 288 1 13
289 12 25 300 1 13
301 13 26 312 1 13
memeber property indian
end memebbers
1 to 12 145 to 156 pri ax 0.001 ix 0.0001 iy 0.0001 iz 0.0001
end members outer girders
13 14 23 24 133 134 143 144 pri ax 2.09 ix 0.216 iy 0.00001 iz 1.23
end memebbers inner girders
25 26 35 36 49 50 59 to 62 71 72 85 86 pri ax 1.95 ix 0.213 iy 0.00001 iz
1.11
107 108 121 122 131 to 132 95 to 98 pri ax 1.95 ix 0.213 iy 0.00001 iz 1.11
taper memebbers outer girders
15 22 135 142 pri ax 1.81 ix 0.13 iy 0.00001 iz 1.145
taper members inner girders
27 34 51 58 63 70 87 99 123 94 106 130 pri ax 1.672 ix 0.127 iy 0.0001 iz
1.037
middle memebbers - end girders
16 to 21 136 to 141 pri ax 1.53 ix 0.0436 iy 0.00001 iz 1.06
middle memebbers - inner girders
28 to 33 52 to 57 64 to 69 pri ax 1.393 ix 0.041 iy 0.00001 iz 0.964
88 to 93 100 to 105 124 to 129 pri ax 1.393 ix 0.041 iy 0.00001 iz 0.964
memebbers at bearing location

```

```

37 to 48 73 to 84 109 to 120 pri ax .0000275 ix 0.00286 iy 0.00001 iz
0.0000143
end cross girders
170 to 179 290 to 299 pri ax 1.76 ix 0.3136 iy 0.00001 iz 1.5
mid cross girders
230 to 239 pri ax 0.66 ix 0.023 iy 0.00001 iz 0.58
deck members near end cross girder
182 to 191 278 to 287 pri ax 0.435 ix 0.00045 iy 0.00001 iz 0.0023
deck members near mid cross girder
218 to 227 242 to 251 pri ax 0.935 ix 0.0097 iy 0.00001 iz 0.0049
deck memebers
193 to 216 253 to 276 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
deck end memebers - along width
157 to 168 301 to 312 pri ax 0.0001 ix 0.00001 iy 0.00001 iz 0.00001
deck end memebers - along span
181 192 217 228 229 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
241 252 277 288 240 pri ax 1.235 ix 0.0129 iy 0.00001 iz 0.0064
deck corner memebers
169 180 289 300 pri ax 0.62 ix 0.0064 iy 0.00001 iz 0.0032
Constants
E concrete all
Poi concrete all
den 25 all
supports
41 80 119 pinned
51 90 129 fixed but fx mz
load 1
memeber load
end members outer girders
13 14 23 24 133 134 143 144 uni gy -52.25
end memebers inner girders
25 26 35 36 49 50 59 to 62 71 72 85 86 uni gy -48.75
107 108 121 122 131 to 132 95 to 98 uni gy -48.75
taper memebers outer girders
15 22 135 142 95 to 98 uni gy -45.25
taper members inner girders
27 34 51 58 63 70 87 99 123 94 106 130 uni gy -41.8
middle memebers - end girders
16 to 21 136 to 141 uni gy -38.25
middle memebers - inner girders
28 to 33 52 to 57 64 to 69 uni gy -34.825
88 to 93 100 to 105 124 to 129 uni gy -34.825
memebers at bearing location
37 to 48 73 to 84 109 to 120 uni gy -6.875
end cross girders
170 to 179 290 to 299 uni gy -44
mid cross girders
230 to 239 uni gy -16.5
load 2
memeber load
crash barrier
1 to 12 145 to 156 uni gy -9.4
Wearing course
13 to 24 uni gy -1.65 0.5
133 to 144 uni gy -1.65 0.3
25 to 132 uni gy -1.65
median .3*25 = 7.5
7.5-1.65 = 5.85)
73 to 84 uni gy -5.85
DEFINE MOVING LOAD
CLASS A four LANES
TYPE 1 LOAD 13.5 13.5 57 57 34 34 34 34 DISTANCE 1.1 3.2 1.2 4.3 3.0 3.0 3.0
TYPE 2 LOAD 13.5 13.5 57 57 34 34 34 34 DISTANCE 1.1 3.2 1.2 4.3 3.0 3.0 3.0
LOAD GENERATION 60
a ECCENTRIC (near FP)
TYPE 1 -18.8 0. -0.9 XINC 1.
TYPE 2 -18.8 0. -2.7 XINC 1.
TYPE 1 -18.8 0. -4.4 XINC 1.

```

```

TYPE 2 -18.8 0. -6.2 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
b ECCENTRIC (near median FP)
TYPE 1 -18.8 0. -7.6 XINC 1.
TYPE 2 -18.8 0. -5.8 XINC 1.
TYPE 1 -18.8 0. -4.1 XINC 1.
TYPE 2 -18.8 0. -2.3 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
C ECCENTRIC TWO LANE (near fp)
TYPE 1 -18.8 0. -0.9 XINC 1.
TYPE 2 -18.8 0. -2.7 XINC 1.
TYPE 1 -18.8 0. -4.4 XINC 1.
TYPE 2 -18.8 0. -6.2 XINC 1.
D ECCENTRIC TWO LANE(near median)
TYPE 1 -18.8 0. -7.6 XINC 1.
TYPE 2 -18.8 0. -5.8 XINC 1.
TYPE 1 -18.8 0. -4.1 XINC 1.
TYPE 2 -18.8 0. -2.3 XINC 1.
CLASS 70R WHEELED
DEFINE MOVING LOAD
TYPE 3 LOAD 40 60 60 85 85 85 85 DISTANCE 3.96 1.52 2.13 1.37 3.05 1.37
TYPE 4 LOAD 40 60 60 85 85 85 85 DISTANCE 3.96 1.52 2.13 1.37 3.05 1.37
LOAD GENERATION 55
E 70R WHEELED + CLASS A (70R E1) (70R near FP)
TYPE 3 -13.4 0. -2.13 XINC 1.
TYPE 4 -13.4 0. -4.06 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
F CLASS A + 70 R (70RE2) (70R near median)
TYPE 3 -13.4 0. -6.87 XINC 1.
TYPE 4 -13.4 0. -4.94 XINC 1.
TYPE 1 -18.8 0. -9.4 XINC 1.
TYPE 2 -18.8 0. -11.2 XINC 1.
TYPE 1 -18.8 0. -12.9 XINC 1.
TYPE 2 -18.8 0. -14.7 XINC 1.
G 70R WHEELED (on both 2 lanes near FP)
TYPE 3 -13.4 0. -2.13 XINC 1.
TYPE 4 -13.4 0. -4.06 XINC 1.
TYPE 3 -13.4 0. -10.63 XINC 1.
TYPE 4 -13.4 0. -12.56 XINC 1.
H 70R WHEELED (on both 2 lanes near FP)
TYPE 3 -13.4 0. -6.87 XINC 1.
TYPE 4 -13.4 0. -4.94 XINC 1.
TYPE 3 -13.4 0. -10.63 XINC 1.
TYPE 4 -13.4 0. -12.56 XINC 1.
perform analysis
print maxforce envelope
print support reactions
FINISH

```



Shear Force Results From Grillage Analysis

Girder	Node	DL	SIDL	Live Load								Max Load Case	Total	Max Load Case	Total	40 stage 1	
				LL A	LL B	LL C	LL D	LL E	LL F	LL G	LL H					Node	
				0	0.01	0	0	0.01	0.01	0.01	0.01					0.01	0.01
G1	13	786.34	214.73	239.75	143.43	219.68	130.81	204.51	211.26	104.6	109.95	239.75	1216.85	1216.845	1478.34	1	510.09
	14	724.05	171.93	236.7	140.72	221.7	130.69	203.85	209.94	102.4	107.47	236.7	1109.01	1109.01	1375.53	2	463.89
	15	579.25	145.97	223.73	124.62	212.94	129.01	185.95	189.4	90.13	95.19	223.73	926.577	926.577	1182.27	3	363.09
	16	388.9	98.36	200.42	116.85	180.59	124.82	162.62	166.92	87.77	92.67	200.42	667.638	667.638	918.144	4	242.06
	17	199.94	36.16	171.97	108.1	152.39	120.62	148.08	152.44	85.41	89.99	171.97	390.873	390.873	631.856	5	121.03
	18	10.43	26.78	34.8	0.65	54.36	0.7	35.83	28.9	0.7	0.68	54.36	86.134	86.134	112.971	6	0
	19	200.6	36.9	157.63	102.19	139.63	114.44	162.14	166.25	86.15	90.32	166.25	387.125	387.125	608.233	7	121.03
	20	389.71	99.08	196.2	117.14	174.51	125.15	167.35	172.82	88.39	92.41	196.2	665.37	665.37	912.173	8	242.06
	21	579.66	146.64	223.85	126.26	208.16	130.69	189.02	193.81	91.1	95.23	223.85	927.765	927.765	1185.1	9	363.09
	22	724.46	172.59	237.09	140.15	215.02	130.16	212.14	211.32	103.3	107.71	237.09	1110.43	1110.431	1376.63	10	463.89
	23	786.53	215.91	236.74	142.52	211.91	129.89	214.48	213.04	105.9	110.04	236.74	1215.51	1215.506	1473.22	11	510.09
	24	0.01	215.98	331.31	0.01	0	0	0.01	0.01	0.02	0.01	331.31	514.169	514.169	794.072	12	0
G2	25	727.4	116.83	337.89	227.05	236.48	214.7	326.63	332.03	134.5	141.97	337.89	1148.33	1148.331	1598.44		
	26	667.87	65.87	310.12	199.3	211.09	182.33	284.72	290.72	131.1	138.99	310.12	1012.85	1012.848	1420.98		
	27	534.11	59.46	264.57	176.3	181.68	157.77	236.74	244.54	113.2	120.45	264.57	831.683	831.683	1189.38		
	28	361.57	52.57	194.02	135.41	132.47	114.83	166.13	173.44	96.91	100.77	194.02	588.758	588.758	859.312		
	29	188.59	50.63	134.55	88.69	84.28	68.94	102.84	95.87	92.68	94.72	134.55	360.315	360.315	547.524		
	30	16.55	42.48	107.98	77.54	106.89	92.73	201.56	193.87	24.24	16.85	201.56	240.434	240.434	401.911		
	31	188.67	50.61	115.16	39.05	71.73	55.76	85.81	91.68	92.97	99.39	115.16	342.924	342.924	462.842		
	32	360.68	52.57	173.9	118.76	117.23	98.1	183.41	184.81	100.5	104.56	184.81	579.579	579.579	814.281		
	33	535.83	59.7	242.06	159.48	164.83	139.87	257.86	254.9	124.5	124.42	257.86	827.604	827.604	1146.38		
	34	672.57	66.15	289.89	185.6	196.53	167.32	312.97	307.64	144.8	141.32	312.97	1020.39	1020.393	1393.84		
	35	731.07	152.45	306.91	201.58	208.03	182.41	367.68	361.6	146.1	143.56	367.68	1214.43	1214.432	1601.48		
	36	0	151.56	0.41	0.02	0	0	0.01	0	0.09	0.07	0.41	151.929	151.929	137.166		
G3	49	729.03	88.98	429.92	246.44	209.97	224.51	206.89	215.2	322	327.61	429.92	1204.94	1204.938	1760.8		
	50	670.53	64.85	387.8	219.97	181.97	189.58	184.86	194.86	284.7	292.05	387.8	1084.4	1084.4	1583.73		
	51	535.83	57.94	321.17	187.39	154.9	155.67	152.21	162.5	251.1	261.36	321.17	882.823	882.823	1303.1		
	52	363.12	46.67	235.74	137.71	109.33	100.23	116.03	124.31	181	192.66	235.74	621.956	621.956	933.21		
	53	190.49	35.54	136.88	93.54	64.12	55.55	87.52	88.42	159	147.22	158.95	369.085	369.085	562.597		
	54	19.08	23.52	191.44	94.71	102.01	120.4	82.26	69.25	257.5	244.25	257.53	274.377	274.377	533.089		
	55	191.11	35.53	121.74	78.6	55.08	64.46	94.05	102.42	118.2	123.24	123.24	337.556	337.556	504.14		
	56	362.95	46.58	208.85	121.5	94.7	83	134.42	138.41	219.6	217.57	219.63	607.197	607.197	876.827		
	57	534.41	57.66	289.98	169.55	138.66	135.31	180.03	176.19	294.5	285.15	294.49	857.111	857.111	1230.93		
	58	664.65	64.52	345.37	203.15	165.81	170.22	218.18	208.23	332	319.02	345.37	1040	1040.003	1484.52		
	59	720.95	66.5	360.72	218.5	178.07	186.08	247.49	235.45	356.2	344.27	360.72	1112.1	1112.098	1580.11		
	60	0	45.96	0.16	0	0.06	0.03	0.03	0.03	0.04	0	0	0.16	46.104	46.104	41.6536	

		0.01	12.09	149.94	0	0	0	0.01	0.03	0.03	0	149.94	147.046	149.94	284.886
G4	61	720.88	91.38	220.26	238.81	102.19	184.04	143.93	138.26	257.3	269.67	269.67	1054.96	1054.963	1408.54
	62	663.85	89.4	218	224.41	102.23	173.3	140.81	135.85	244.9	261.34	261.34	988.456	988.456	1332.02
	63	533.13	75.54	191.25	194.78	90.88	150.81	123.23	120.47	211.3	226.05	226.05	812.115	812.115	1116.93
	64	362.68	51.57	153.57	148.66	75.11	109.41	110.39	116.71	140.6	161.68	161.68	559.762	559.762	806.016
	65	190.9	24.8	134.97	106.8	63.38	64.08	104.35	123.98	123.2	114.3	134.97	337.173	337.173	545.226
	66	18.86	3.06	33.56	98.69	24.54	118.48	6.42	2.08	197.9	176.5	197.88	200.012	200.012	312.261
	67	189.66	24.76	117.24	97.42	58.47	69.01	111.11	123.94	123.7	133.48	133.48	334.552	334.552	515.757
	68	361.6	51.6	143.07	138.32	72.66	90.74	106.9	116.88	188.6	183.52	188.6	582.94	582.94	806.036
	69	533.41	75.83	181.56	183.54	88.66	127.08	115.84	121.13	260.1	247.04	260.11	843.339	843.339	1124.11
	70	667.21	89.85	211.75	213.29	101.12	148.16	134.48	137.62	299.6	283.7	299.61	1026.71	1026.709	1349.08
	71	725.71	91.83	219.31	222.64	102.35	156.8	137.04	140.62	310.7	288.83	310.66	1097.13	1097.134	1429.37
	72	0	20.01	180.84	0	0	0	0.04	0.03	0.03	0	180.84	182.766	182.766	345.329
G5		0.01	10.49	49.12	0	0.08	0.07	0.03	0.03	0.04	0.01	49.12	54.708	54.708	98.3572
	85	721.32	91.42	296.56	238.4	54.46	73.51	216.48	190.87	259.4	230.19	296.56	1079.64	1079.644	1506.64
	86	664.25	89.44	258.99	224.44	52.17	70.95	206.57	187.77	247.4	226.41	258.99	986.781	986.781	1371.53
	87	533.03	75.62	225.81	194.72	46.98	60.34	187.58	169.25	211.7	191.07	225.81	811.879	811.879	1151.22
	88	362.29	51.64	180.85	148.63	45.11	51.19	153	136.03	164.2	138.63	180.85	576.695	576.695	848.506
	89	190.43	24.83	126.01	106.79	43.67	46.98	103.33	104.67	121.1	115.19	126.01	328.669	328.669	528.602
	90	18.4	3	87.67	98.68	0.28	6.69	90.37	94.22	122.9	114.27	122.88	131.992	131.992	305.143
	91	189.49	24.8	112.09	97.42	43.15	42.05	97.07	120.87	108.3	132.23	132.23	333.297	333.297	509.477
	92	361.25	51.56	164.09	138.32	44.79	48.73	130.93	152.23	131.9	153.28	164.09	560.491	560.491	806.852
	93	532.09	75.51	210.05	183.56	47.28	58.63	154.55	185.7	176.1	207.4	210.05	796.645	796.645	1110.59
	94	664.16	89.32	244.44	213.14	51.31	68.52	169.41	205.03	207.4	243.48	244.44	973.476	973.476	1333.71
95	776.65	91.3	260.42	222.4	51.74	70.08	175.4	206.15	211.3	244.71	260.42	1102.33	1102.328	1474.92	
	96	0	19.67	24.09	0	0.08	0.07	0.03	0.03	0.01	0.01	24.09	41.351	41.351	61.3059
G6		0.01	90.79	28.44	0	0	0	0.01	0	0.01	0	28.44	116.396	116.396	133.196
	97	780.54	91.68	302.65	246.35	29.61	39.78	243.73	313.65	260.3	323.41	323.41	1163.29	1163.289	1595.96
	98	667.74	64.73	272.98	219.95	28.86	39.07	212.66	279.27	231.6	286.2	286.2	990.05	990.05	1383.98
	99	533.84	57.85	227.77	187.38	27.54	37.54	179	235.32	198.9	245.02	245.02	812.208	812.208	1146.14
	100	361.8	46.69	161.45	137.7	28.09	38.11	122.59	158.76	143.8	175.47	175.47	566.413	566.413	808.922
	101	189.93	35.64	107.8	93.54	28.62	40.07	72.62	128.77	91.08	117.63	128.77	341.463	341.463	508.657
	102	18.58	23.65	117.16	94.68	0.2	0.25	110.68	233.55	105.2	215.82	233.55	252.425	252.425	439.023
	103	190.62	35.67	92.37	78.61	28.65	40.13	64.99	113.66	87.13	102.87	113.66	328.584	328.584	466.706
	104	362.54	46.73	143.39	121.52	28.47	38.7	105.26	174.9	122.8	191.11	191.11	581.269	581.269	788.052
	105	534.32	57.72	206.19	169.58	28.08	37.84	151.35	249.95	162.1	259.66	259.66	825.734	825.734	1118.93
	106	665.08	64.42	252.19	203.06	28.26	38.25	182.96	298.95	193.1	305.78	305.78	1004.7	1004.702	1359.48
107	721.43	66.4	268.68	218.21	27.13	37.28	198.22	342.55	208.7	352.74	352.74	1105.3	1105.296	1481.66	
	108	0	46.46	13.56	0	0	0	0	0	0	0	13.56	58.664	58.664	66.3576

		0.01	117.75	0.12	0.02	0.06	0.05	0.03	0.07	0.02	0.04	0.12	117.868	117.868	106.221
G7	121	728.19	118.64	274.49	227.45	9.99	22.38	220.46	169.48	238.5	178.19	274.49	1093.87	1093.871	1486.42
	122	669.69	66.37	235.34	199.3	10.02	22.43	188.02	154.9	210.6	167.38	235.34	947.866	947.866	1287.72
	123	534.77	59.91	197.67	176.36	9.06	21.97	161.76	125.22	187	144.85	197.67	772.583	772.583	1069.35
	124	361.63	52.77	145.02	135.45	7.56	20.91	117.24	85.97	141.7	111.01	145.02	544.918	544.918	770.896
	125	188.29	50.8	88.69	88.71	8.17	21.29	71.4	63.11	89.82	87.32	89.82	319.928	319.928	465.335
	126	16.51	42.66	116.4	77.52	0.08	0.12	91.22	68.47	81.09	50.2	116.4	163.93	163.93	341.457
	127	188.55	50.81	75.75	74.87	8.21	21.3	58.98	74.33	84.34	97.89	97.89	327.461	327.461	445.335
	128	360.76	52.79	126.48	118.76	7.44	20.78	100.52	96.55	124.6	122.11	126.48	527.382	527.382	719.884
	129	535.94	60.01	176.98	159.47	8.68	21.69	142.81	134.32	159.5	156.06	176.98	755.232	755.232	1016.16
	130	672.68	66.47	214.45	185.59	10.67	23.05	170.66	166.01	182.3	180	214.45	932.155	932.155	1238.98
	131	731.18	154.47	230.64	201.57	11.64	23.86	186.31	185.47	198.1	188.52	230.64	1093.23	1093.226	1416.11
	132	0	153.58	0.4	0.02	0.06	0.05	0.03	0.05	0.01	0.04	0.4	153.94	153.94	138.966
G8		0.01	216.81	110.05	0.01	0.01	0.01	0.01	0.02	0.01	0.01	110.05	315.865	315.865	394.339
	133	786.58	216.41	96.46	143.41	0.11	14.42	124.55	94.78	154.4	124.48	154.35	1141.91	1141.905	1267.58
	134	724.29	173.06	96.58	140.71	1.03	11.39	120.79	91.55	151.8	122.39	151.76	1033.93	1033.934	1167.83
	135	579.49	146.77	93.47	124.59	4.56	0.07	100.9	76.07	136.3	111.87	136.3	848.93	848.93	982.097
	136	389.2	98.89	92.19	116.84	22.96	4.56	94.68	73.96	128.7	110.08	128.71	603.929	603.929	752.566
	137	200.25	36.4	92.22	107.1	26.7	7.96	91.68	73.63	120.5	110.5	120.51	345.109	345.109	509.918
	138	10.84	26.84	8.21	0.65	31.78	12.58	12.27	0.59	0.74	0.85	31.78	66.282	66.282	68.5138
	139	200.95	36.896	91.11	102.17	26.82	8.05	77.69	73.78	114.4	111.12	114.4	340.806	340.806	500.005
	140	390	99.33	90.43	117.12	23.01	4.58	94.78	73.84	129	109.81	129	605.43	605.43	752.437
	141	579.87	147.18	92.2	126.25	4.55	0.09	110.38	76.27	136	112.16	135.96	849.414	849.414	982.923
	142	724.67	173.46	95.89	140.14	1.04	11.45	124.77	91.91	144.5	122.85	144.5	1028.18	1028.18	1161.39
	143	786.73	217.35	95.79	142.51	0.04	14.46	126.54	95.15	145.8	124.89	145.82	1135.32	1135.318	1260.09
144	0.01	217.74	112.35	0.01	0.01	0.01	0.01	0.02	0.01	0.01	112.35	318.865	318.865	399.339	

Results From Grillage Analysis

Girder	Node	Girder alone	Deck slab	DL	SIDL	Live Load								Max Load Case	Total
						LL A	LL B	LL C	LL D	LL E	LL F	LL G	LL H		
		0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01
G1	13	5.61	1.87	31.1	72.16	28.46	4.39	19.18	89.57	40.97	54.02	0.01	0.01	89.57	160.253
	14	578.77	294.31	937.09	111.06	280.51	289.49	199.89	172.49	278.72	309.64	251.94	282.95	309.64	1262.82
	15	1901.94	993.93	3024.09	519.45	952.05	733.33	818.49	571.57	876.69	933.67	565.75	601.48	952.05	4272.17
	16	3396.67	1816.37	5410.99	948.03	1797.54	1317.1	1644.17	1158.19	1684.99	1753.72	1006.7	1056.61	1797.54	7778.86
	17	4293.5	2326.13	6866.33	1084.85	2303.9	1810.65	2168.23	1674.99	2272.5	2350.77	1442.81	1510.8	2350.77	9820.17
	18	4592.44	2523.22	7383.85	912.73	2461.49	2184.52	2384.48	2106.95	2718.08	2807.06	1866.87	1956.87	2807.06	10554.7
	19	4293.5	2326.13	6865.59	1084.76	2185.13	1775.36	2052.4	1639.59	2309.35	2361.72	1444.04	1510.48	2361.72	9829.94
	20	3396.67	1816.37	5407.51	947.59	1666.36	1289.74	1508.71	1125.62	1786.54	1794.28	1019.81	1057.71	1794.28	7775.48
	21	1901.94	993.93	3014.57	517.73	891.93	715.85	733.7	545.84	1014.46	975.87	605.95	603.58	1014.46	4326.61
	22	578.77	294.31	926.8	109.13	263.83	277.6	163.04	156.21	370.22	324.52	313.65	285.85	370.22	1315.41
	23	5.61	1.87	20.58	74.42	32.16	7.37	159.85	88.22	80.71	57.36	8.77	0.01	159.85	225.765
	24	0	0	0	0.01	0	0	0	0	0	0	0	0	0	0.01
		0	0	0	0	0	0	0	0	0	0	0	0	0	
G2	25	5.61	2.28	25.63	44.6	53.55	54.14	86.36	202.35	202.35	193	0.03	1.25	202.35	234.605
	26	578.77	366.65	812.14	137.37	251.89	223.1	209.77	199.35	199.35	214.26	192.8	211.91	251.89	1309.49
	27	1901.94	1240.45	2735.3	366.05	917.52	807.41	812.47	969.73	969.73	1009.96	607.6	648.49	1009.96	4417.4
	28	3396.67	2275.2	4945.38	643.24	1717.92	1527.95	1496.84	1831.48	1831.48	1912.68	1146.08	1221.58	1912.68	8036.52
	29	4293.5	2928.69	6298.92	883.35	2116.3	1903.07	1773.25	2253.61	2253.61	2347.03	1536.84	1628.17	2347.03	10217.9
	30	4592.44	3200.85	6805.05	1111.23	2126.72	1861.94	1664.26	9.67	9.67	2354.4	1882.17	1966.52	2354.4	11023.5
	31	4293.5	2928.69	6298.6	883.32	1971.83	1762.9	1640.89	2359.44	2359.44	2374.68	1610.17	1638.49	2374.68	10242.7
	32	3396.67	2275.2	4947.45	643.76	1559.41	1389.07	1337.25	1980.7	1980.7	2358.16	1258.85	1249.31	2358.16	8437.97
	33	1901.94	1240.45	2741.85	367.45	844.09	746.67	721.17	1120.58	1120.58	1971.24	687.86	666.11	1971.24	5283.96
	34	578.77	366.65	813.22	137.61	226.81	200.74	169.6	267.6	267.6	1091.26	235.15	219.77	1091.26	2065.16
	35	5.61	2.28	28.96	43.91	45.85	45.38	82.14	193.61	193.61	248.82	4.81	0.87	248.82	275.738
	36	0	0	0	0	0	0	0	0	0	0	0	0	0	0
		0	0	0	0	0	0	0	0	0	0	0	0	0	
G3	49	5.61	2.28	29.72	26.08	35.42	50.11	75.47	93.06	40.69	27.31	188.29	170.36	188.29	203.431
	50	578.77	366.65	810.01	119.46	249.54	252.21	184.51	189.22	210.31	237.47	180.22	205.62	252.21	1291.87
	51	1901.94	1240.45	2738.65	348.67	871.73	897.66	702.67	729.83	778.81	838.21	880.65	940.22	940.22	4337.26
	52	3396.67	2275.2	4956.35	637.05	1629.42	1650.44	1274.39	1295.41	1511.93	1609.33	1613.37	1722.89	1722.89	7859.52
	53	4293.5	2928.69	6320.49	863.19	2036.05	1982.47	1476.35	1422.78	1953.84	2068.53	1935.51	2066.12	2068.53	9947.06
	54	4592.44	3200.85	6836.17	1028.58	2066.29	1909.43	1361.63	3.8	2143.52	2226.22	1949.29	2075.73	2226.22	10825.5
	55	4293.5	2928.69	6316.91	862.42	1900.56	1843.96	1355.81	1321.75	2072.26	2081.53	2090.1	2081.09	2090.1	9965.7
	56	3396.67	2275.2	4949.11	635.63	1486.9	1489.05	1127.78	1139.2	1693.06	1654.82	1798.37	1766.28	1798.37	7926.03
	57	1901.94	1240.45	2734.14	347.86	807.25	812.48	617.62	626.61	930.3	901.54	1048.41	1012.14	1048.41	4433.82
	58	578.77	366.65	821.27	121.75	232.5	220.98	154.19	160.22	272.75	266.09	246.86	233.86	272.75	1312.65
	59	5.61	2.28	8.77	30.39	37.36	42.87	59.78	83.39	35.96	17.62	188.62	177.54	188.62	208.038
	60	0	0	0	0	0	0	0	0	0	0.02	0	0	0.02	0.018

40 stage 1	
Node	
	0
1	5.61
2	578.77
3	1901.94
4	3396.66
5	4293.5
6	4592.44
7	4293.5
8	3396.66
9	1901.94
10	578.78
11	5.61
12	0





## DESIGN OF PSC GIRDER

### Design of Girder - 40m psan

#### Calculation of Bending Moments due to the following at Various X<sup>n</sup>

Summary of Shear & Moments at Various X<sup>ns</sup> obtained from respective STAAD analysis

Moment / Shear due to		1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
		<b>0.000</b>	<b>4.940</b>	<b>9.880</b>	<b>14.820</b>	<b>19.760</b>
1) DL of girder	M	-0.56	190.19	339.67	429.35	459.24
	S	72.90	53.58	36.31	19.05	1.91
2) DL of deck slab & diaphragm	M	-0.23	124.05	227.52	292.87	320.09
	S	70.00	50.00	35.00	18.00	1.50
2) S.I.D.Load	M	-4.46	36.75	64.38	88.33	111.12
	S	8.90	5.79	4.67	3.55	2.35
Vehicular Loads						
a) Class _70r -- Total	M	20.94	204.02	244.07	245.78	243.68
	S	49.44	36.93	27.11	18.28	29.62
b) Class _A --Total	M	-0.33	107.92	176.20	209.24	220.25
	S	30.39	24.34	19.05	13.99	9.03
Ultimate	M	<b>42.25</b>	<b>1054.91</b>	<b>1589.71</b>	<b>1874.44</b>	<b>2000.44</b>
Moment / Shear	S	<b>355.75</b>	<b>259.30</b>	<b>184.08</b>	<b>108.38</b>	<b>83.86</b>
		<b>M</b> Moment	<b>S</b> Shear			

The effect of vehicular impact is taken as follows

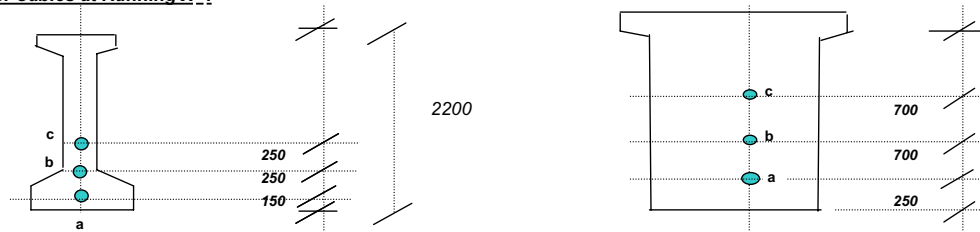
Impact Factor for Class 70r

Impact Factor

$$= \frac{4.5}{6 + 38.44} = 15.00\% \text{ (Vide Fig-5 on page 23 of IRC-6:2000)}$$

$$= 10.126\% \text{ (Vide Cl:211.2 on page 22 of IRC-6:2000)}$$

#### Position of Cables at Running X<sup>n</sup>:



#### Prestressing Details

Type of Cable Used = 22 T 15 with duct diameter 100mm 2 numbers  
 = 22 T 15 with duct diameter 90mm 1 number

Cable make = **BBRV Cona make.**

Sheathing type	=	<b>Corrugated HDPE pipe of approved make.</b>
Anchorage block size.	=	<b>375</b> mm
Distance bet <sup>n</sup> anchorages	=	<b>650</b> mm
Value of wooble co-eff	=	<b>0.002</b> per meter length of cable.as per IRC:18-2000.
Value of friction co-eff	=	<b>0.17</b> As per IRC:18-2000.
Conduit ID assembled	=	<b>100mm</b>
No of Cables Used (15mm dia)	=	<b>2</b>
No of Cables Used (15mm dia)	=	<b>1</b>
Ultimate Strength / strand (15mm dia)	=	<b>27.9 T</b> 186 Kn as per BBRV's brochure.
Ultimate Strength / strand (15mm dia)	=	<b>27.9 T</b>
Stressing to	=	<b>0.74 times UTS</b>
Stressing @ 0.74times UTS (15mm dia)	=	<b>20.646 T / STRAND</b>
Stressing @ 0.74times UTS (15mm dia)	=	<b>20.646 T / STRAND</b>
Force per cable (A ,B) (15mm dia)	=	<b>454.212 T</b> Force per cable as per BBR's brochure is 2827Kn at 0.8UTS.
Force per cable C (15mm dia)	=	<b>454.212 T</b>
C/S Area of 1 Cable (15mm dia)	=	<b>33 cm<sup>2</sup></b> As per BBRV's brochure.
C/S Area of 1 Cable (15mm dia)	=	<b>33 cm<sup>2</sup></b>
∴ Total Force	=	<b>1362.636 T</b>
	=	<b>1362636 Kg</b>
∴ Total C/S Area	=	<b>33</b> x                      2                      + 33.0                      x                      1
	=	<b>99 cm<sup>2</sup></b>
Clear cover to Prestressing Cables	=	<b>100 mm,</b> <b>75 mm</b> whichever is greater.



**Girder No : 1**

**Summary of Stresses at Various X<sup>ns</sup>**

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

19.6794

43.15473

**B** Bottom **T** Top

**Lifting point for cables**

Half length of cable = 19760 mm

c	=	18760 mm	6.108	1000
b	=	17760 mm	3.549	2000
a	=	13760 mm	0.833	6000

y	x	k	θ
1000.000	18760.0	2.8E-06	0.107
550.000	17760.0	1.7E-06	0.062
100	13760.0	5.3E-07	0.015

6.10829206  
3.54872508  
0.83278749

Cables		19760.000	14820.000	9880	4940	0
		Section 1 - 1	Section 3 - 3	Section 5 - 5	Section 7 - 7	Section 9 - 9
Cables	Dist	1650.00	1192.69	874.06	694.11	650.00
<b>C</b>	θ	0.11	0.08	0.05	0.02	0.00
Cables	Dist	950.000	686.585	508.275	400.000	400.000
<b>b</b>	θ	0.062	0.045	0.027	0.000	0.000
Cables	Dist	250.000	191.087	157.951	150.000	150.000
<b>a</b>	θ	0.015	0.009	0.004	0.000	0.000
Resultant C.G		<b>950.00</b>	<b>690.12</b>	<b>513.43</b>	<b>414.70</b>	<b>400.00</b>

6.108292

3.548725

0.832787

**Calculation of Losses in Prestress :-**

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

1) Instantaneous Losses consisting of the following :-

a) Frictional Loss : (for galvanised wire cable)

Wobble Coefficient = k = 0.002       $P_x = P_0 x e^{-(kx + \mu \theta)}$

Coefficient of Friction =  $\mu$  = **0.17**

Cable Nos	Cable Lengths	length m	k x length	$\theta$ radians	$\mu \theta$	$e^{-(kx+\mu\theta)}$
<b>c</b>	AB	0.810	0.002	0.000	0.000	0.9984
	BC	18.787	0.038	0.107	0.018	0.9458
	CD	1.000	0.002	0.000	0.000	0.9980
<b>b</b>	AB	0.890	0.002	0.000	0.000	0.9982
	BC	17.769	0.036	0.062	0.011	0.9550
	CD	2.000	0.004	0.000	0.000	0.9960
<b>a</b>	AB	0.890	0.002	0.000	0.000	0.9982
	BC	13.760	0.028	0.015	0.002	0.9705
	CD	6.000	0.012	0.000	0.000	0.9881

Cable Nos	Section	Tension Factor	Prestress Force(T)	Ave Force ( T )	Elongation (cms) $FxL / AxE$	Total Elongation (cms)
<b>c</b>	A	1	454.21		0	13.115
	B	0.99838	453.48	453.84	0.56	
	C	0.94429	428.91	441.19	12.56	
	D	0.94241	428.05	428.48	0.00	
<b>b</b>	A	1	454.21		0	13.853
	B	0.99822	453.40	453.81	0.61	
	C	0.95328	432.99	443.20	11.93	
	D	0.94947	431.26	432.13	1.31	
<b>a</b>	A	1	454.21		0	13.902
	B	0.99822	453.40	453.81	0.61	
	C	0.96873	440.01	446.71	9.31	
	D	0.95717	434.76	437.38	3.98	

b) Slip Loss :

Assuming 6mm Slip in the Prestressing Cables & that the Cables are stressed from both ends.

Force due to 6mm Slip. (for 13mmdia)for Cable A and B

$$F = \frac{AE \delta}{L}$$

$$= 33.000 \times 2.0E+06 \times 0.6 = 4.0E+05 \text{ Kg}$$

For Both end stressing case 1/2 force on either side of midpan is considered.

$$\therefore \text{Force} = \frac{396000}{2 \times 1000} = 198.00 \text{ T}$$

Force due to 6mm Slip. (for 13mm dia) for Cable C

$$F = AE \delta = 33.000 \times 2.0E+06 \times 0.6 = 4.0E+05 \text{ Kg}$$

For Both end stressing case 1/2 force on either side of midpan is considered.

$$\therefore \text{Force} = \frac{396000}{2 \times 1000} = 198.00 \text{ T}$$

Computation of Force after slip for the cables:

Cable Nos	Force in Tonne	Force Diagram after 6mm Slip
c	B 453.5	
	C 428.9	
	D 428.1	
b	B 453.4	
	C 433.0	
	D 431.3	
a	B 453.8	
	C 446.7	
	D 437.4	

Summary of Cable Forces after slip loss at various  $X^n$ .

	19760	14820	9880	4940	0
Cable Nos	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
c	419.4	426.1	432.8	440.2	428.1
b	410.7	416.43	422.10	427.78	431.26
a	421.68	424.23	426.78	429.33	437.38
<b>Σ Force</b>	1251.81	1266.73	1281.66	1297.27	1296.70

Average force in the cables (I cable) = 429.3 T  
 Force / strands =  $\frac{429.3}{22} = 19.5$  T  
 Ultimate Force / strand = 27.9 T  
 Ultimate stress / strand = 0.699 < 0.74 times ultimate. Hence safe

Average force in the cables (II and III cable) = 849.5  
 Force / strands =  $\frac{849.5}{44} = 19.3$  T  
 Ultimate Force / strand = 27.9 T  
 Ultimate stress / strand = 0.692 < 0.74 times ultimate. Hence safe  
 Average Stress in the cables = 12917.5 Kg/cm<sup>2</sup>

Horizontal Component of Cable forces at various  $X^n$ .

Cable Nos	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
c	417.0	424.8	432.2	440.0	428.1
b	410.0	416.0	421.9	427.8	431.3
a	421.6	424.2	426.8	429.3	437.4
<b>Σ Force</b>	1248.60	1264.99	1280.94	1297.16	1296.70

Vertical Component of Cable forces at various  $X^n$ .

Cable Nos	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
c	44.6	33.4	21.8	9.9	0.0
b	25.4	18.6	11.6	0.0	0.0
a	6.1	4.0	1.7	0.0	0.0
<b>Σ Force</b>	76.18	55.99	35.18	9.85	0.00

**Effect of Prestressing :-**

Sections	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
Force, F( T )	1248.6	1265.0	1280.9	1297.2	1296.7
Area, A(cm <sup>2</sup> )	15400.0	9800.0	9800.0	9800.0	9800.0
F/A (Kg/cm <sup>2</sup> )	81.1	129.1	130.7	132.4	132.3
Vb (cms)	110.0	105.7	105.7	105.7	105.7
e = Vb-x (cms)	15.0	36.7	54.4	64.2	65.7
Zt (cm <sup>3</sup> )	564667	433708	433708	433708	433708
Zb (cm <sup>3</sup> )	564666.7	468873.9	468874	468874	468874
F x e / Zt	33.2	107.0	160.6	192.1	196.5
Fxe / Zb	33.2	99.0	148.5	177.7	181.7
F/A -(Fxe / Zt)	<b>47.9</b>	<b>22.0</b>	<b>-29.9</b>	<b>-59.8</b>	<b>-64.2</b>
F/A +(Fxe / Zt)	<b>114.2</b>	<b>228.1</b>	<b>279.2</b>	<b>310.1</b>	<b>314.1</b>

2) Time Dependent Losses consisting of the following :-

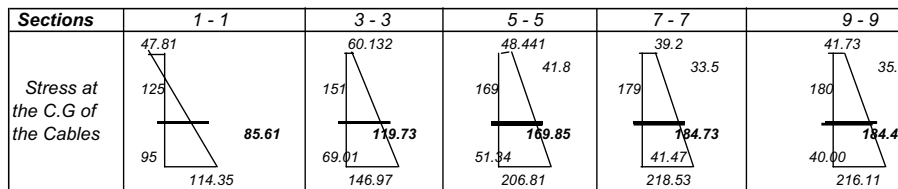
a) Elastic Shortening of Wires: (Vide Cl:11.1 of I.R.C:-18-2000)

$$\text{Loss} = 0.5 \times m \times \text{Stress at C.G of the Cables at that } X^n.$$

It is proposed to stress the cables after 28-Days when the concrete attains 100% of its full strength.

The effect of Prestress & Dead Load acts together.

Descriptions	1 - 1		3 - 3		5 - 5		7 - 7		9 - 9	
	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$
Prestress	47.910	114.246	22.032	228.100	-29.876	279.249	-59.781	310.096	-64.156	314.053
Dead Load	-0.099	0.099	38.101	-81.128	78.317	-72.443	98.995	-91.570	105.888	-97.946
<b>Resultant</b>	<b>47.810</b>	<b>114.345</b>	<b>60.132</b>	<b>146.972</b>	<b>48.441</b>	<b>206.806</b>	<b>39.214</b>	<b>218.526</b>	<b>41.732</b>	<b>216.107</b>



Average stress at C.G of the X<sup>n</sup>. = **148.86 Kg/cm<sup>2</sup>**

$$\therefore \text{Loss} = 0.5 \times 5.657 \times 148.9$$

$$= 421.05 \text{ Kg/cm}^2$$

$$\% \text{ Loss} = \frac{421.0510026}{12917.5} \times 100 = 3.260 \%$$

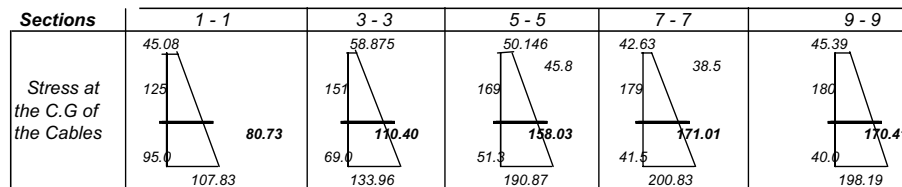
Initial relaxation loss (Cl-11.4 IRC 18-2000) **2.446 %**

$$m = \frac{E_s}{E_c} = \frac{2000000}{353553} = 5.66$$

b) Creep of Concrete:-

(Vide Cl:11.1 of I.R.C:-18-2000)

Descriptions	1 - 1		3 - 3		5 - 5		7 - 7		9 - 9	
	$\sigma_t$	$\sigma_b$	$\sigma_t$	$\sigma_b$	$\sigma_t$	$\sigma_b$	$\sigma_t$	$\sigma_b$	$\sigma_t$	$\sigma_b$
Prestress	47.910	114.246	22.032	228.100	-29.876	279.249	-59.781	310.096	-64.156	314.053
Dead Load	-0.099	0.099	38.101	-81.128	78.317	-72.443	98.995	-91.570	105.888	-97.946
Inst: Loss % 5.706	-2.734	-6.519	-1.257	-13.015	1.705	-15.934	3.411	-17.694	3.661	-17.920
<b>Resultant</b>	<b>45.077</b>	<b>107.826</b>	<b>58.875</b>	<b>133.957</b>	<b>50.146</b>	<b>190.872</b>	<b>42.625</b>	<b>200.832</b>	<b>45.392</b>	<b>198.187</b>



Average stress at C.G of the cables = **138.12** Kg/cm<sup>2</sup>  
 Creep Strain at 100% Maturity at 28-days = **0.00040** / 10 Mpa  
 Es = 2000000  
 $\therefore$  Loss =  $\frac{0.00040 \times 138.12 \times 2000000}{100} = \mathbf{1104.93}$  Kg/cm<sup>2</sup>

c) Shrinkage of Concrete:-

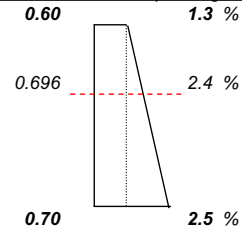
(Vide Cl:11.1 of I.R.C:-18-2000)

Shrinkage Strain at 100% Maturity at 28-days = **0.00019**  
 $\therefore$  Loss =  $0.00019 \times 2000000 = \mathbf{380.0}$  Kg/cm<sup>2</sup>

d) Relaxation of H.T.Steel

(Vide Cl:11.4 of I.R.C:-18-2000)

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



Relaxation Loss corresponding **2.446** % of the average prestress ie... 12918 Kg/cm<sup>2</sup> = 632.04 Kg/cm<sup>2</sup> ie... **4.89%**  
 $\therefore$  Total Time Dependent Losses = **2116.96** Kg/cm<sup>2</sup>

$$\therefore \% \text{ Loss} = \frac{2116.96}{12917.5} \times 100 = 16.388 \%$$

3) Time Dependent Losses consisting of the following :-  
20 % higher time dependent losses as per Cl 7.2.4 of I.R.C:-18-2000.

- a) Creep of Concrete:- = 1104.93 Kg/cm<sup>2</sup>  
b) Shrinkage of Concrete:- = 380.00 Kg/cm<sup>2</sup>  
c) Relaxation of H.T.Steel = 1.47 %

$$\% \text{ Loss} = \frac{1484.927}{12917.5} \times 20 + 1.47 = \text{Total } 3.767 \% = 1484.93 \text{ Kg/cm}^2$$

Recapitulation of Stresses at Various X<sup>n</sup>.

Descriptions	1 - 1		3 - 3		5 - 5		7 - 7		9 - 9	
	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$	$\sigma t$	$\sigma b$
Prestress	47.910	114.246	22.032	228.100	-29.876	279.249	-59.781	310.096	-64.156	314.053
Dead Load	-0.099	0.099	38.101	-81.128	78.317	-72.443	98.995	-91.570	105.888	-97.946
Inst: Loss % <b>5.706</b>	-2.734	-6.519	-1.257	-13.015	1.705	-15.934	3.411	-17.694	3.661	-17.920
<b>Resultant (28-Days)</b>	<b>45.077</b>	<b>107.826</b>	<b>58.875</b>	<b>133.957</b>	<b>50.146</b>	<b>190.872</b>	<b>42.625</b>	<b>200.832</b>	<b>45.392</b>	<b>198.187</b>
Deck slab	-0.040	0.040	43.936	-24.004	52.459	-48.525	67.527	-62.462	73.802	-68.267
S.I.D.L	-0.797	0.515	1.186	-2.610	4.154	-9.147	5.700	-12.550	7.170	-15.788
Bal: Loss % <b>16.388</b>	-7.852	-18.723	-3.611	-37.382	4.896	-45.764	9.797	-50.820	10.514	-51.468
Vehicular Load	3.740	-2.418	6.582	-14.494	15.749	-34.678	13.501	-34.921	15.724	-34.622
20%Extra Loss <b>3.767</b>	-1.805	-4.304	-0.830	-8.592	1.125	-10.519	2.252	-11.681	2.417	-11.830
<b>Resultant</b>	<b>38.324</b>	<b>82.937</b>	<b>106.138</b>	<b>46.874</b>	<b>128.529</b>	<b>42.239</b>	<b>141.402</b>	<b>28.398</b>	<b>155.019</b>	<b>16.211</b>

Remarks about Stresses at various Stages of Prestressing:

(Vide Cl: 7.1 to 7.1.4 of IRC:-18-2000.)

1) Permissible Temporary Stress in Concrete

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

$$300 \text{ Kg/cm}^2 \text{ or } 0.5 F_{cj}$$

$F_{cj}$  = Expected Concrete Strength at the time of Prestressing.

$$= 0.5 \times 500 = 250 \text{ Kg/cm}^2$$

Max Compressive Stress developed = **200.83 Kg/cm<sup>2</sup>** Hence O.K

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

$$= \frac{1}{10} \text{ of Maximum Compressive Stress immediately after Prestressing}$$

$$= \frac{-1}{10} \times 250.00 = -25.00 \text{ Kg/cm}^2$$

$$\text{Minimum Stress developed} = 42.63 \text{ Kg/cm}^2$$

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

2) Permissible Stress in Concrete at Service Condition

a) Maximum Compressive Stress allowed during Service Condition

$$= 0.33 \times F_{ck} = 0.33 \times 500 = 165 \text{ Kg/cm}^2$$

Maximum Compressive Stress attained at Service

$$= 155.0 \text{ Kg/cm}^2 \text{ Hence O.K}$$

b) Minimum Stress attained at Service

$$= 16.211 \text{ Kg/cm}^2 \text{ No Tension is developed .The Stresses are Compressive only.Hence O.K}$$

Check for Deflection at Midspan.

Downward deflection is given by  $\delta$

$$= \frac{5}{48} \times \frac{M \times L^2}{E \times I}$$

$$\begin{aligned} M &= \text{Moment} &= 1134.132 \text{ Tm} \\ L &= \text{Span} &= 39.52 \text{ m} \\ E &= \text{Modulus of Elasticity of Concrete} &= 353553.39 \text{ Kg/cm}^2 \\ I &= \text{Moment of Inertia} &= 6.2.E+07 \text{ cm}^4 \\ \therefore \delta_1 &= \frac{5}{48} \times \frac{113413240}{353553.39} \times \frac{3952^2}{62113333} \\ &= 8.402075 \text{ cms} \\ &= 84.02075 \text{ mm} \end{aligned}$$

Upward deflection due to prestress

$$= \frac{P \times e \times L^2}{8 \times E \times I}$$

$$\begin{aligned} P &= \text{Prestressing Force at Mid Span} &= 1296.699 \text{ T} \\ e &= \text{eccentricity} &= 65.7 \text{ cms} \\ \therefore \delta_2 &= \frac{1.3E+06}{8} \times \frac{65.714286}{353553.39} \times \frac{3952^2}{62113333} \\ &= 7.5753576 \text{ cms} \\ &= 75.75 \text{ mm} \end{aligned}$$

$$\begin{aligned} \therefore \text{Net } \delta &= 84.02 - 75.75358 \\ &= 8.27 \text{ Downward} \end{aligned}$$

Permissible Deflection

$$= \frac{L}{800} = \frac{39520}{800} = 49.4 \text{ mm Hence O.K}$$



**Check for Ultimate Strength at Various X<sup>n</sup>:**

(Vide Cl:13 of I.R.C:-18-2000)

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

$$M_{ult(Steel)} = 0.9 d_b A_s f_p$$

A<sub>s</sub> = Area of High Tensile Steel

f<sub>p</sub> = The Ultimate Tensile Strength of Steel .

d<sub>b</sub> = The Depth of the beam from the maximum compression edge to C.G of Tendons.

(ii) Failure by crushing of concrete ( over reinforced section )

$$M_{ult(Conc)} = 0.176 b d_b^2 f_{ck} + (2/3) \times 0.8 (B_f - b) (d_b - t / 2) \times t \times f_{ck}$$

b = Total width of the Webs.

B<sub>f</sub> = Overall width of the top flange

t = Average thickness of flange.

$$UTS = 0.74$$

Sections	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
A <sub>s</sub> (m <sup>2</sup> )	0.00990	0.010	0.010	0.010	0.010
f <sub>p</sub> (T/m <sup>2</sup> )	186000	186000	186000	186000	186000
d <sub>b</sub> (m)	1.500	1.760	1.937	2.035	2.050
M <sub>ult(Steel)</sub> (Tm)	<b>2485.89</b>	<b>2916.58</b>	<b>3209.40</b>	<b>3373.02</b>	<b>3397.38</b>
b (m)	0.700	0.350	0.350	0.350	0.350
B <sub>f</sub> (m)	2.200	2.200	2.200	2.200	2.200
t (m)	0.550	0.550	0.550	0.550	0.550
M <sub>ult(Conc)</sub> (Tm)	<b>4081.00</b>	<b>4982.91</b>	<b>5663.49</b>	<b>6052.14</b>	<b>6110.54</b>
Min M <sub>Ult</sub>	<b>2485.89</b>	<b>2916.58</b>	<b>3209.40</b>	<b>3373.02</b>	<b>3397.38</b>
M <sub>Ultimate</sub>	<b>42.25</b>	<b>1054.91</b>	<b>1589.71</b>	<b>1874.44</b>	<b>2000.44</b>
Remarks	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>

**Check for Ultimate Shear Strength at Various X<sup>n</sup>:**

(Vide Cl:14.1.2 of I.R.C:-18-2000)

Sections Uncracked in flexure

$$V_{oo} = 0.67bd \sqrt{f_t + 0.8 f_{cp} + f_t}$$

V<sub>oo</sub> = Ultimate Shear Resistance of the X<sup>n</sup> .

b = Width of Webs - (2/3 x Duct Diametre) if the Cables are grouted.

d = Overall depth

f<sub>t</sub> = Max principal stress  $0.24 \sqrt{f_{ck}}$

f<sub>cp</sub> = Stress at c.g at the section due to prestress after inst: loss is accounted.

Sections Cracked in flexure

$$V_{cr} = 0.037bd_b \sqrt{f_{ck}} + (M_t \times V/M)$$

d<sub>b</sub> = Distance of extreme comp.fibre from centroid of tendons.

M<sub>t</sub> = (0.37  $\sqrt{f_{ck}} + 0.8 f_{pt}$ ) I/y

V and M = Ultimate Shear & corresponding moment at the section

$$V_{cr} (min) = 0.1bd \sqrt{f_{ck}}$$

Acc.to IRC :18 - 2000 Cl. No. 14.1.5 & Table 6.

V<sub>Capacity</sub> = ( **530** x b web x db ) + P Sin(θ) if the X<sup>n</sup> is Uncracked.

db = 0.8 x Overall Depth or Dist: from comp: face to C.G of Tendons, which ever is more.

Sections	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
$b$ (m)	0.5000	0.1500	0.1500	0.1500	0.1500
$d$ (m)	2.2000	2.2000	2.2000	2.2000	2.2000
$f_{cp}$ ( $T/m^2$ )	807.299	1104.04	1580.296	1710.094	1704.063
$f_t$ ( $T/m^2$ )	169.706	169.706	169.706	169.706	169.706
$V_{co}$ (T)	<b>222.7238</b>	<b>169.3343</b>	<b>199.2360</b>	<b>203.8136</b>	<b>199.5911</b>
$d_b$ (m)	1.500	1.76	1.937	2.035	2.050
$I$ ( $m^4$ )	0.621	0.496	0.496	0.496	0.496
$y = Vb$ (m)	1.100	1.06	1.057	1.057	1.057
$\sigma_t$ ( $T/m^2$ )	479.096	220.32	-298.758	-597.812	-641.560
$\sigma_b$ ( $T/m^2$ )	1142.460	2281.00	2792.488	3100.960	3140.531
$f_{pt}$ ( $T/m^2$ )	1142.460	2281.002	2792.488	3100.960	3140.531
$M_t$ (Tm)	663.821	978.273	1170.131	1285.839	1300.682
$M_{ult}$ (Tm)	42.255	1054.91	1589.706	1874.441	2000.441
$V_{ult}$ (T)	<b>355.753</b>	<b>259.299</b>	<b>184.077</b>	<b>108.380</b>	<b>83.856</b>
$V_{cr}$ (T)	5608.5	247.4	143.1	82.3	62.6
$V_{cr(min)}$ (T)	77.782	23.335	23.335	23.335	23.335
$\therefore V_{cr}$ (T)	<b>5609</b>	<b>247.37</b>	<b>143.093</b>	<b>82.334</b>	<b>63</b>
Remarks	<b>Cracked</b>	<b>Cracked</b>	<b>Uncracked</b>	<b>Uncracked</b>	<b>Uncracked</b>
$P \sin(\theta) T$	0.000	0.000	35.177	9.854	0.000
$b_{web}$	0.700	0.350	0.350	0.350	0.350
$db$ (m)	1.960	1.960	1.960	2.035	2.050
$V_{Capacity}$ (T)	<b>727.160</b>	<b>363.580</b>	<b>398.757</b>	<b>387.402</b>	<b>380.275</b>
Remarks	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>

**Check for Shear Reinforcement Requirement :-**

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

Sections	1 - 1	3 - 3	5 - 5	7 - 7	9 - 9
$V_{ult}$ (T)	355.7525	259.2989	184.0773	108.3796	83.8559
$V_c$ (T)	222.724	169.334	143.093	82.334	62.568
Spacing (mm)	150	150	150	200	200
$A_{sv}$ ( $cm^2$ )	4.78	3.23	1.47	1.25	1.02
$A_{sv}(cm^2)/Web$	<b>4.78</b>	<b>3.23</b>	<b>1.47</b>	<b>1.25</b>	<b>1.02</b>
$\phi$ of Bar(mm)	12	10	10	10	10
No of Legs	2	2	2	2	2
$A_{sv_{Prov}}$ ( $cm^2$ )	<b>15.080</b>	<b>10.472</b>	<b>10.472</b>	<b>7.854</b>	<b>7.854</b>
Remarks	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>

## EFFECTS OF DIFFERENTIAL SHRINKAGE

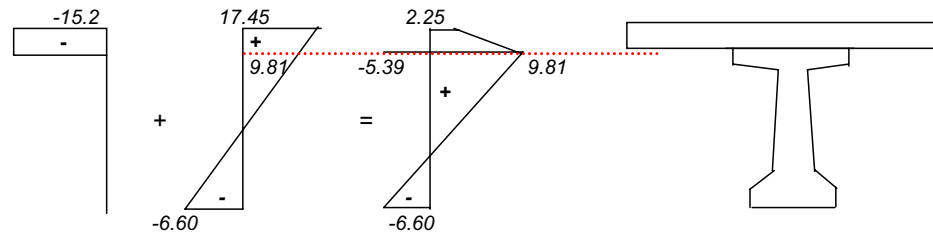
It is generally assumed that the time lag between transfer of prestress and casting of deck slab will be about 30 days.

Differential shrinkage strain, $\epsilon_{sh}$	=	<b>0.0001</b>
Force induced by differential shrinkage, $F = \eta \cdot A \cdot E \cdot \epsilon_{sh}$	where	$A = 0.55 \text{ m}^2$
Refer BS 5400, Chapter 8, page 110.		$E = 5000 \sqrt{50}$
		$= 35355 \text{ Mpa}$
		$= 3535533.91 \text{ Kg/cm}^2$
	$\eta =$	<b>0.43</b> (As per BS:5400)
		$= 83.6 \text{ T}$
Eccentricity of the above force from top of the composite section		$= 0.125 \text{ m}$
$Y_t$ (Distance of the NA of composite section from top)		$= 0.937 \text{ m}$
Eccentricity of the above force CG of the composite section		$= 0.812 \text{ m}$
Moment due to above force		$= 67.90 \text{ Tm}$
$Z_t$		$= 564666.67 \text{ cm}^3$
$Z_b$		$= 564666.67 \text{ cm}^3$
$Z_j$		$= 1549750.00 \text{ cm}^3$
$A$		$= 15400.00 \text{ cm}^2$

Stresses induced due to differential shrinkage

$\sigma_t =$	$\frac{83615.4}{15400.0} + \frac{6790333.7}{564666.7}$	$= 17.45 \text{ Kg/m}^2$
$\sigma_b =$	$\frac{83615.4}{15400.0} - \frac{6790333.7}{564666.7}$	$= -6.60 \text{ Kg/m}^2$
$\sigma_j =$	$\frac{83615.4}{15400.0} + \frac{6790333.7}{1549750.0}$	$= 9.81 \text{ Kg/m}^2$
Tensile stress in deck slab	$\frac{F}{A} = \frac{-83615.4}{5500.0}$	$= -15.2 \text{ Kg/m}^2$

Stresses due to differential shrinkage

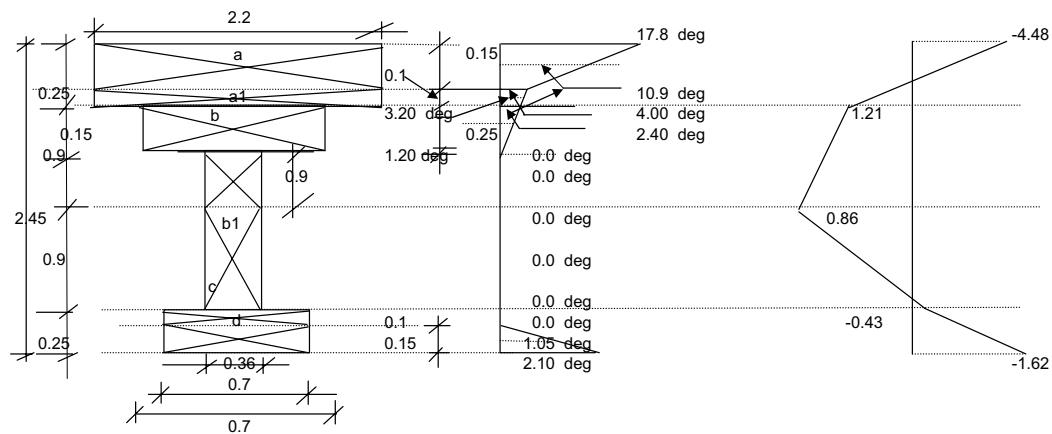


**Analysis for temperature stresses**

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

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

**Geometric section**



Concrete mix	=	M45
fck	=	45 Mpa
E concrete	=	$5000 \sqrt{fck}$ 3.35E+04
Coefficient of thermal expansion of concrete	=	1.17E-05 /deg C

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

Zone	Area (m <sup>2</sup> )	Y (m)	AY (m <sup>3</sup> )	AY <sup>2</sup> (m <sup>4</sup> )	t (Deg)	At	AY
a	0.33	0.075	0.02475	0.00185625	10.9 deg	3.597	0.269775
a1	0.22	0.2	0.044	0.0088	3.2 deg	0.704	0.1408
b	0.105	0.325	0.034125	0.011090625	1.20 deg	0.126	0.04095
b1	0.324	0.85	0.2754	0.23409	0.0 deg	0	0
c	0.324	1.75	0.567	0.99225	0.0 deg	0	0
d	0.07	2.1	0.147	0.3087	0.0 deg	0	0
d1	0.105	2.375	0.249375	0.592265625	1.05 deg	0.11025	0.26184375
Sum	1.48E+00		1.3417E+00	2.1490525		4.53725	0.71336875

For equilibrium

$$\epsilon_0 \cdot \Sigma A - \theta \Sigma AY = \alpha A t \quad (1)$$

$$\epsilon_0 \Sigma AY - \theta \Sigma AY^2 = \alpha A Yt \quad (2)$$

$$\begin{aligned} \epsilon_0 \cdot 1.48E+00 - \theta \cdot 1.34E+00 &= 1.17E-05 \cdot x \cdot 4.54 \\ \epsilon_0 \cdot 1.34E+00 - \theta \cdot 2.15E+00 &= 1.17E-05 \cdot x \cdot 0.71 \end{aligned}$$

$$\begin{aligned} \epsilon_0 \cdot x \cdot 1.00 - \theta \cdot 0.90774696 &= 3.59E-05 \quad (3) \\ \epsilon_0 \cdot x \cdot 1.00 - \theta \cdot 1.60179816 &= 6.22E-06 \quad (4) \end{aligned}$$

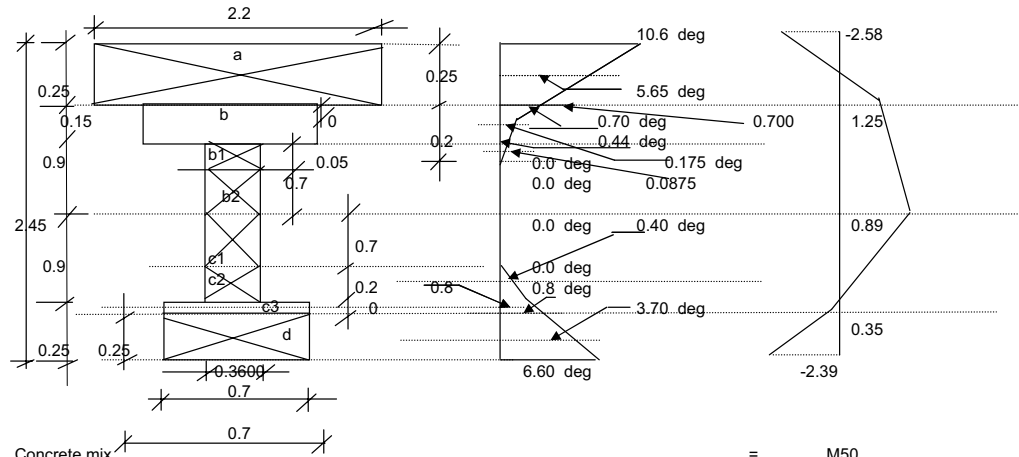
$$\begin{aligned} \theta \cdot 0.6940512 &= 2.97E-05 \\ \theta &= 4.28E-05 \\ \epsilon_0 &= 7.48E-05 \end{aligned}$$

**Calculation of thermal stresses**

Sl No.	Y (m)	Yθ	t	α t	f <sub>ci</sub> = Ec (ε <sub>0</sub> - Yθ - α t)
1	0.00E+00	0.00E+00	17.80 deg	2.08E-04	-4.48E+00
2	0.25	1.07E-05	2.40 deg	2.808E-05	1.21E+00
3	1.15	4.92E-05	0.00 deg	0.00E+00	8.57E-01
4	2.05	8.77E-05	0.00 deg	0.00E+00	-4.35E-01
5	2.3	9.84E-05	2.1 deg	2.46E-05	-1.62E+00

**Calculation of eigen stresses (Reverse temperaturure difference)**

**Equivalent I - section for running section**



Concrete mix = M50  
 fck = 50 Mpa  
 E concrete =  $5000 \sqrt{fck}$  = 3.54E+04  
 Coefficient of thermal expansion of concrete = 1.17E-05 /deg C

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

Zone	Area (m <sup>2</sup> )	Y (m)	AY (m <sup>3</sup> )	AY <sup>2</sup> (m <sup>4</sup> )	t (Deg)	At	AtY
a	0.55	0.125	0.06875	0.00859375	5.65 deg	3.1075	0.3884375
b	0.105	0.325	0.034125	0.011090625	0.438 deg	0.0459375	0.014929688
b1	0.018	0.425	0.00765	0.00325125	0.0875 deg	0.001575	0.000669375
b2	0.252	0.8	0.2016	0.16128	0.0 deg	0	0
c1	0.252	1.5	0.378	0.567	0.0 deg	0	0
c2	0.072	1.95	0.1404	0.27378	0.40 deg	0.0288	0.05616
c3	0	2.05	0	0	0.80 deg	0	0
d	0.175	2.325	0.406875	0.945984375	3.7 deg	0.6475	1.5054375
Sum	1.42E+00		1.2374E+00	1.97098		3.8313125	1.965634063

For equilibrium

$$\epsilon_0 \cdot \Sigma A - \theta \Sigma AY = \alpha A t \quad (1)$$

$$\epsilon_0 \cdot \Sigma AY - \theta \Sigma AY^2 = \alpha A Yt \quad (2)$$

$$\begin{array}{l} \epsilon_0 \quad 1.42E+00 - \theta \quad 1.24E+00 = \quad 1.17E-05 \quad \times 3.83 \\ \epsilon_0 \quad 1.24E+00 - \theta \quad 1.97E+00 = \quad 1.17E-05 \quad \times 1.97 \end{array}$$

$$\begin{array}{l} \epsilon_0 \quad \times 1.00 - \theta \quad 0.86896067 = \quad 3.15E-05 \quad (3) \\ \epsilon_0 \quad \times 1.00 - \theta \quad 1.59283983 = \quad 1.86E-05 \quad (4) \end{array}$$

$$\begin{array}{l} \theta \quad 0.72387915 = \quad 1.29E-05 \\ \theta \quad = \quad 1.78E-05 \\ \epsilon_0 \quad = \quad 4.70E-05 \end{array}$$

Calculation of thermal stresses

Sl No.	Y (m)	Y $\theta$	t	$\alpha t$	fci = Ec ( $\epsilon_0 - Y \theta - \alpha t$ )	
1	0.00E+00	0.00E+00	10.60 deg	1.24E-04	-2.58E+00	
2	0.25	4.45E-06	0.44 deg	5.119E-06	1.25E+00	
3	1.15	2.05E-05	0.00 deg	0.00E+00	8.88E-01	
4	2.05	3.65E-05	0.00 deg	0.00E+00	3.50E-01	
5	2.3	4.10E-05	6.6 deg	7.72E-05	-2.39E+00	

**Design for Bursting Tensile Force :**

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

$$\begin{aligned} 2 Y_o &= \text{Width of End Block} &= & 800 \text{ mm} \\ \therefore Y_o & &= & 400 \text{ mm} \\ 2 Y_{Po} &= \text{Width of Bearing Plate} &= & 375 \text{ mm assumed} \\ \therefore Y_{Po} & &= & 187.5 \text{ mm} \\ P_k &= \text{Force at anchorage} &= & 454.212 \text{ T} \\ \frac{Y_{Po}}{Y_o} &= \frac{187.5}{400} &= & 0.469 \\ \frac{F_{bst}}{P_k} &= 0.179375 \text{ (From Table- 8) for corresponding } \frac{Y_{Po}}{Y_o} \\ \therefore F_{bst} &= 0.1794 \times 454.212 &= & 81.474 \text{ T} \end{aligned}$$

Calculation of area of steel required.

Permissible tensile stress in H.Y.S.D steel

$$\begin{aligned} &= 0.87 \times 4150 &= & 3610.5 \text{ Kg/cm}^2 \\ \text{Ast reqd} &= \frac{81.4742775 \times 1000}{3610.5} &= & 22.566 \text{ cm}^2 \end{aligned}$$

Provide **16** mm dia bars, **4** nos in **3** layers at the front face of anchoring cone.

$$\text{Ast prov} = 24.127 \text{ cm}^2 \quad \text{Safe}$$

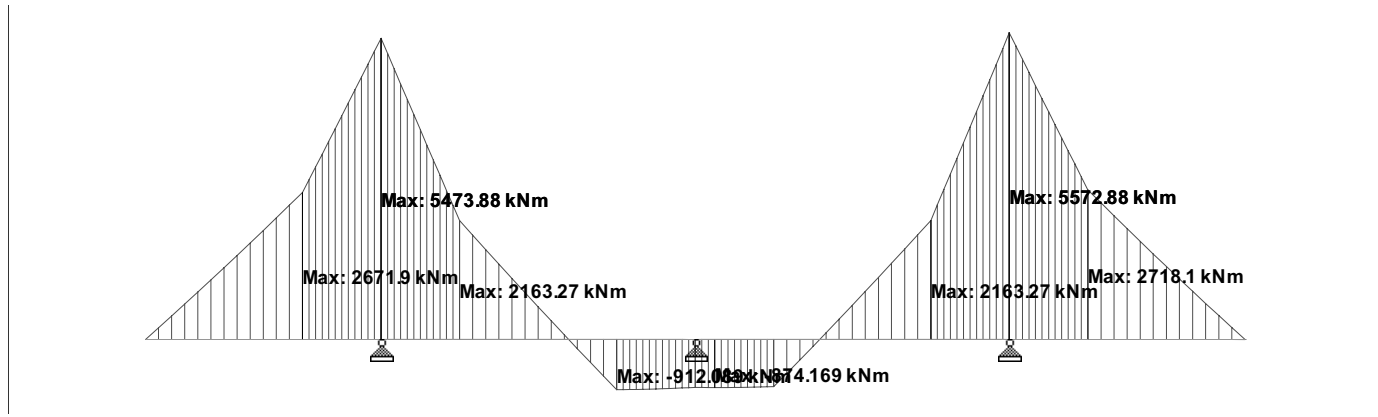


## **Appendix D-2: Super Structure Design - Cross Girder**

```
STAAD PLANE cross-girder
START JOB INFORMATION
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
joint coordinates
1 0 0 0; 2 2.2 0 0; 3 3.3 0 0; 4 4.4 0 0, 5 6.6 0 0
6 7.7 0 0; 7 8.8 0 0; 8 11 0 0; 9 12.1 0 0; 10 13.2 0 0;
11 15.4 0 0;
member incidences
1 1 2 10
member property indian
1 to 10 pri ax 2.56 ix 0.001 iz 0.57
constants
e concrete all
poi concrete all
supports
3 6 9 pinned
load 1
member load
*****cross girder self weight
1 to 10 uni gy -55
*****girder load
joint load
1 2 4 5 7 8 10 11 fy -531
*****slab load
1 11 fy -262
2 4 5 7 8 10 fy -326
***SIDL
1 fy -216
2 fy -118
4 fy -88
5 fy -92
7 fy -92
8 fy -90
10 fy -118
11 fy -217
****live load
1 fy -145
2 fy -267
4 fy -576
5 fy -395
7 fy -320
8 fy -689
10 fy -294
11 fy -165
perform analysis
PRINT MAXFORCES ENVELOPE
print member forces
print support reactions
print displacements
FINISH
```

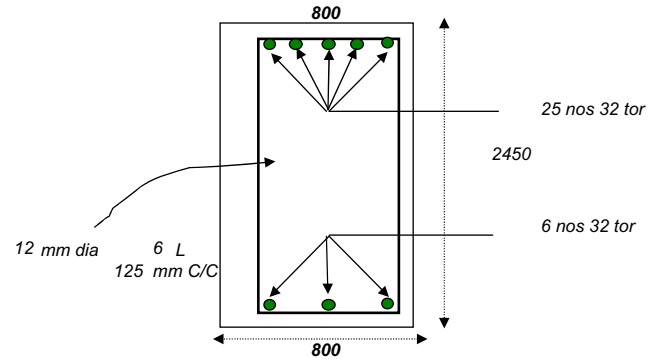


Cross girder – Analysis model



BMD-Cross girder

**DESIGN OF END CROSS GIRDER**



Design forces are taken from STAAD

SECTION	1	2	BM	
<b>SUM</b>	554.00	-134.00	+ve	554 Tm
<b>SUM</b>	269.00		-ve	134 Tm
Torsional moment	0		Shear	269 T

**NOTE :-**  
 + ve = Hogging  
 - ve = Sagging

Grade of Concrete =	<b>500</b>	
eff. depth reqd. (mm) =	1479	727
eff. depth provided (mm) =	2384	2384
Total depth provided (mm) =	<b>2450</b>	<b>2450</b>
Dia of bar provided (mm)	<b>32</b>	<b>32</b>
No of bars reqd.	25	6
Shear Stress Kg/cm <sup>2</sup> =	14.10	
Allow Shear Stress Kg/cm <sup>2</sup> =	4.04	
<b>Safe</b>		
<b>Shear Reinf Provided</b>		
Dia of Bar provided	<b>12</b>	
No of legs	<b>6</b>	
Spacing	125	

**Check for effective depth at section 1-1**

Moments due to		Shear due to	25.68969491
Total Moment	=	Total Shear	=
	<b>554.00 Tm</b>		<b>269.00 T</b>
Check for eff. Depth			
deff	=	$\sqrt{\frac{554.000}{31.67} \times \frac{100000}{80}}$	Bar Size =
	=	<b>147.86 cms</b>	<b>32 mm</b>
Depth provided at 1-1	=	<b>2450 mm</b>	Cover =
deff prov:	=	<b>238.4 cms</b>	<b>50 mm</b>
Ast Reqd	=	$\frac{554.000}{2000} \times \frac{100000}{1.00} \times 139.5$	<b>Safe</b>
	=	<b>198.57 cm<sup>2</sup></b>	
No. of bars reqd.	=	<b>25</b>	

**Check for effective depth at section 2 - 2**

Moments due to

Total Moment = 134.00 Tm

Shear due to

Total Shear = 269.00 T

Check for eff. Depth

$$\sqrt{= \frac{134.000}{31.67} \times \frac{100000}{80}}$$

$$= 72.72 \text{ cms}$$

$$= 2450 \text{ mm}$$

Bar Size = 32 mm

Cover = 50 mm

Depth provided at 2-2

= 238.4 cms

Ast Reqd

$$= \frac{134.000}{2000} \times \frac{100000}{1.00 \times 139.5} = 48.03 \text{ cm}^2$$

Safe

No. of bars reqd.

= 6

2) Maximum Permissible Shear Stress :-

Shear stress ,

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

V = The design shear across the section

d = Effective depth of the section

B = Breadth of slab

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

Maximum Permissible Shear Stress :-

$$\tau_{\text{max}} = 2.50 \text{ Mpa} \quad (\text{Vide cl - 304.7.1.2 of I.R.C:-21-2000})$$

for M40 and above concret grade

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

Calculation of permissible Shear Stress :-

$$\frac{100 A_s}{bd} = 1.307 \quad (\text{Vide cl - 304.7.1.3.3 of I.R.C:-21-2000, Table -12B})$$

$\therefore$  Permissible shear stress in concrete corresponding to

1.307239896 value from table is 4.04 Kg/cm<sup>2</sup>

$$\therefore \tau_c = 4.04 \text{ Kg/cm}^2$$

Required

3) Shear Reinforcement Reqd :-

( Vide cl - 304.7.1.4 of I.R.C:-21-2000 )

Shear reinforcement is provided to carry a shear equivalent to

Design shear  $V_s = (V - \tau_c bd) = (269.0 - 77.05088) T = 191.9 T$

$$A_{sw} = \frac{V}{\sigma_{st}} \times \frac{s}{d}$$

$$= \frac{269.00}{T} \times \frac{s}{d}$$

$$s, \text{ spacing} = 125 \text{ mm} \quad (\text{assumed})$$

$$\therefore A_{sw} = \frac{191.95}{2000} \times \frac{1000}{238.4} \times 12.5 = 5.0 \text{ cm}^2$$

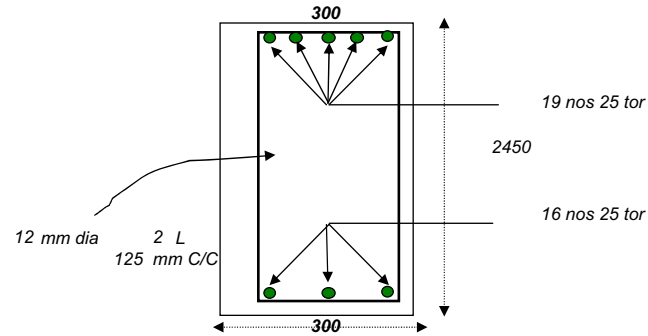
Using 12 mm dia stirrups 6 Legged at spacing 125 mm.

Asw = 6.8 cm<sup>2</sup> > 5.0 cm<sup>2</sup>

Hence O.K

**DESIGN OF INTERMEDIATE DIAPHRAGM:-**

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



SECTION	1	2
<b>SUM</b>	153.00	125.00
<b>SUM</b>	47.00	
Torsional moment	0	

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

Grade of Concrete =	0	
eff. depth reqd. (mm) =	1269	1147
eff. depth provided (mm) =	2090	2090
Total depth provided(mm) =	<b>2150</b>	<b>2150</b>
Dia of bar provided (mm)	<b>25</b>	<b>25</b>
No of bars reqd.	19	16
Shear Stress Kg/cm <sup>2</sup> =	7.50	
Allow Shear Stress Kg/cm <sup>2</sup> =	4.20	
<b>Safe</b>		
<b>Shear Reinf Provided</b>		
Dia of Bar provided	<b>12</b>	
No of legs	<b>2</b>	
Spacing	125	

**Check for effective depth at section 1-1**

Total Moment = 153.00 Tm      Total Shear = 47.00 T

Check for eff. Depth

$$deff = \sqrt{\frac{153.000}{31.67} \times \frac{100000}{30}}$$

= 126.89 cms

Depth provided at 1-1 = 2150 mm

deff prov: = 209 cms

Ast Reqd =  $\frac{153.000}{2000} \times \frac{100000}{1.00 \times 130.5} = 58.62 \text{ cm}^2$

Bar Size = 20 mm  
 Cover = 50 mm

No. of bars reqd. = 19

**Check for effective depth at section 2 - 2**

Total Moment = 125.00 Tm      Total Shear = 47.00 T

Check for eff. Depth

$$\sqrt{\frac{125.000}{31.67} \times \frac{100000}{30}} = 114.69 \text{ cms}$$

Bar Size = 20 mm  
Cover = 50 mm

Depth provided at 2-2

= 2150 mm

deff prov:

= 209 cms

Safe

Ast Reqd

$$= \frac{125.000}{2000} \times \frac{100000}{1.00 \times 130.5} = 47.89 \text{ cm}^2$$

No. of bars reqd. = 16

2) Maximum Permissible Shear Stress :-

Shear stress ,

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

V = The design shear across the section

d = Effective depth of the section

B = Breadth of slab

$$\therefore \tau = \frac{47.000}{30} \times \frac{1000}{209} = 7.4960 \text{ Kg/cm}^2$$

Maximum Permissible Shear Stress :-

$$\tau_{max} = 2.50 \text{ Mpa} \quad (\text{Vide cl - 304.7.1.2 of I.R.C:-21-2000})$$

for M40 and above concret grade

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

Calculation of permissible Shear Stress :-

$$\frac{100 A_s}{bd} = 2.740$$

(Vide cl - 304.7.1.3.3 of I.R.C:-21-2000, Table -12B)

∴ Permissible shear stress in concrete corresponding to

2.740125171 value from table is 4.2 Kg/cm<sup>2</sup>

$$\therefore \tau_c = 4.2 \text{ Kg/cm}^2 \quad \text{Required}$$

3) Shear Reinforcement Reqd :-

(Vide cl - 304.7.1.4 of I.R.C:-21-2000)

Shear reinforcement is provided to carry a shear equivalent to

Design shear  $V_s = (V - \tau_c bd) = (47.0 - 26.334) T = 20.7 T$

$$A_{sw} = \frac{V_s}{\sigma_{st} \times \frac{s}{d}}$$

$$V_s = 20.7 T \quad (\text{assumed})$$

$$\therefore A_{sw} = \frac{20.67}{2000} \times \frac{1000}{209.0} \times 12.5 = 0.6 \text{ cm}^2$$

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

$$A_{sw} = 2.3 \text{ cm}^2 > 0.6 \text{ cm}^2$$

Hence O.K

## **Appendix D-3: Design of Substructure & Foundation**



### Design of Trestle Abutment and Pile Foundation

#### Design Data for Substructure:-

Type	Simply supported Precast Prestressed Girders and RCC slab		
C/C Distance bet <sup>n</sup> piers	=	40000	mm
Carriageway Width	=	7500	mm
Overall Width of Deck.	=	17000	mm
Width of Crash Barrier	=	500	mm
Height of Crash Barrier	=	1050	mm
No of Bearings	=	3	
Overall Depth of I.Girders	=	2200	mm
Depth of Deck Slab	=	250	mm

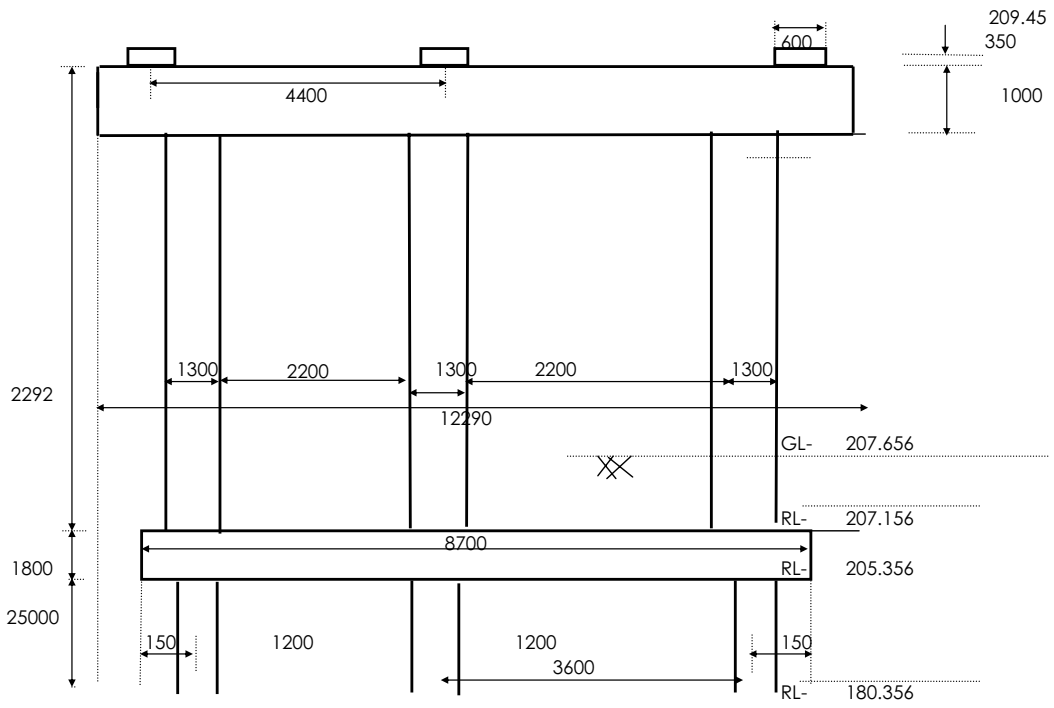
Both sides

#### Design Data for Pier :-

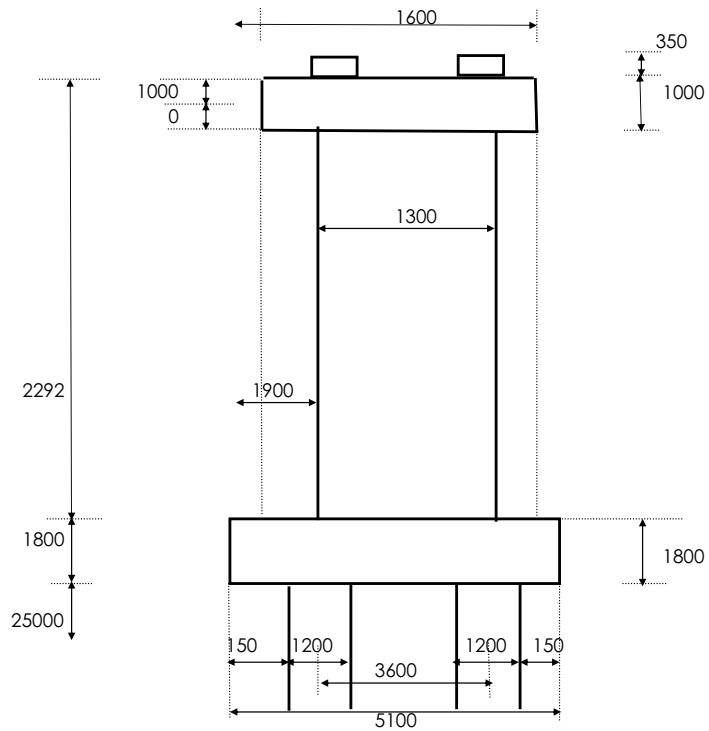
Formation level at abutment locc	=	212.600	m
R.L at abutment cap top	=	209.448	m
Existing Road level	=	207.656	m
Pile cap top below existing road	=	500	mm
R.L at Pile cap top	=	207.16	m
R.L at Pile cap bottom	=	205.36	m
Depth of Pile below pilecap	=	25000	mm
Founding level for Piles.	=	180.36	m
Overall Heighth of Substructure	=	2292	mm
Diameter of pier	=	1300	mm
Transverse width of pier	=	1300	mm
No of piers	=	3	
C/C distance between piers	=	3500	mm
Pier Cap Width in Long Dir <sup>n</sup> .	=	1600	mm
Pier Cap Length in Trans Dir <sup>n</sup> .	=	12290	mm
Straight Depth of Pier Cap	=	1000	mm
Type of Bearing	=	POT PTFE BEARING	
Size of Pedestals	=	600	x 600 x 350
Distance betn Pedestals	=	4400	mm
Longitudinal width of pile cap	=	5100	mm
Transverse width of pile cap	=	8700	mm
Straight Depth of pile cap	=	1800	mm
Varying Depth of pile cap	=	0	mm
P.C.C Projections	=	150	mm
Diameter of Pile	=	1200	mm
Distance betn Piles in	=	3600	mm
Distance betn Piles in transverse	=	3600	mm
dirn	=	3600	mm
No of Piles	=	6	
Edge projection in longitudinal	=	150	mm
dirn	=	150	mm
Edge projection in transverse	=	150	mm
dirn	=	150	mm
Grade of Concrete	=	<b>M</b> 50	
Permissible flexural stress	=	16.67	N/mm <sup>2</sup>
Grade of Steel	=	Fe - 415	
Permissible tensile stress	=	200	N/mm <sup>2</sup>
Total Height of Pier	=	1292	mm
Density of Concrete	=	24	kN/m <sup>3</sup>
Density of Concrete for PSC Girde	=	25	kN/m <sup>3</sup>

#### References.

- I.R.C :- 6 - 2000 - Loads & Stresses.  
 I.R.C :- 21 - 2000 - Permissible Stresses  
 I.R.C :- 78 - 2000 - Sub Structure and Foundation



**Transverse View**



**Longitudinal View**

**Load Calculations:-**

**1) DEAD LOADS**

Total Load = 7184.00 kN

**2a) SIDL**

Wearing coat load = 429.00 kN

Median = 144.00 kN

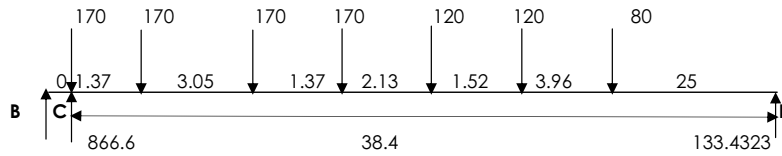
Crash barrier load = 249.60 kN

SIDL = 822.60 kN

**3) LIVE LOAD**

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

**70 R**



$$R_C = (170 \times 38.4 + 170 \times 37.03 + 170 \times 33.98 + 170 \times 32.61 + 120 \times 30.48 + 120 \times 28.96 + 80 \times 25) / 38.4$$

$$R_C = 866.5677 \text{ kN}$$

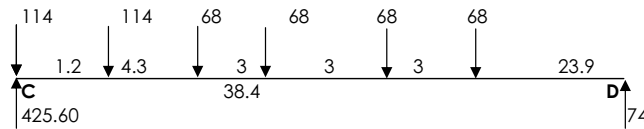
$$R_D = 170 + 170 + 170 + 170 + 120 + 120 + 80 - 866.568$$

$$R_D = 133.432 \text{ kN}$$

**70 R one lane**

**Class A One lane**

$$= 866.57 \text{ kN}$$



$$R_C = (114 \times 38.4 + 114 \times 37.2 + 68 \times 32.9 + 68 \times 29.9 + 68 \times 26.9 + 68 \times 23.9) / 38.4$$

$$R_C = 425.60 \text{ kN}$$

$$R_D = 74.40 \text{ kN}$$

Reaction at pier Support = 425.60 kN

Class A One lane = 425.60 kN

10% reduction of live load is considered for three lane as per Clause 208

Class 70R = 866.57 kN

Class A two lane = 851.21 kN

Critical live load = 866.57 kN

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

**Calculation of Dead Load from Sub - Structure :-**

**1) From Pedestals**

Volume of 1 Pedestal = 0.6 x 0.6 x 0.35

= 0.126 m<sup>3</sup> Rows

No: of Pedestals = 3 x 1 = 3 Nos

∴ Total Volume = 0.126 x 3 = 0.38 m<sup>3</sup>

∴ Total Load = 0.378 x 24 = 9.07 kN

**2) From Bed - Block :**

= ( 1.6 x 1 ) = 1.6 m<sup>2</sup>

Total C/S Area in longitudinal Dir<sup>n</sup>

Width in the Transverse Dir<sup>n</sup> = 12.29 m

∴ Total Volume = 1.60 x 12.29 = 19.66 m<sup>3</sup>

∴ Total Load = 19.66 x 24 = 471.94 kN

**3) From Pier :-**

Diameter of pier = 1.3 m

Area of pier = 1.327 m<sup>2</sup>

Ht: of Pier = 1.292 m

∴ Total Volume = 1.327 x 1.3 = 1.714 m<sup>3</sup>

No of pier = 3

∴ Total Load = 1.714 x 3 x 24 = 123.43 kN

4) From Pile Cap :-  
Volume of footing = ( 5.1 x 8.7 x 1.8 )  
= 79.866 m<sup>3</sup>  
∴ Dead Load = 79.866 x 24 = 1916.78 kN

5) From Pile :-  
Diameter of Pile = 1.2 m  
Depth of Pier with Circular X<sup>n</sup> = 25 m  
C/S Area of Circular Pier = 1.131 m<sup>2</sup>  
No of Piles = 6  
∴ Total Volume of Concrete = 169.646 m<sup>3</sup>  
∴ Total Load = 169.646 x 24 = 4071.50 kN

5) From Dirt Wall :-  
Volume of dirt wall = 3.152 x 12.0 x 0.30  
= 11.349 m<sup>3</sup>  
∴ Total Load = 11.349 x 24 = 272.376 kN

Options

**A) Both Spans on.**

**Case (i)**

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

1) **Due to Braking**

b) **Braking.**

( Vide :- cl 214.2 (a) & (b) of I.R.C : 6 - 2000 . )

Braking. Since the movement of bearing under the girders on one side is restricted to move in the longitudinal direction half the effect of braking is considered in the design.

1) 20 % of I<sup>st</sup> Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.

2) 20 % of I<sup>st</sup> Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.  
+ 5 % of Loads on the lanes exceeding Two.

3 Lanes of Class A Wheeled Vehicles.

Total Load of 1 Vehicle = 554 kN one span

∴ Braking Force = 110.8 kN

1 Lane of Class 70 R Wheeled Vehicles.+ Class A one lane.

Total Load of 70R Vehicle = 1000 kN one span

Total Load of CI A Vehicle = 554 kN

∴ Braking Force = 200 kN

Max Braking Force = 200 kN

Vertical reaction due to braking

$$= \frac{200(1.2+0.075+2.45)}{(40-0.6)}$$

$$= 17.5888 \text{ kN}$$

Longitudinal moment due to vertical load of braking

Longitudinal Eccentricity = 0.45

Moment due to long.Eccentricity = 7.91497 kNm

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

(ii)  $\frac{F_h}{2} + m (R_g + R_q)$  = 543.66 kN  
Total Force = 543.66 kN  
∴ Moment at Pile Cap top = 544 x 3.69 = 2006.91 kNm  
Moment at Pile Cap bottom = 544 x 5.49 = 2985.50 kNm

3) **Moment due to Longitudinal Eccentricity**

Longitudinal Eccentricity	=	0.80	m		
		Normal	Seismic		
		Case	Case		
Due to DL	=	0	0	kNm	kNm
Due to SIDL	=	0	0	kNm	kNm
Due to LL	=	693	347	kNm	kNm
Moment	=	693	347	kNm	kNm

**Due to Live Load**

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

**Class 70R 1 lane**

$$\text{Moment in Transverse Dir}^{\circ} = 4683.80 \text{ kNm}$$

**Class A 2 lane.**

$$\text{Moment in Transverse Dir}^{\circ} = 5778.09 \text{ kNm}$$

**70 R One lane- both carriage ways**

$$\text{Moment in Transverse Direction} = 2001.77 \text{ kNm}$$

**Class A two lane-both carriage ways**

$$\text{Moment in Transverse Direction} = 1634.21 \text{ kNm}$$

$$\text{Critical Moment in Transverse Direction} = 5778.09 \text{ kNm}$$

Axial Load

$$\text{Pile Cap Top} = 7184 + 822.6 + 866.57 + 9.08 + 471.94 + 123.43 + 17.59$$

$$\text{Pile Cap Bottom} = 7184 + 822.6 + 866.57 + 9.08 + 471.94 + 123.43 + 1916.79$$

Longitudinal moment

$$\text{Pile Cap Top} = 2006.92 + 693.26$$

$$\text{Pile Cap Bottom} = 2985.51 + 693.26$$

Transverse moment

$$\text{Pile Cap Top} = 5778.1$$

$$\text{Pile Cap Bottom} = 5778.1$$

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	9750.0	11666.8
$M_L$ (Tm)	2700.2	3678.8
$M_T$ (Tm)	5778.1	5778.1

**C) Both Spans on Under Seismic in Longitudinal Direction.****Case (ii)**

As per Modified Clause 222

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

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

$$A_h = \text{Horizontal Seismic coefficient}$$

$$= \left( \frac{Z}{2} \right) \left( \frac{S_a}{g} \right) I$$

$$\text{Zone No} = \text{IV}$$

$$\text{Zone Factor, } Z = 0.24$$

$$S_a/g = \text{Average acceleration coefficient}$$

$$\text{Response Modification factor, } R = 3.3$$

$$\text{Importance Factor, } I = 1.2$$

$$T = 2 \sqrt{\frac{D}{1000 F}}$$

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

$$= 8006.60 \text{ kN}$$

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

$$F = \left( \frac{3 EI \delta}{l^3} \right)$$

$$\text{Modulus of Elasticity of concrete, } E = 3.54E+07 \text{ kN/m}^2$$

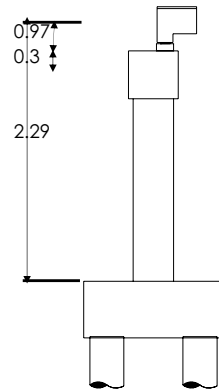
$$I_{xx} = 0.42 \text{ m}^4$$

$$\text{Deflection, } \delta = 0.001 \text{ m}$$

Distance from bottom to the centre of mass of super structure, y

$$= 0.97 \text{ m}$$

Height of substructure, h = 2.29 m  
 Total height measured from the centre of mass of super structure, l = 3.56 m



F = 990.85 kN  
 T = 0.18  
 Soil type = II  
 Sa/g = 2.50  
 A<sub>h</sub> = 0.36  
 F<sub>eqx</sub> = 2882.38 kN  
 Design of seismic force in longituc = F<sub>eqz</sub> / R = 873.45 kN

**Seismic force in transverse direction**

The seismic force due to live load shall be considered when acting in the direction perpendicular to traffic  
 The horizontal seismic force in the direction perpendicular to traffic shall be computed by taking 20% of live load (excluding impact factor)

Dead load and appropriate live = 8179.91 kN  
 Feq = A<sub>h</sub> (Dead load + Appropriate Live Load)  
 I<sub>yy</sub> = 0.42 m<sup>4</sup>  
 F = 990.85 kN  
 T = 0.18  
 Soil type = II  
 Sa/g = 2.50  
 A<sub>h</sub> = 0.36  
 F<sub>eqx</sub> = 2944.77 kN  
 Design Seismic force in trans dirn, l = 892.35 kN  
 Seismic vertical component, V<sub>f</sub> = 594.90 kN      2/3 \* 892.36  
 Resultant force in longi dirn = 1319.62 kN      DF<sub>eqz</sub>+0.3 DF<sub>eqx</sub>+ 0.3V<sub>f</sub>  
 CG of loads = 3.492861 m  
 Moment, M<sub>eqz</sub> at Pile Cap top = 4609.26 kNm  
 Moment, M<sub>eqx</sub> at Pile Cap bottom = 6984.59 kNm  
 Seismic vertical load ( Acting downwards-considering critical condition)  
 At pile cap top = 1124.64      0.3DF<sub>eqz</sub>+0.3 DF<sub>eqx</sub>+ V<sub>f</sub>  
 At pile cap bottom = 1405.80      (0.3DF<sub>eqz</sub>+0.3 DF<sub>eqx</sub>+ V<sub>f</sub>) \* 1.25  
 Axial load  
 Pile Cap Top = 7184+822.6+866.57 x 0.5+9.08+471.936+123.43  
 Pile Cap Bottom = 7184+822.6+866.57 x 0.5+9.08+471.936+123.43 + 1916.79

**Longitudinal moment**

Pile Cap Top = ( 100 x 0.5 + 46.8 ) x 2 x 3.692 +4609.26+ 346.627  
 Pile Cap Bottom = ( 100 x 0.5 + 46.8 ) x 2 x 5.49 +6984.58+ 346.627

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

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	10441.34	12639.28
M <sub>L</sub> (Tm)	6698.26	9923.17
M <sub>T</sub> (Tm)	2889.05	2889.05

**D) Both Spans on Under Seismic in Transverse Direction.****Case ( iii )**

Design Seismic force in trans

$$\text{dirn, } D_{F_{eqx}} = 892.35 \text{ kN}$$

$$\text{Resultant force in trans dirn} = 1332.859 \text{ kN}$$

$$\text{CG of loads} = 3.492861 \text{ m}$$

$$\text{Moment, } M_{eqx} \text{ at Pile Cap top} = 4655.492 \text{ kN}$$

$$\text{Moment, } M_{eqx} \text{ at Pile Cap} = 7054.638 \text{ kN}$$

Axial load is same as that of Case iii

Longitudinal moment

$$\text{Pile Cap Top} = (100 \times 0.5 + 46.8) \times 2 \times 3.7 + 4655.49$$

$$\text{Pile Cap Bottom} = (100 \times 0.5 + 46.8) \times 2 \times 5.49 + 7054.63$$

Transverse moment

$$\text{Pile Cap Top} = 5778.09 \times 0.5 + 4655.5$$

$$\text{Pile Cap Bottom} = 5778.09 \times 0.5 + 7054.64$$

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

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	10441.34	12639.28
$M_L$ (Tm)	6744.49	9993.22
$M_T$ (Tm)	7544.54	9943.68

**G) Service condition with Wind in Transverse direction****Case ( iv )**

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

$$\text{Height of the exposed surface above ground level} = 4.94 \text{ m}$$

$$\text{Exposed depth of C/Barrier \& Superstructure} = 2.45 \text{ m} + 1.05$$

$$= 3.50 \text{ m}$$

**Due to crash Barrier**

$$\text{Avg Height of Crash barrier from GL} = 5.47 \text{ m}$$

$$\text{Intensity of Wind pressure corres : to height} = 463.70 \text{ Kg/m}^2$$

$$\text{Average Exposed Length} = 40.00 \text{ m}$$

$$\text{Effective area of crash barrier} = 42.00 \text{ m}^2$$

$$\text{Force} = P_z A_1 G C_d$$

$$G = 2$$

$$C_d = 1.30$$

$$\text{Force} = 506.36 \text{ kN}$$

**Due to Deck Slab and girder**

$$\text{Average ht of deck slab and girder from GL} = 2.67 \text{ m}$$

$$\text{Intensity of Wind pressure corres : to height} = 463.70 \text{ Kg/m}^2$$

$$\text{Effective area of deck slab + girder} = 98.00 \text{ m}^2$$

$$\text{Force} = P_z A_1 G C_d \text{ kN}$$

$$G = 2$$

$$C_d = 1.95$$

$$\text{Force} = 177.23 \text{ kN}$$

$$\text{Live Load}$$

$$\text{Effective length} = 40.00 \text{ m}$$

$$\text{Depth} = 1.30 \text{ m}$$

$$\text{Height of the exposed surface above ground level} = 6.44 \text{ m}$$

$$\text{Area} = 52.00 \text{ m}^2$$

$$\text{Pressure} = 463.70 \text{ N/m}^2$$

$$\text{Force} = P_z A_1 G C_d$$

$$G = 2$$

$$C_d = 1.20$$

$$\text{Force} = 57.87 \text{ kN}$$

**Due to Pier**

$$\text{Average ht of pier and pier cap from GL} = 1.15 \text{ m}$$

$$\text{Intensity of Wind pressure corres : to height} = 463.70 \text{ kg/m}^2$$

$$\text{Effective area of pier and pier cap} = 1.60 \text{ m}^2$$

$$\text{Force} = P_z A_1 G C_d \text{ kN}$$

$$G = 2$$

$C_d$	=	0.50	
Force	=	0.74 kN	
Total transverse force	=	742.20 kN	
Transverse Moment due to wind			
Pile Cap Top	=	3615.62 kNm	
Pile Cap Bottom	=	4017.76 kNm	
Longitudinal Force			
Crash barrier	=	25 % of trans force	
	=	126.59	
Superstructure	=	25 % of trans force	
	=	44.31 kN	
On Live load	=	25 % of trans force	
	=	14.47 kN	
Substructure	=	25 % of trans force	
	=	0.19	
Total longitudinal force	=	185.55 kN	
Longitudinal Moment due to wind			
Pile Cap Top	=	829.48 kNm	
Pile Cap Bottom	=	1237.89 kNm	
Vertical load	=	$P_z A_3 G C_L$	
	$P_z$	=	463.70 N/m <sup>2</sup>
	$A_3$	=	680.00
	$G$	=	2.00
	$C_L$	=	0.75
		=	472.97 kN      Acting upwards or downwards

Axial loads and Longitudinal Moments are same as Case 1

**Summary of Axial Loads & Moments**

Descriptions	Pilecap Top	Pile cap Bottom
Axial Load	10222.95	12139.73
$M_L$ (Tm)	3529.65	4916.65
$M_T$ (Tm)	9393.71	9795.85

**Case ( iv b )**

Descriptions	Pilecap Top	Pile cap Bottom
Axial Load	9277.00	11193.79
$M_L$ (Tm)	3529.65	4916.65
$M_T$ (Tm)	9393.71	9795.85

**I) Effect of collision in longitudinal direction**

**Case v**

**Axial load**

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

**Longitudinal moment**

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

**Transverse moment**

Pile Cap Top	=	0.00 kNm
Pile Cap Bottom	=	0.00 kNm

**Summary of Axial Loads & Moments**

Descriptions	Footing top	Footing bottom
Axial Load	8611.03	10527.82
$M_L$ (Tm)	1000.00	1900.00
$M_T$ (Tm)	0.00	0.00



**J) Effect of collision in transverse direction**

**Case vi**

**Axial load**

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

**Longitudinal moment**

Pile Cap Top = 0.00 kNm  
 Pile Cap Bottom = 0.00 kNm

**Transverse moment**

Collision load in longitudinal direction = 250.00 kN acting at 1.5m above carriageway level of service road  
 Pile Cap Top = 250x(1.5+0.5) = 500 kNm  
 Pile Cap Bottom = 250(1.5+2.3000000000000001) = 950.00 kNm

**Summary of Axial Loads & Moments**

Descriptions	Footing top	Footing bottom
Axial Load	8611.03	10527.82
M <sub>L</sub> (Tm)	0.00	0.00
M <sub>T</sub> (Tm)	500.00	500.00

The Axial loads and moments will be equally shared by the 3 piers.

Description	(i)	(ii)	(iii)	(iv)	(iv b)	(V)	(Vi)
<b>At Pile Cap top</b>							
Axial Load ( T )	3249.99	3480.45	3480.45	3407.65	3092.33	2870.34	2870.344
Moment (Long - Tm)	900.06	2232.75	2248.16	1176.55	1176.55	333.33	0
Moment (Trans - Tm)	1926.03	963.02	2514.85	3131.24	3131.24	0.00	166.6667
% steel assumed	1.70	1.70	1.70	1.70	1.70	1.70	1.70
Stress in concrete (N/mm <sup>2</sup> )	10.58	14.17	19.81	18.20	18.24	3.04	2.41
Stress in steel (N/mm <sup>2</sup> )	87.38	159.33	272.91	227.00	241.35	2.85	14.11
<b>At Pile Cap bottom</b>							
Axial Load ( T )	3888.92	4213.09	4213.09	4046.58	3731.26	3509.27	3509.272
Moment (Long - Tm)	1226.25	3307.72	3331.07	1638.88	1638.88	633.33	0
Moment (Trans - Tm)	1926.03	963.02	3314.56	3265.28	3265.28	0.00	166.6667

The stresses in concrete and steel is calculated by a programme developed for columns subjected to axial load and biaxial bending. In non of the load cases, the stresses in concrete and steel is exceeded beyond the permissible limits. Hence the section adopted is safe.

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

Calculation of Pier Reinforcements.

Longitudinal Reinforcements:

( Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000 )

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

Transverse Reinforcements:-

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

Longitudinal Reinforcements:

C/S Area of Pier Section. = 1.327 m<sup>2</sup> = 13273.23 cm<sup>2</sup>  
 a) 0.3 % C/S Area. =  $\frac{13273.23 \times 0.3}{100}$  = 39.81969 cm<sup>2</sup>  
 b) Direct Stress =  $\frac{P}{A}$  P max = 3480.45 kN = 348044.5 Kg  
 $\sigma_{cbc}$  = 16.66667 Kg/cm<sup>2</sup>  
 $\therefore$  Area =  $\frac{348044.5}{16.66667}$  = 2088 cm<sup>2</sup>  
 0.8 % of Min C/S Area. =  $\frac{2088.267 \times 0.8}{100}$  = 16.706 cm<sup>2</sup>

Assumed % of Longitudinal Reinforcement = 1.70  
 $\therefore$  Longitudinal Reinforcement Provided.  

$$= \frac{13273.23}{100} \times 1.70 = 225.6449 \text{ cm}^2$$

Using 32 mm f bars,  $A_{s1} = 8.042 \text{ cm}^2$   
 No of bars reqd:  $= \frac{225.645}{8.042477} = 28.06$  Nos

Provided steel 32 mm  $\phi$  bars, at 30 no of bars

Transverse Reinforcements:-

a) Diameter of Transverse Reinforcement  

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

b) Minimum Diameter = 10 mm

Pitch of Transverse Reinforcement

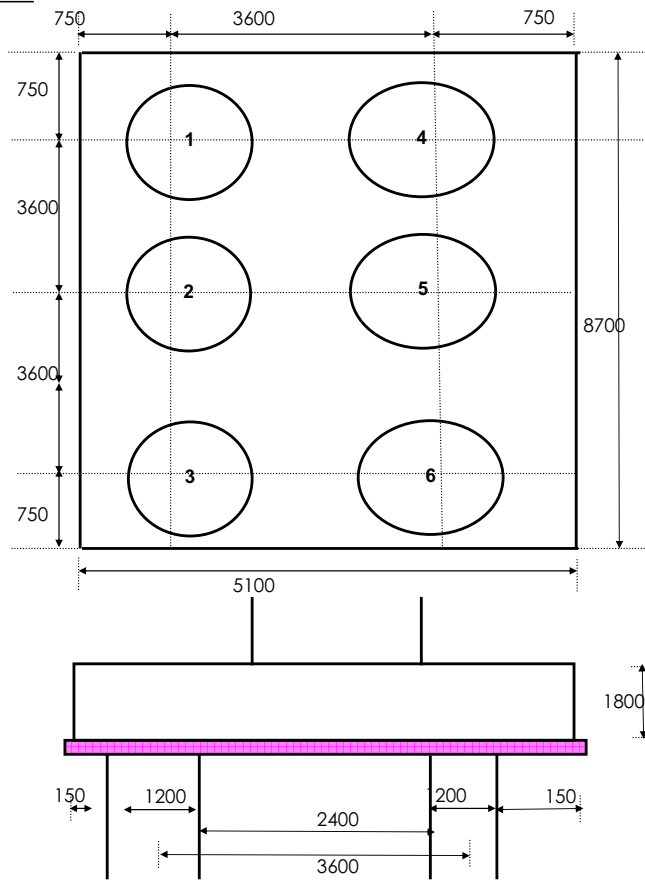
a) The least Lateral Dimension of the Column. = 1300 mm

b)  $12 \times 32 = 384$  mm

c) Maximum Allowable Spacing = 300 mm

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

**Design of Pilecap & Pile**



Length of pile = 25 m  
 S.B.C of Pile = 4520 kN

Moment due to tilt of pile

As per CI 709.1.6 of IRC:78-2000, for vertical piles,

Permissible shift of pile = 75 mm

Permissible tilt of pile ( 1:150 ) = 166.7 mm

Moment due to tilt of pile = Axial load/pile x ( 166.7 ) mm

Depth of fixity = 7000 mm

Reduction factor = 0.82

a) Piles :-

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

$$\text{Load /pi} = \frac{W}{n} \pm \frac{M \times y}{\Sigma y^2} \pm \frac{M \times x}{\Sigma x^2}$$

W = Total Load  
 n = No of piles = 6  
 y = 1.800 m Considering 75mm permissible shift  
 x = 1.800 m  
 $\Sigma x^2 = 12.960 \text{ m}^2$   
 $\Sigma y^2 = 19.440 \text{ m}^2$

Reduction Factor = 0.8

Considering the total load from the 4 piers

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

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

Description	(i)	(ii)	(iii)	(iv)	(ivb)
Axial Load kN	2996.99	3128.01	3778.89	3654.79	3497.13
Horizontal load/pile-Long	90.61	512.66	32.27	90.61	90.61
Horizontal load/pile-Trans	0.00	0.00	0.00	123.70	123.70
Moment due to horizontal load-Long	260.05	1471.34	92.61	260.05	260.05
Moment due to horizontal load-Trans	0.00	0.00	0.00	355.02	355.02
$M_r = M_L^2 + M_T^2$	260.05	1471.34	92.61	440.07	440.07
Moment /pile due to tilt of piles	499.50	521.34	629.81	609.13	582.86
Total moment	759.55	3464.02	815.03	1309.25	1282.98
e ;eccentricity (M/N)	0.25	1.11	0.22	0.36	0.37
R ;radius of column	0.60	0.60	0.60	0.60	0.60
r ;radius of reinf: ring	0.53	0.53	0.53	0.53	0.53
Bar size in mm	20	20	20	20	20
Perimeter in mm	3298.67	3298.67	3298.67	3298.67	3298.67
Spacing in mm	180.0	180	180	180	180
No of bars	18.33	18.33	18.33	18.33	18.33
p;% of reinforcement	0.0051	0.0051	0.0051	0.0051	0.0051
n ;modular ratio	10.00	10.00	10.00	10.00	10.00
e / R	0.422	1.846	0.36	0.60	0.61
r / R	0.875	0.875	0.875	0.875	0.875
n p	0.05	0.05	0.05	0.05	0.05
constant "c"(from Manohar chart)	2.060	2.450	2.110	2.040	2.000
constant "k"(from Manohar chart)	0.200	1.390	0.700	0.540	0.350
"fc" c(M/R <sup>3</sup> ) Kg/cm <sup>2</sup>	7.24	39.29	7.96	12.37	1187.94
"fs" n k fc Kg/cm <sup>2</sup>	14.49	546.14	55.73	66.77	4157.80
Per: stress in conc:	16.67	25.00	25.00	22.22	22.22
Per: stress in steel	200	300	300	267	267
Remarks	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>
% increase in stress	1.00	1.50	1.50	1.33	1.50
Vertical Capacity of Pile kN	4520	5650	5650	5650	5650
Horizontal Capacity of Pile kN	200	200	200	200	200
Remarks	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>	<b>Safe</b>

#### Calculation of Pile Reinforcements.

##### Longitudinal Reinforcements:

( Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000 )

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

##### Transverse Reinforcements:-

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

##### Longitudinal Reinforcements:

$$\begin{aligned}
 \text{C/S Area of Pile Section.} &= 1.131 \text{ m}^2 &= 11309.73 \text{ cm}^2 \\
 \text{a) 0.4 \% C/S Area.} &= \frac{11309.73 \times 0.4}{100} &= 45.24 \text{ cm}^2
 \end{aligned}$$

Assumed % of Longitudinal Reinforcement = 0.509  
 $\therefore$  Longitudinal Reinforcement Provided.  

$$= \frac{11309.73 \times 0.509}{100} = 57.57 \text{ cm}^2$$

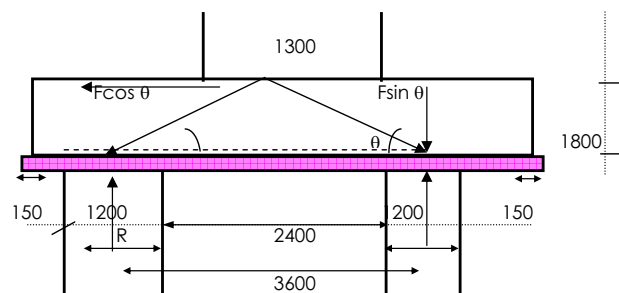
Using 20 mm f bars,  $A_{s1} = 3.142 \text{ cm}^2$   
 No of bars reqd:  $= \frac{57.573}{3.141593} = 18.33 \text{ Nos}$

Provided steel 20 n  $\phi$  bars, at 19 no of bars

Transverse Reinforcements:-

- a) Diameter of Transverse Reinforcement  
 $= \frac{1}{4} \times 20 = 5 \text{ mm}$
- b) Minimum Diameter = 10 mm  
Pitch of Transverse Reinforcement
- a) The least Lateral Dimension of the Column. = 1200 mm  
 b)  $12 \times 20 = 240 \text{ mm}$   
 c) Maximum Allowable Spacing = 300 mm  
 So provide 10 n f bars, at 240 mm C/C.

Pilecap :-



Design of steel in the Longitudinal direction.

Check for Pile Cap depth  
 Maximum Moment 2267.33 kNm  
 Moment of resistance factor 1.90 N/mm<sup>2</sup>  
 Effective depth required, d<sub>req</sub> 1092.40 mm  
 Effective depth provided, d<sub>pro</sub> 1649.00 mm  
 Hence Safe

Design of reinforcement in longitudinal direction

$R = F \sin \theta$   
 Providing 32 mm dia bar at bottom of pile cap.  
 $\tan \theta = 0.95$   
 $\theta = 43.40$   
 Horizontal force ,H 3996.47 kN  
 A<sub>s1</sub> Provided 19982.4 mm<sup>2</sup>  
 Provide 80 % steel in each band (above pile)(Refer Cl 307.2.5 IRC 21:2000)  
 Steel area 15985.9 mm<sup>2</sup>  
 No of bars Reqd: 15985.9  
 804.25  
 19.88 Nos  
 No of bars in each band 20.00 Nos  
 Provide 20 bars of 32mm dia in each band in longitudinal direction at bottom  
 ( providing 16084.95 mm<sup>2</sup> )  
 1.5 times the diameter is taken as band width as per Cl 307.2.5.2 -IRC 21-2000  
 A<sub>s1</sub> Required in the remaining portion 3996.5 mm<sup>2</sup>  
 Assume 20 mm dia bars  
 Provide 20 mm dia bar @ 120 mm c/c in longitudinal direction at pile cap bottom  
 in the remaining portion.  
 Provide min reinforcement of 0.06% in the pile cap top  
 A<sub>s1</sub> required in the pile cap top 1080 mm<sup>2</sup>  
 Provide 16 mm dia bar @ 150 mm c/c in at pile cap top in both directions

Design of reinforcement in transverse dirn

This has be designed as cantilever bending due to pile load.

Moment, M = 4345.72 kNm

As<sub>t</sub> required = 14190.6 mm<sup>2</sup>

Min As<sub>t</sub> required = 0.85bd/fy = 16820 mm<sup>2</sup>

Assume 25 mm dia bars spacing 140

Provide 25mm bars @ 140mm c/c

Min Stirrup Reinforcement Reqd :-

Assume 8 legged 10 mm diameter stirrups  
S<sub>v</sub> = 162.039 mm

Provide 8 legged 10 mm diameter stirrups @ 150 mm C/c

**Design of Bed Block**

Moment at face of Pier = 750.00 kNm Refer staad

Torsion due to Live load from one side = 56.41 kNm

Equivalent longitudinal moment due to torsion = 49.77 kNm 56.41 \* (1+1/1.4)/1.7

Σ Moments = 750.00 kNm

Effective Depth Reqd "deff" =  $\sqrt{\frac{M}{Q \times B}}$  Clear Co<sub>r</sub> = 50 mm  
Bar Dia = 25 mm

deff reqd: =  $\sqrt{\frac{750.00}{1.47} \times \frac{1000000}{1600}}$

= 564.69 cms < 993.75 cms Hence O.K  
deff provided = 1000 - 5 - 1.25 = 993.75 mm

As<sub>t</sub> reqd =  $\frac{M}{\sigma_{st} \times j \times d}$

∴ As<sub>t</sub> reqd =  $\frac{750.00}{200} \times \frac{1000000}{0.89 \times 993.75} = 4239.98 \text{ mm}^2$

As<sub>1</sub> = 490.8739 cm<sup>2</sup>

No of bars Reqd: =  $\frac{4239.983}{490.8739} = 8.64 \text{ Nos}$

Spacing of Bars =  $\frac{1600}{9} = 185.2 \text{ mm}$

So provic 25 mm dia bars , 185 mm c/c spacing

2) Check for Shear at pier face.

Calculation of S.Force at face of Pier due to

Due to DL, SIDL and LL = 1014 kN Refer staad

Equivalent shear due to torsion = 56.41 kN

S Shear = 1071.07 kN

Shear stress , τ =  $\frac{V}{B \times d}$  ( Vide cl - 304.7.1.1 of I.R.C:-21-2000 )

V = The design shear across the section

d = Effective depth of the section

B = Breadth of slab

∴ τ =  $\frac{1071.1}{1600} \times \frac{1000}{993.75} = 0.7 \text{ Kg/cm}^2$

Maximum Permissible Shear Stress :-

τ max = 2.3 Mpa

τ max perm = 23.0 Kg/cm<sup>2</sup>

Calculation of permissible Shear Stress :- ( Vide cl - 304.7.3.1 of I.R.C:-21-2000 )

d = 993.75 Effective Depth

p =  $\frac{100 \times A_s}{B \times d}$  As = 4239.98 mm<sup>2</sup>

p =  $\frac{4240.0}{1600} \times \frac{100}{993.8} = 0.267$

∴ τ<sub>c</sub> = 0.2 kg/cm<sup>2</sup>

Since τ > τ<sub>c</sub> Shear Reinforcement Reqd:

Shear Reinforcement Reqd :-

$$\begin{aligned}
 A_{sw} &= \frac{V}{s} \times \frac{x}{d} \\
 V &= 685.75 \text{ kN} \\
 s, \text{ spacing} &= 100 \text{ mm} \quad (\text{assumed}) \\
 \therefore A_{sw} &= \frac{685.75}{200} \times \frac{1000}{993.8} \times 100 = 345.0 \text{ mm}^2 \\
 \text{Using } 12 \text{ mm dia stirrups } &= 6 \text{ Legged at spacing } 100 \text{ mm.} \\
 A_{sw} &= 678.6 \text{ cm}^2 > 345.0 \text{ mm}^2 \\
 &\text{Hence O.K}
 \end{aligned}$$

B) Design of Cantilever Portion in the Longitudinal Direction.

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

**Design of Elliptical Pier with Pile Foundation - P7**

Design Data for Substructure:-

Type		PSC Girder		
C/C Distance bet <sup>n</sup> piers	=	40000	mm	on left side
	=	40000	mm	on right side
Carriageway Width	=	7500	mm	on both sides
Overall Width of Deck.	=	17000	mm	
Width of Crash Barrier	=	500	mm	One side
Width of Median	=	1000	mm	One side
Height of Crash Barrier	=	1050	mm	
No of Bearings	=	3	on left side	
		3	on right side	
Overall Depth of I.Girders	=	2200	mm	
Depth of Deck Slab	=	250	mm	
Depth of cross girder	=	2200	mm	
Width of cross girder	=	800	mm	

Design Data for Pier :-

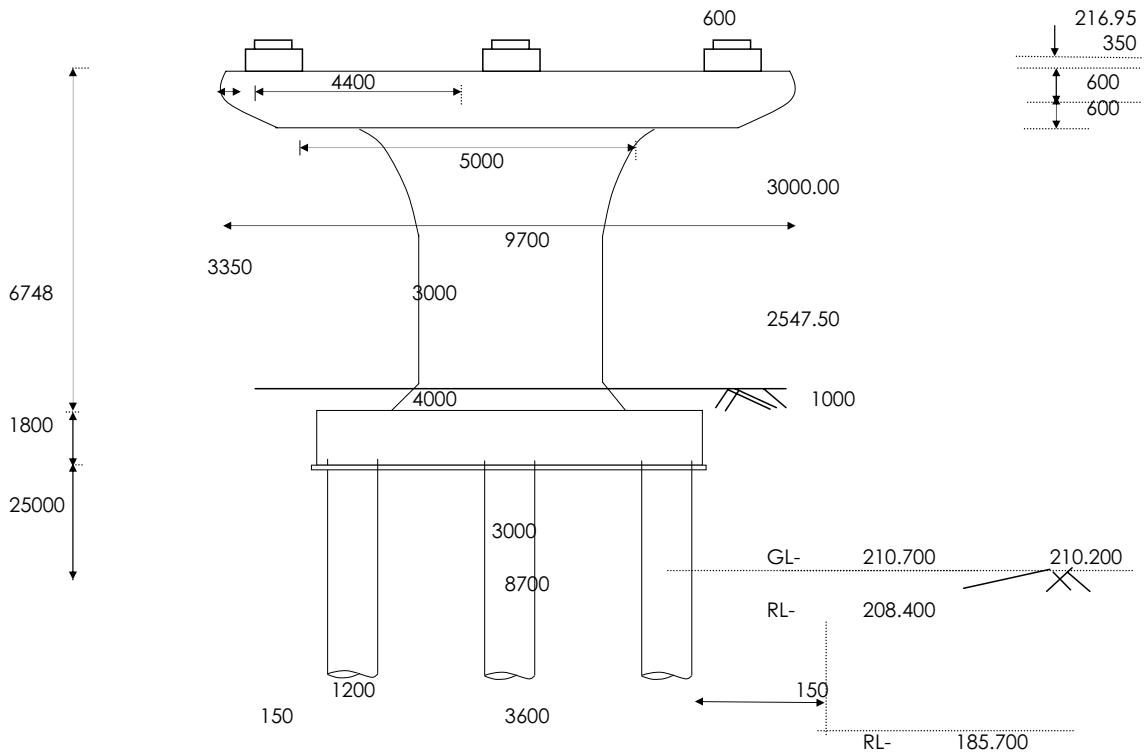
Formation level at pier locatic	=	220.100	m	
R.L at pier cap top	=	216.95	m	
Height of varying portion of pi	=	3000.00	mm	
Height of straight portion of pi	=	2547.50	mm	
Existing Ground level	=	210.700	m	
Pile cap top below existing gr	=	500	mm	
R.L at Pile cap top	=	210.20	m	
R.L at Pile cap bottom	=	208.40	m	
Depth of Pile below GL	=	25000	mm	
Founding level for Piles.	=	185.70	m	
Overall Height of Substructure	=	6748	mm	
Longitudinal width of pier at b	=	1700	mm	
Transverse width of pier at bo	=	3000	mm	
Transverse width of pier above pile cap	=	4000	mm	
Longitudinal width of pier at top	=	2500	mm	
Transverse width of pier at top	=	5000	mm	
No of piers	=	1		
Pier Cap Width in Long Dir <sup>n</sup> .	=	2500	mm	
Pier Cap Length in Trans Dir <sup>n</sup> .	=	9700	mm	
Straight Depth of Pier Cap	=	600	mm	
Varying Depth of Pier Cap	=	600	mm	
Type of Bearing	=	Pot PTFE bearing		
Size of Pedestals	=	600	x	600 x 350
Distance betn Pedestals	=	4400	mm	on left side
	=	4400	mm	on right side
Longitudinal width of pile cap	=	8700	mm	
Transverse width of pile cap	=	8700	mm	
Straight Depth of pile cap	=	1800	mm	
Varying Depth of pile cap	=	0	mm	
P.C.C Projections	=	150	mm	
Diameter of Pile	=	1200	mm	
Distance betn Piles in longitudinal dirn	=	3600	mm	
Distance betn Piles in transverse dirn	=	3600	mm	
No of Piles	=	8		
Edge projection in	=	150	mm	
Edge projection in transverse	=	150	mm	
Grade of Concrete for pier	=	<b>M</b> 50		
Permissible flexural stress	=	16.67	N/mm <sup>2</sup>	
Grade of Steel	=	Fe - 415		
Permissible tensile stress	=	200	N/mm <sup>2</sup>	
Total Height of Pier	=	5548	mm	
Density of Concrete	=	24	kN/m <sup>3</sup>	
Density of Concrete for Earth	=	18	kN/m <sup>3</sup>	

References.

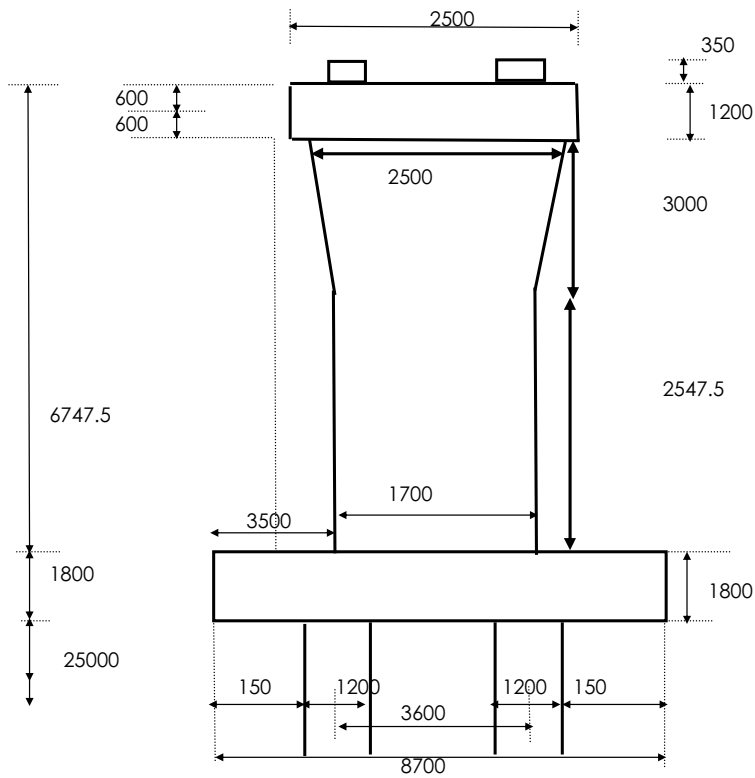
I.R.C :- 6 - 2000 - Loads & Stresses.

I.R.C :- 21 - 2000 - Permissible Stresses

I.R.C :- 78 - 2000 - Sub Structure and Foundation



**Transverse View**



**Longitudinal View**



**Load Calculations:-**

**1) DEAD LOADS**

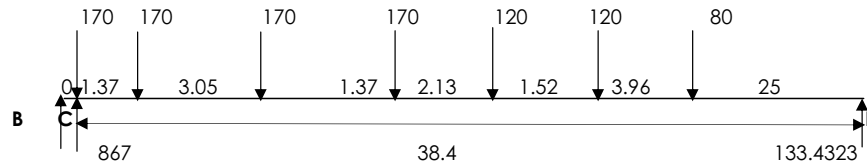
Load from slab, girder & diaphragm = 7184 kN for 40m span  
 Load from slab, girder & diaphragm = 7184 kN for 40m span  
 Total dead load = 14368.00 kN

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

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

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

**70 R**



$$R_C = (170 \times 38.4 + 170 \times 37.03 + 170 \times 33.98 + 170 \times 32.61 + 120 \times 30.48 + 120 \times 28.96 + 80 \times 25) / 38.4$$

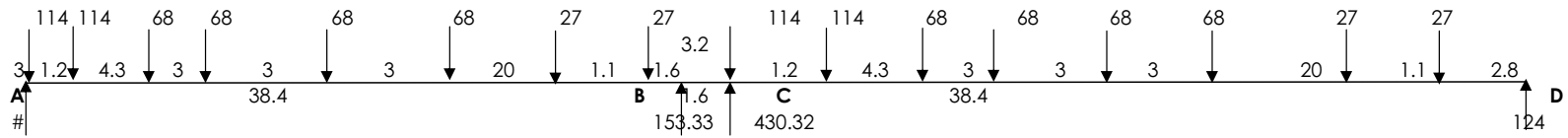
$$R_C = 866.568 \text{ kN}$$

$$R_D = 170 + 170 + 170 + 170 + 120 + 120 + 80 - 866.568$$

$$R_D = 133.432 \text{ kN}$$

**70 R one lane = 866.57 kN**

**Class A One lane**



$R_A$	=	400.67 kN
$R_B$	=	153.33 kN
$R_C$	=	430.32 kN
$R_D$	=	123.68 kN
Reaction at pier Support	=	583.65 kN
Class A One lane	=	583.65 kN
Class 70R	=	866.57 kN
Class A two lane	=	1167.29 kN
Class A One lane ( One span dislodged)	=	430.32 kN
Critical live load	=	866.57 kN

**Calculation of Dead Load from Sub - Structure :-**

<b>1) From Pedestals</b>						
Volume of 1 Pedestal	=	0.6	x	0.6	x	0.35
	=	0.126	m <sup>3</sup>			
No: of Pedestals	=	6	x	1	=	6 Nos
∴ Total Volume	=	0.126	x	6	=	0.76 m <sup>3</sup>
∴ Total Load	=	0.756	x	24	=	18.14 kN
<b>2) From Bed - Block :</b>						
Total C/S Area in transverse Dir <sup>n</sup>	=	11.25	m <sup>2</sup>			
Width in the longitudinal Dir <sup>n</sup>	=	2.5	m			
∴ Total Volume	=	28.13	m <sup>3</sup>			
∴ Total Load	=	28.13	x	24	=	675.00 kN
<b>3) From Pier :-</b>						
Area at top	=	10.62	m <sup>2</sup>			
Area at bottom	=	4.35	m <sup>2</sup>			
Averaged area	=	7.485	m <sup>2</sup>			
Height of varying portion	=	3	m			
Volume of varying portion	=	22.455	m <sup>3</sup>			
Height of straight portion	=	1.5475	m			
Volume of straight portion	=	6.73	m <sup>3</sup>			
Area of pier above pilecap	=	5.78	m <sup>2</sup>			
Volume of tapered portion	=	5.78	m <sup>3</sup>			
∴ Total Volume	=	34.967	m <sup>3</sup>			
∴ Total Load	=	34.967	x	1 x 24	=	839.20 kN
<b>4) From Pile Cap :-</b>						
Volume of pile cap	=	( 8.7	x	8.7	x	1.8 )
	=	136.242	m <sup>3</sup>			
∴ Dead Load	=	136.242	x	24	=	3269.81 kN
<b>5) From Pile :-</b>						
Diameter of Pile	=	1.2	m			
Depth of Pier with Circular X <sup>n</sup>	=	25	m			
C/S Area of Circular Pier	=	1.131	m <sup>2</sup>			
No of Piles	=	8.00				
∴ Total Volume of Concrete	=	226.195	m <sup>3</sup>			
∴ Total Load	=	226.195	x	24	=	5428.67 kN

Options

**A) Both Spans on.**

**Case (i)**

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

**1) Due to Braking**

b) Braking.

( Vide :- cl 214.2 (a) & (b) of I.R.C : 6 - 2000 . )

Braking. Since the movement of bearing under the girders on one side is restricted to move in the longitudinal direction half the effect of braking is considered in the design.

1) 20 % of 1<sup>st</sup> Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.

2) 20 % of 1<sup>st</sup> Train Load. + 10% of succeeding Train Loads for Single or a Two Lane Bridge.

+ 5 % of Loads on the lanes exceeding Two.

2 Lanes of Class A Wheeled Vehicles.

Total Load of 1 Vehicle = 554 kN one span

∴ Braking Force = 110.8 kN

1 Lanes of Class 70 R Wheeled Vehicles

Total Load of 70R Vehicle = 1000 kN one span

∴ Braking Force = 200 kN

Max Braking Force = 200 kN

Vertical reaction due to braking

$$= 200(1.2+0.065+2.45)/(40-1.6)$$

$$= 19.35 \text{ kN}$$

Longitudinal moment due to vertical load of braking

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

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

Fixed Bearing

Coefficient of Friction,m = 0.05  
 (i)  $F_h - m (R_g + R_q)$  = -200.33 kN  
 (ii)  $\frac{F_h + m (R_g + R_q)}{2}$  = 543.66 kN

Free Bearing

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

3) **Moment due to Longitudinal Eccentricity**

Longitudinal Eccentricity	=	0.80	m		
		Normal Case		Seismic Case	
Due to DL	=	0	kNm	0	kNm
Due to SIDL	=	0	kNm	0	kNm
Due to LL	=	693	kNm	347	kNm
Moment	=	693	kNm	347	kNm

4 **Centrifugal force**

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

Centrifugal force acts at 1.2m above the level of carriage way.

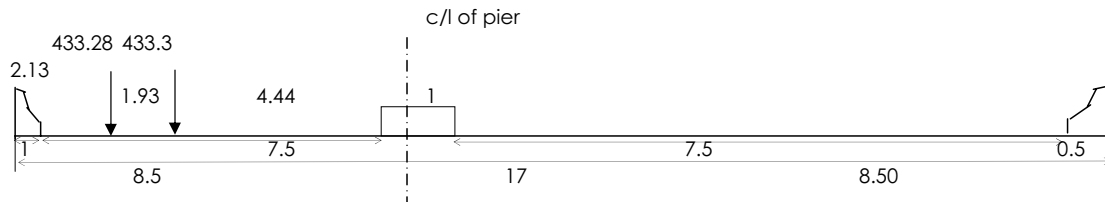
Increase in impact effect is not considered.(Refer CI 215-IRC 6:2000)

\ Moment at Pile Cap top = 29 x 10.91 = 316.85 kNm  
 \ Moment at Pile Cap bot = 29 x 12.72 = 369.41 kNm

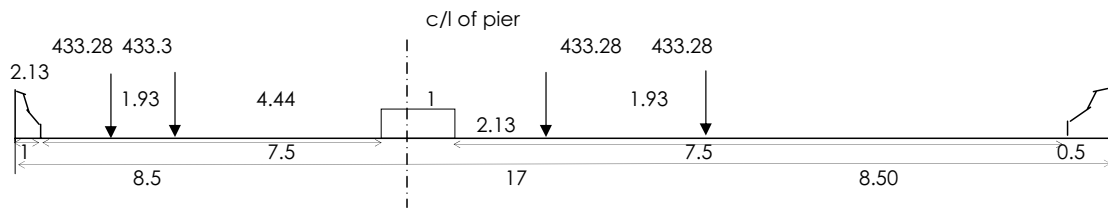
**Due to Live Load**

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

**Class 70R 1 lane**

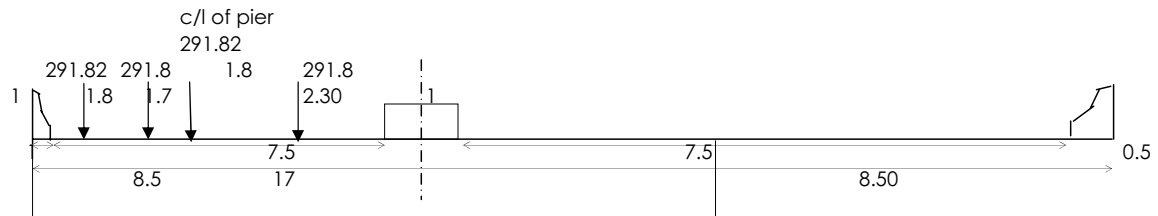


Moment in Transverse Dirn = 4683.80 kNm

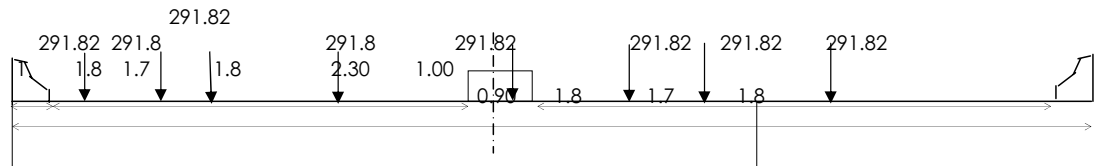


Moment in Transverse Dirn = 2001.77 kNm

**Class A 2 lane.**

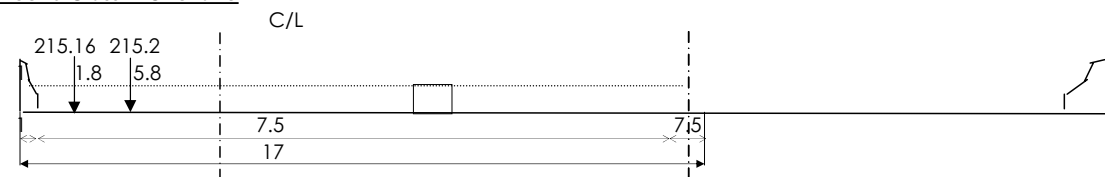


Moment in Transverse Dir<sup>n</sup>. = 5778.09 kNm



Moment in Transverse Dir<sup>n</sup>. = 1634.21 kNm

**Due to Class A One lane**



Transverse Eccentricity = 3.80 m  
 Moment in Transverse Dir<sup>n</sup>. = 2883.11 kNm

**Axial Load**

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

**Longitudinal moment**

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

**Transverse moment**

Pile Cap Top = 6094.94  
 Pile Cap Bottom = 6147.50

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	18431.5	21701.3
M <sub>L</sub> (Tm)	7503.1	9202.3
M <sub>T</sub> (Tm)	6094.9	6147.5

**B) One Span Dislodged with one lane of class A**

**Case ( ii )**

Calculation of Longitudinal Moments at Pier Base & Pile cap Base

1) Due to Temp & Shrinkage of Bearings :

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

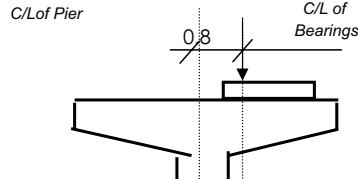
$$\begin{aligned} \text{Vertical reaction due to braking} &= 110.8(1.2+0.065+2.45)/(40-1.6) \\ &= 10.72 \text{ kN} \end{aligned}$$

Longitudinal moment due to vertical load of braking

$$\begin{aligned} \text{Longitudinal Eccentricity} &= 0.800 \text{ m} \\ \text{Moment due to long.Eccentri} &= 9 \text{ kNm} \end{aligned}$$

2) Due to Eccentricity of Dead Load and SIDL :-

$$\text{Eccentricity of Loading} = 0.8 \text{ m}$$



$$\begin{aligned} \text{Dead Load +SIDL} &= 8006.60 \text{ kN} \\ \text{Longitudinal Moment} &= 6405.28 \text{ kNm} \\ \text{Transverse moment for sidl} &= 0.00 \text{ kNm} \\ \text{Axial Load} & \end{aligned}$$

$$\begin{aligned} \text{Pile Cap Top} &= 822.6 + 7184 + 430.32 + 18.15 + 675 + 839.2 + 10.72 \\ \text{Pile Cap Bottom} &= 822.6 + 7184 + 430.32 + 18.15 + 675 + 839.2 + 3269.81 + 10.72 \end{aligned}$$

Longitudinal moment

$$\begin{aligned} \text{Pile Cap Top} &= 6405.28 + 430.32 \times 1.125 + 499.06 \times 7.2 + 8.58 \\ \text{Pile Cap Bottom} &= 6405.28 + 430.32 \times 1.125 + 499.06 \times 9 + 8.58 \end{aligned}$$

Transverse moment

$$\begin{aligned} \text{Pile Cap Top} &= 2883.12+871.2+157.34 \\ \text{Pile Cap Bottom} &= 2883.12+871.2+183.43 \end{aligned}$$

4 Centrifugal force

$$\begin{aligned} \text{Live load} &= 430 \text{ kN} \\ \text{Design Speed} &= 100 \text{ km/hr} \\ \text{Radius of curvature} &= 2350.00 \text{ m} \\ \text{Centrifugal force} &= \frac{WV^2}{127R} \\ &= 14 \text{ kN} \end{aligned}$$

Centrifugal force acts at 1.2m above the level of carriage way.

Increase in impact effect is not considered.(Refer CI 215-IRC 6:2000)

$$\begin{aligned} \text{Moment at Pile Cap top} &= 14 \times 10.91 = 157.34 \text{ kNm} \\ \text{Moment at Pile Cap bottc} &= 14 \times 12.72 = 183.44 \text{ kNm} \end{aligned}$$

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	9979.98	13249.79
M <sub>L</sub> (Tm)	10350.08	11248.39
M <sub>T</sub> (Tm)	3754.31	3754.31

**3) Both Spans on Under Seismic in Longitudinal Direction.**

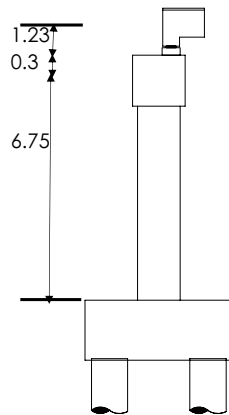
**Case ( iii )**

As per Modified Clause 222

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

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

$A_h$	=	Horizontal Seismic coefficient
	=	$\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)I$
Zone No	=	IV
Zone Factor,Z	=	0.24
Sa/g	=	Average accelaration coefficient
Response Modification factor,R	=	3.3
Importance Factor,I	=	1.2
T	=	$2\sqrt{\frac{D}{1000F}}$
Dead load of the super structure, and appropriate live load in kN,D	=	16013.20 kN
F	=	Horizontal force in kN required to be applied at the centre mass of the super structure for one mm horizontal deflection at the top of the pier /abutment along the considered direction of horizontal force.
F	=	$\left(\frac{3EI\delta}{l^3}\right)$
Modulus of Elasticity of concrete,E	=	3.0E+07 kN/m <sup>2</sup>
$I_{xx}$	=	0.41 m <sup>4</sup>
Deflection, $\delta$	=	0.001 m
Distance from bottom to the centre of mass of super structure,y	=	1.23 m
Height of substructure,h	=	6.75 m
Total height measured from the centre of mass of super structure,l	=	8.27 m



F	=	64.27 kN
T	=	1.00
Soil type	=	II
Sa/g	=	1.362
$A_h$	=	0.20
$F_{eqx}$	=	3141.24 kN
Design Seismic force in long d	=	$F_{eqz} / R$
$DF_{eqz}$	=	951.89 kN

Seismic force in transverse direction

The seismic force due to live load shall be considered when acting in the direction perpendicular to traffic  
The horizontal seismic force in the direction perpendicular to traffic shall be computed by taking 20% of live load (excluding impact factor)

Dead load and appropriate	=	16186.51 kN (0 + 0.2 x 0)
$F_{eq}$	=	$A_h$ (Dead load +Appropriate Live Load)

$I_{yy} = 0.41 \text{ m}^4$   
 $F = 64.27 \text{ kN}$   
 $T = 1.00$   
 Soil type = II  
 $Sa/g = 1.35$   
 $A_h = 0.2$   
 $F_{eqx} = 3158.19 \text{ kN}$   
 Design Seismic force in trans = 957.03 kN  
 Seismic vertical component, V = 638.02  $\frac{2}{3} * 957.03$   
 Resultant force in longi dirn = 1430.40 kN  $DF_{eqz} + 0.3 DF_{eqx} + 0.3V_f$   
 CG of loads = 7.52

Moment,  $M_{eqz}$  at Pile Cap top = 10756.85 kNm  
 Moment,  $M_{eqz}$  at Pile Cap bottom = 13331.58 kNm  
 Seismic vertical load ( Acting downwards-considering critical condition)  
 At pile cap top = 1210.69 kN  $0.3DF_{eqz} + 0.3 DF_{eqx} + V_f$   
 At pile cap bottom = 1513.37 kN  $(0.3DF_{eqz} + 0.3 DF_{eqx} + V_f) * 1.25$

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

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

Transverse moment  
 Pile Cap Top = 5778.09 x 0.5 + 0 + 316.85 x 0.5  
 Pile Cap Bottom = 5778.09 x 0.5 + 0 + 369.4 x 0.5  
 As per Table 1 Load Combination, 50 % of LL is considered in seismic case

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	16787.48	19754.62
$M_L$ (Tm)	17397.51	21542.42
$M_T$ (Tm)	3047.47	3073.75

**D) Both Spans on Under Seismic in Transverse Direction.**

**Case ( iv )**

The Seismic effect on Live Load is taken in this Case

Design Seismic force in trans dirn,  $DF_{eqx} = 957.03 \text{ kN}$   
 Resultant force in trans dirn = 1434.00 kN  $0.3DF_{eqz} + DF_{eqx} + 0.3V_f$   
 CG of loads = 7.52 m

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

Axial load is same as that of Case iii  
 Longitudinal moment  
 Pile Cap Top = ( 100 x 0.5 + 46.8 ) x 7.19 + 10756.84  
 Pile Cap Bottom = ( 100 x 0.5 + 46.8 ) x 8.99 + 13331.57

Transverse moment  
 Pile Cap Top = 3047.47 + 10783.9  
 Pile Cap Bottom = 3047.47 + 13365.1  
 As per Table 1 Load Combination, 50 % of LL is considered in seismic case



**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	16787.48	19754.62
$M_L$ (Tm)	6625.18	8195.36
$M_T$ (Tm)	13989.79	16570.99

**E) One Spans Dislodged Under Seismic in Longitudinal Direction.**

**Case ( v )**

D	=	8006.60	kN
T	=	0.71	
Soil type	=	II	
Sa/g	=	1.93	
$A_h$	=	0.28	
$F_{eqz}$	=	2221.19	kN
Design Seismic force in long d	=	$F_{eqz} / R$	
$DF_{eqz}$	=	673.09	kN
<u>Seismic force in transverse direction-One span dislodged</u>			
Dead load and appropriate live load in kN,D			
live load in kN,D	=	8092.66	kN
Feq	=	$A_h$ (Dead load +Appropriate Live Load)	
$I_{yy}$	=	0.41	$m^4$
F	=	64.27	kN
T	=	0.71	
Soil type	=	II	
Sa/g	=	1.92	
$A_h$	=	0.28	
$F_{eqx}$	=	2233.10	kN
Design Seismic force in trans dirn, $DF_{eqx}$			
dirn, $DF_{eqx}$	=	676.70	kN
Seismic vertical component,V	=	451.13	kN
Resultant force in longi dirn	=	1011.44	kN
CG of loads	=	7.27	m
Moment, $M_{eqx}$ at Pile Cap top			
Moment, $M_{eqx}$ at Pile Cap bottom	=	7348.98	kNm
	=	9169.56	kNm
Seismic vertical load ( Acting downwards-considering critical condition)			
At pile cap top	=	856.07	kN
At pile cap bottom	=	1070.08	kN
			$0.3DF_{eqz}+0.3 DF_{eqx}+ V_f$
			$(0.3DF_{eqz}+0.3 DF_{eqx}+ V_f) \cdot 1.25$
Axial load			
Pile Cap Top	=	$8006.6+430.32 \cdot 0.5+18.15+675+839.2 + 10.72$	
Pile Cap Bottom	=	$8006.6+430.32 \cdot 0.5+18.15+675+839.2+3269.81 + 10.72$	
Longitudinal moment			
Pile Cap Top	=	$6405.28 + ( 50 + 46.8 ) \times 7.2 + 172.13 + 7348.98 + 8.58$	
Pile Cap Bottom	=	$6405.28 + ( 50 + 46.8 ) \times 8.99 + 172.13 + 9169.56 + 8.58$	
Transverse moment			
Pile Cap Top	=	$2883.12 \times 0.5+0$	
Pile Cap Bottom	=	$2883.12 \times 0.5+0$	

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	8908.75	11964.55
$M_L$ (Tm)	17281.23	19938.67
$M_T$ (Tm)	1520.23	1533.27

The seismic force caused due to dead load of the girders placed over the Sand Jacks are considered.

**F) One Spans Dislodged Under Seismic in Transverse Direction.**

**Case ( vi )**

Design Seismic force in trans  
 dirn,  $DF_{eqx}$  = 676.70 kN  
 Resultant force in trans dirn = 1013.96 kN  
 CG of loads = 7.27 m

Moment,  $M_{eqx}$  at Pile Cap top = 7367.33 kNm  
 Moment,  $M_{eqx}$  at Pile Cap  
 bottom = 9192.46 kNm

Axial load is same as for Case v  
 Longitudinal moment  
 Pile Cap Top =  $6405.28 + (50 + 46.8) \times 7.197 + 172.13$   
 Pile Cap Bottom =  $6405.28 + (50 + 46.8) \times 8.99 + 172.13$

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

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	8908.75	11964.55
$M_L$ (Tm)	9923.68	10760.53
$M_T$ (Tm)	8966.22	10817.45

**G) Service condition with Wind in Transverse direction**

**Case ( viia )**

**Transverse direction**

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

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

Due to crash Barrier

Avg Height of Crash barrier from GL = 6.77 m  
 Intensity of Wind pressure corres : to height = 463.70 Kg/m<sup>2</sup>  
 Average Exposed Length = 40.00 m  
 Effective area of crash barrier = 42.00 m<sup>2</sup>  
 Force =  $P_z A_1 G C_d$   
 G = 2  
 $C_d$  = 1.30  
 Force = 506.36 kN

Due to Deck Slab and girder

Average ht of deck slab and girder from GL = 7.12 m  
 Intensity of Wind pressure corres : to height = 463.70 Kg/m<sup>2</sup>  
 Effective area of deck slab + girder = 98.00 m<sup>2</sup>  
 Force =  $P_z A_1 G C_d$   
 G = 2  
 $C_d$  = 1.95  
 Force = 177.23 kN

Live Load

Effective length = 40.00 m  
 Depth = 3.00 m  
 Height of the exposed surface above ground level = 7.75 m  
 Area = 120.00 m<sup>2</sup>  
 Pressure = 463.70 N/m<sup>2</sup>  
 Force =  $P_z A_1 G C_d$   
 G = 2  
 $C_d$  = 1.20  
 Force = 133.55 kN

Due to Pier			
Average ht of pier and pier cap from GL	=	3.37	m
Intensity of Wind pressure corres : to height	=	463.70	kg/m <sup>2</sup>
Effective area of pier and pier cap	=	10.73	m <sup>2</sup>
Force	=	$P_z A_1 G C_d$	kN
G	=	2	
$C_d$	=	0.80	
Force	=	7.96	kN
Total transverse force	=	825.09	kN
Transverse Moment due to wind			
Pile Cap Top	=	5753.12	kNm
Pile Cap Bottom	=	6495.71	kNm
Longitudinal Force			
Crash barrier		25 % of trans force	
		126.59	kN
Superstructure		25 % of trans force	
		44.31	kN
On Live load		25 % of trans force	
		33.39	kN
Substructure		25 % of trans force	
		1.99	kN
Total longitudinal force		206.27	kN
Longitudinal Moment due to wind			
Pile Cap Top	=	1438.28	kNm
Pile Cap Bottom	=	1809.57	kNm
Vertical load		$= P_z A_3 G C_L$	
	$P_z$	= 463.70	N/m <sup>2</sup>
	$A_3$	= 680.00	
	G	= 2.00	
	$C_L$	= 0.75	
		= 472.97	kN Acting upwards or downwards

Axial loads and Longitudinal Moments are same as Case 1

**Summary of Axial Loads & Moments**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	18904.43	22174.24
$M_L$ (Tm)	8941.37	11011.84
$M_T$ (Tm)	11848.07	12643.21

**Case ( vii-b )**

Descriptions	Pile Cap Top	Pile Cap Bottom
Axial Load	17958.49	21228.29
$M_L$ (Tm)	8941.37	11011.84
$M_T$ (Tm)	11848.07	12643.21

**I) Effect of collision in longitudinal direction**

**Case viii**

**Axial load**

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

**Longitudinal moment**

Collision load in longitudinal direction	=	500.00	kN	acting at 1.5m above carriageway level of service road
Pile Cap Top	=	$500 \times (1.5+0.5)$	= 1000.00	kNm
Pile Cap Bottom	=	$500 \times (1.5+2.30)$	= 1900.0	kNm

**Transverse moment**

Pile Cap Top	=	0.00	kNm
Pile Cap Bottom	=	0.00	kNm

**Summary of Axial Loads & Moments**

Descriptions	Footing top	Footing bottom
Axial Load	17545.54	20815.35
M <sub>L</sub> (Tm)	1000.00	1900.00
M <sub>T</sub> (Tm)	0.00	0.00

**J) Effect of collision in transverse direction****Case ix****Axial load**

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

**Longitudinal moment**

Pile Cap Top	=	0.00	kNm
Pile Cap Bottom	=	0.00	kNm

**Transverse moment**

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

**Summary of Axial Loads & Moments**

Descriptions	Footing top	Footing bottom
Axial Load	17545.54	20815.35
M <sub>L</sub> (Tm)	0.00	0.00
M <sub>T</sub> (Tm)	500.00	950.00

**Summary of loads and moment for all 7 cases**

Description	( i )	( ii )	( iii )	( iv )	( v )	( vi )	( viia )	( viib )	( viii )	( ix )
<b>At Pile Cap top</b>										
Axial Load ( T )	18431.46	9979.98	16787.48	16787.48	8908.75	8908.75	18904.43	17958.49	17545.54	17545.54
Moment (Long -	7503.09	10350.08	17397.51	6625.18	17281.2	9923.68	8941.37	8941.37	1000.00	0.00
Moment (Trans -	6094.94	3754.31	3047.47	13989.79	1520.23	8966.22	11848.07	11848.07	0.00	500.00
% steel assumed	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89	1.89
Stress in concrete	9.20	10.18	15.40	12.0	14.1	12.6	12.8	12.8	9.1	8.2
Stress in steel (N/mm <sup>2</sup> )	28.80	97.04	145.60	67.4	203.6	130.3	68.3	72.9	59.5	68.9
<b>At Pile Cap bottom</b>										
Axial Load ( T )	21701.27	13249.79	19754.62	19754.62	11964.5	11964.5	22174.24	21228.29	20815.35	20815.35
Moment (Long -	9202.27	11248.39	21542.42	8195.36	19938.7	10760.5	11011.84	11011.84	1900.00	0.00
Moment (Trans -	6147.50	3754.31	3073.75	16570.99	1533.27	10817.4	12643.21	12643.21	0.00	950.00

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

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

Calculation of Pier Reinforcements.Longitudinal Reinforcements:

( V<sub>l</sub> ( Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000 )

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

Transverse Reinforcements:-

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

- b) Minimum of 8mm Diameter.  
Pitch of Transverse Reinforcement shall be the least of the following.
- The least Lateral Dimension of the Column.
  - 12 Times the Diameter of the smallest Longitudinal Reinforcement.
  - Maximum allowable spacing of 300 mm

Longitudinal Reinforcements:

C/S Area of Pier Section. = 2.270 m<sup>2</sup> = 22698.01 cm<sup>2</sup>

a) 0.3 % C/S Area. =  $\frac{22698.007 \times 0.3}{100}$  = 68.09402 cm<sup>2</sup>

b) Direct Stress =  $\frac{P}{A}$  P max = 22174.24 kN  
= 2217424 Kg

$\sigma_{cbc}$  = 16.666667 Kg/cm<sup>2</sup>

$\therefore$  Area =  $\frac{2217424.2}{16.666667}$  = 13305 cm<sup>2</sup>

0.8 % of Min C/S Area. =  $\frac{13304.545 \times 0.8}{100}$  = 106.436 cm<sup>2</sup>

Assumed % of Longitudinal Reinforcement = 1.890  
 $\therefore$  Longitudinal Reinforcement Provided.

=  $\frac{22698.007 \times 1.89}{100}$  = 428.9923 cm<sup>2</sup>

Using 32 mm f bars,  $A_{s1}$  = 8.042 cm<sup>2</sup>

No of bars reqd: =  $\frac{428.992}{8.0424772}$  = 53.34 Nos

Provided steel 32 mm  $\phi$  bars, at 54 no of bars

Transverse Reinforcements:-

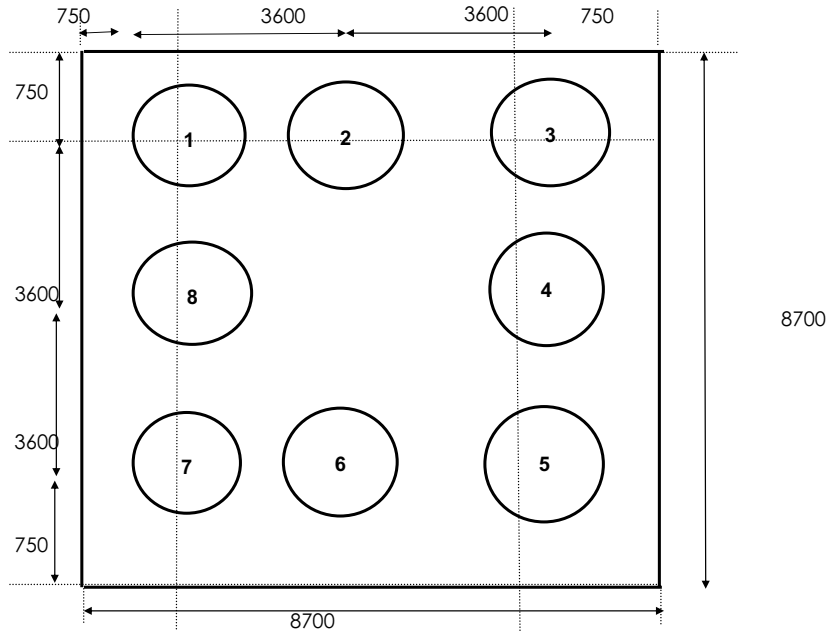
a) Diameter of Transverse Reinforcement =  $\frac{1}{4} \times 32$  = 8 mm

b) Minimum Diameter = 10 mm

Pitch of Transverse Reinforcement

- The least Lateral Dimension of the Column. = 1700 mm
  - 12 x 32 = 384 mm
  - Maximum Allowable Spacing = 300 mm
- So provide 10 mm f bars, at 300 mm C/C.

**Design of Pilecap & Pile**





constant "c"(from	2.000	1.980	2.260	2.000	2.260	2.000	2.120	2.200
constant "k"(from	0.200	0.200	1.150	0.200	1.150	0.200	0.930	0.930
"fc" c(M/R <sup>3</sup> )Kg/cm <sup>3</sup>	8.29	5.17	18.47	8.55	12.83	5.93	15.14	15.51
"fs" n k fc Kg/cm <sup>2</sup>	16.59	10.34	212.45	17.10	147.51	11.86	140.83	144.28
Per:stress in conc	16.67	16.67	25.00	25.00	25.00	25.00	22.22	22.22
Remarks	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe
Per: stress in steel	200	200	300	300	300	300	267	267
Remarks	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe
% increase in stre	1.00	1.00	1.50	1.50	1.50	1.50	1.33	1.33
Capacity of Pile kN	4520	4520	5650	5650	5650	5650	5650	5650
Remarks	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

#### Calculation of Pile Reinforcements.

##### Longitudinal Reinforcements:

( Vide cl:-306.2 & 306.3 of I.R.C :-21 : 2000 )

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

##### Transverse Reinforcements:-

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

##### Longitudinal Reinforcements:

$$\begin{aligned}
 \text{C/S Area of Pile Section.} &= 1.131 \text{ m}^2 = 11309.73 \text{ cm}^2 \\
 \text{a) 0.4 \% C/S Area.} &= \frac{11309.734 \times 0.4}{100} = 45.24 \text{ cm}^2 \\
 \text{b) Direct Stress} &= \frac{P}{A} \\
 P \text{ max} &= 3734.25 \text{ kN} = 373425 \text{ Kg} \\
 s \text{ cbc} &= 16.667 \text{ Kg/cm}^2 \\
 \backslash \text{ Area} &= \frac{373425.42}{166.66667} = 2240.55 \text{ cm}^2 \\
 \text{0.8 \% of Min C/S Area.} &= \frac{2240.5525 \times 0.8}{100} = 17.92 \text{ cm}^2 \\
 \text{Assumed \% of Longitudinal Reinforcement} &= 0.611 \\
 \therefore \text{Longitudinal Reinforcement Provided.} &= \frac{11309.734 \times 0.611}{100} = 69.09 \text{ cm}^2 \\
 \text{Using 20 mm f bars,} &As1 = 3.142 \text{ cm}^2 \\
 \text{No of bars reqd:} &= \frac{69.087}{3.1415927} = 21.99 \text{ Nos} \\
 \text{Provided steel 20 mm } \phi \text{ bars, at} &22 \text{ no of bars}
 \end{aligned}$$

##### Transverse Reinforcements:-

- Diameter of Transverse Reinforcement  

$$= \frac{1}{4} \times 20 = 5 \text{ mm}$$
  - Minimum Diameter = 10 mm  
Pitch of Transverse Reinforcement
    - The least Lateral Dimension of the Column. = 1200 mm
    - 12 x 20 = 240 mm
    - Maximum Allowable Spacing = 300 mm
- So provide 10 mm f bars, at 240 mm C/C.

#### Design of Pilecap :-

Provide min reinforcement of 0.06% in the pile cap top

$$\begin{aligned}
 \text{Ast required in the pile cap top} &= 1080 \text{ mm}^2 \\
 \text{Provide 16 mm dia bar @} &150 \text{ mm c/c in at pile cap top in both directions}
 \end{aligned}$$

Design of reinforcement in transverse dirn

This has be designed as cantilever bending due to pile load.

Load coming on three piles ( normal case)	8981	kN
Moment,M	14369.6	kNm
Ast required	46922.7	mm <sup>2</sup>

Design of reinforcement in longitudinal dirn

Moment,M	24697.8	kNm
Ast required	123095	mm <sup>2</sup>

Min Ast required = 0.85bd/fy 28693 mm<sup>2</sup>

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

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

Min Stirrup Reinforcement Reqd :-

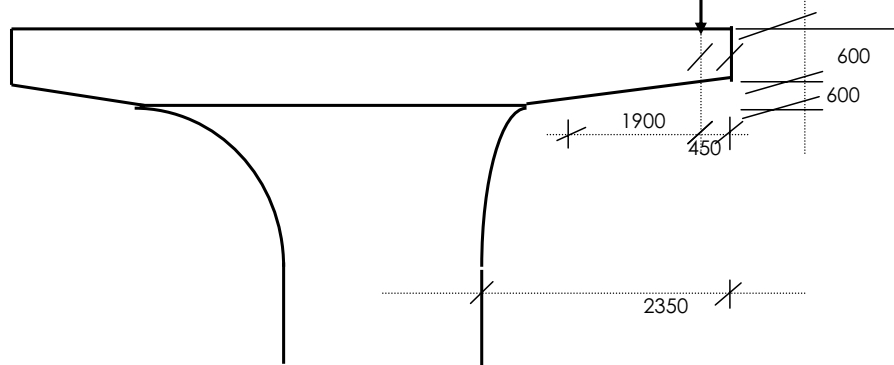
Assume 8 legged 16 mm diameter stirrups  
Sv 166.881 mm  
Provide 8 legged 16 mm diameter stirrups@ 150 mm C/c

3.225

**Design of Bed Block**

Girder + Deck slab  
Live Load  
SIDL

2394.67  
289  
274



1) Design of Cantilever Portion in the Transverse Direction.

Calculation of B.Moments at face of Pier due to'

a) Due to self weight

Width of Bedblock in long: dir <sup>n</sup>	=	2.5	m		
Cantilevered Length	=	2.35	m		
Average depth of cantilever part	=	0.9	m		
Volume	=	5.288	m <sup>3</sup>		
U.D.L /m Length	=	5.288	x	24	= 126.90 KN
C.G of Load	=	1.044	m		
D.Load Moment at face of Pier	=	126.9	x	1.044	= 132.54 KNm

b) Due to Dead load of Girder & Deck Slab

Moment at face of Pier = 4549.87 Tm

c) Due to S.I.D.Load

Moment at face of Pier = 520.98 Tm

d) Due to Vehicular Liveload

Moment at face of Pier = 548.83 kNm



Torsion due to Live load from one side = 71.47 kNm  
 Equivalent longitudinal moment due to torsion = 62.22 kNm

Σ Moments = 5814.43 kNm

Effective Depth Req'd "def" =  $\sqrt{\frac{M}{Q \times B}}$  Clear Co' = 50 mm  
 Bar Dia = 32 mm

deff reqd: =  $\sqrt{\frac{5814.43 \times 1000000}{1.90 \times 2500}}$   
 = 1106.39 cms < 1193.40 mm Hence O.K

deff provided = 1200 - 5 - 1.6 = 1193.40 mm

Ast reqd =  $\frac{M}{\sigma_{st} \times j \times d}$

∴ Ast reqd =  $\frac{5814.43 \times 1000000}{200 \times 0.88 \times 1193.40}$  = 27683 mm<sup>2</sup>

As1 = 804 cm<sup>2</sup>

No of bars Reqd: =  $\frac{27683}{804}$  = 34 Nos

Spacing of Bars =  $\frac{2500}{34}$  = 73 mm

So provide two layers 32 dia bars, 17 nos

2) Check for Shear at pier face.

Calculation of S.Force at face of Pier due to

a) Due to self weight

deff = 1.1934 m  
 Shear due to self wt: = 48.18 KN

b) D load of Girder ,DeckSlab = 2394.67 KN

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

d) Due to LL = 289 kN

Equivalent shear due to tc = 45.74 kN

S Shear = 3051.65 kN

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

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

∴  $\tau$  =  $\frac{3051.6 \times 1000}{2500 \times 1193.40}$  = 1.0 Mpa

Maximum Permissible Shear Stress :-

$\tau_{max}$  = 2.3 Mpa which ever is less

fck = 35 N/mm<sup>2</sup>

Calculation of permissible Shear Stress :- ( Vide cl - 304.7.3.1 of I.R.C:-21-2000 )

$\rho$  =  $\frac{100 \times A_s}{B \times d}$  As = 27682.72 mm<sup>2</sup>

$\rho$  =  $\frac{27682.7 \times 100}{2500 \times 1193.4}$  = 0.928

∴  $\tau_c$  = **0.4** N/mm<sup>2</sup>

Since  $\tau > \tau_c$  Shear Reinforcement Reqd:

Shear Reinforcement Reqd :-

( Vide cl - 304.7.4.2 of I.R.C:-21-2000 )

Asw =  $\frac{V \times s}{\sigma_{st} \times d}$

V = 1846.41 kN  
 s, spacing = 150 mm (assumed)

∴ Asw =  $\frac{1846.41 \times 1000}{200 \times 1193.4} \times 150$  = 1160.4 mm<sup>2</sup>

Using 12 mm dia stirrups 12 Legged at spacing 150 mm.  
 Asw = 1357.2 cm<sup>2</sup> > 1160.4 mm<sup>2</sup>  
 Hence O.K

B) Design of Cantilever Portion in the Longitudinal Direction.

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