INDIAN INSTITUTE OF TECHNOLOGY

Tirupati, A.P. -517619

LAB REPORT ON

ADVANCED DESIGN OF CONCRETE STURCTURES

TERM PROJECT

DETAILED CALCULATION REPORT

ANALYSIS AND DESIGN OF CABLE STAYED BRIDGE.

<u>FACULTY INCHARGE</u>

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1.0) INTRODUCTION

A cable-stayed bridge is a type of suspension bridge in which the bridge deck is directly supported by cables (or stays) that are connected to one or more vertical pylons or towers.

- It has higher stiffness (Bending & Torsional stiffness)
- Volume of traffic needs to be catered
- Self-supporting structure
- Concrete compressive strength can be utilized to its maximum
- Deck undergoes more compression during live load which is very unlikely in other kind of bridges

2.0 PROJECT DISCRIPTION

NAME OF THE PROJECT – ANALYSIS AND DESIGN OF CABLE-STAYED BRIDGE

BRIDGE CONFIGURATION – TWO NEEDLE PYLONS, SEMI-FAN SHAPED CABLES, AND PSC BOX GIRDER.

SPAN OF THE BRIDGE – 238M

LOCATION OF SEISMIC ZONE – II

LOCATION OF SOIL – HARD ROCK OR HARD SOIL

3.0 PRELIMINARY DESIGN

Cable stress (dead load and live load) should be less than or equal to 0.4% of the cable strength

Mid tower displacement less or equal $\frac{H}{350}$

Deck displacement less or equal $\frac{L}{540}$

Deck stiffeners higher stiffnes fewer will be the cables

Tan=18/3

Thetsa= 54.16

Deck depth h=L/50 = 112/50 = 2.24m

Back span to main span ratio (0.45-0.55) = back span/main span ratio

Rule for most economical for a pylon of slenderness = h/l < 0.3 = 37.2/112 = 0.334

Cable spacing -5-15m 6m probably

Cable stay angle theta = 25-65

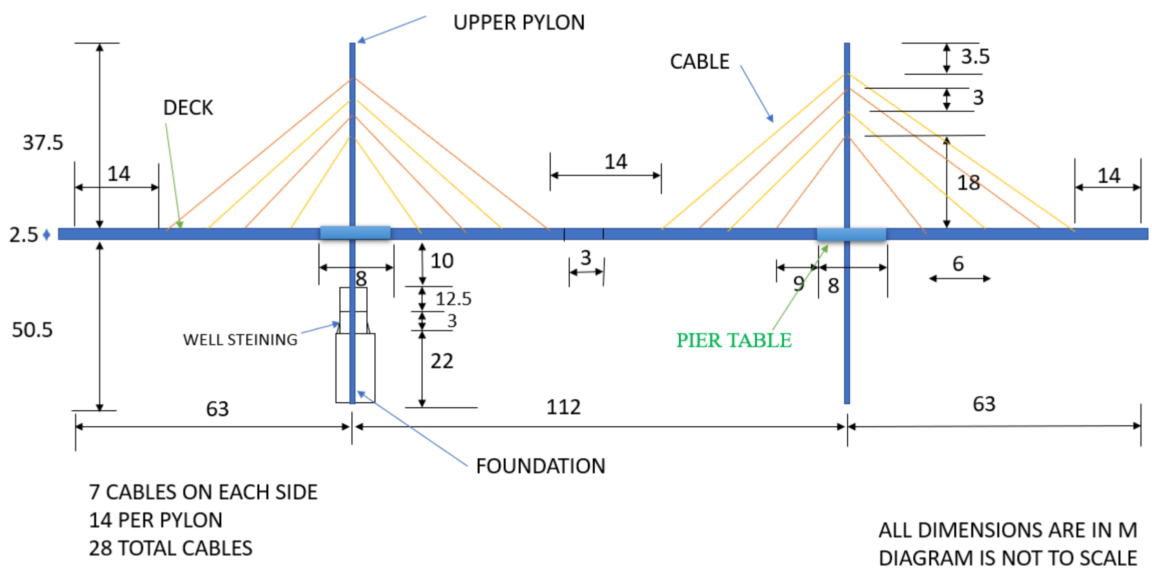
Number of cables 3-6 pairs depends on deck stiffness

7 cables

Anchor position 0.68L – 0.78l = 0.68 x 115 = 78.2 m

Length of the peir table is 8m

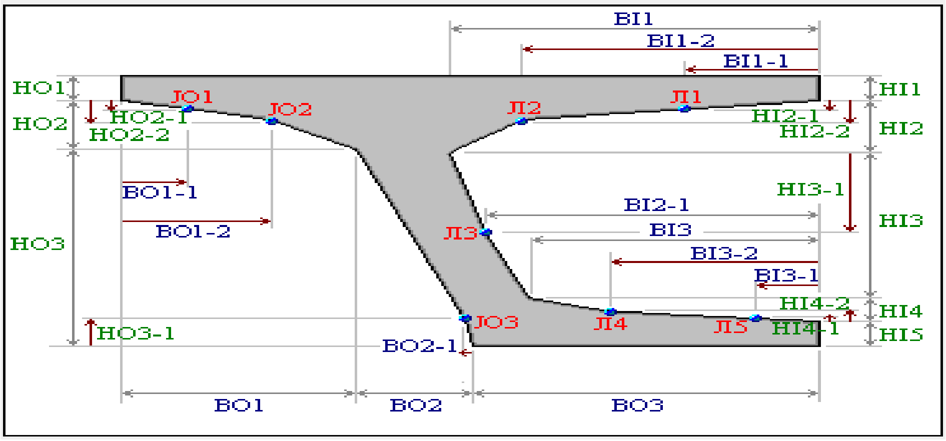
DIMENSIONS OF CABLE BRIDGE.



4.0 SECTIONS AND MATERIALS USED

4.1 SECTIONS USED FOR BRIDGE

4.1.1 SECTION FOR THE DECK



4.1.2 PSC SECTION AT MID SECTION

Inner Section Dimensions

Label	Value (m)		Label	Value (m)
HI1	0.2		BI1	4.5
HI2	0.75		BI1-1	2.5
HI2-1	0.1		BI1-2	4.3
HI2-2	0.2		BI2-1	4.2
HI3	1.25		BI3	3.75
HI3-1	0.8		BI3-1	1.625
HI4	0.1		BI3-2	3.25
HI4-1	0.0		BI4	0.0
HI4-2	0.0			
HI5	0.2			

Outer Section Dimensions

Label	Value (m)		Label	Value (m)
HO1	0.2		BO1	3.0
HO2	0.3		BO1-1	1.0
HO2-1	0.05		BO1-2	2.0
HO2-2	0.1		BO2	1.0
HO3	2.0		BO2-1	0.0
HO3-1	0.2		BO3	4.0

4.1.2 PSC SECTION AT PIER LOCATION

INNER SECTION DIMENSIONS

Label	Value (m)		Label	Value (m)
HI1	0.2		BI1	4.45
HI2	0.6		BI1-1	2.5
HI2-1	0.1		BI1-2	4.3
HI2-2	0.2		BI2-1	4.2
HI3	1.05		BI3	3.7
HI3-1	1.05		BI3-1	1.6
HI4	0.1		BI3-2	3.2
HI4-1	0.0		BI4	0.0
HI4-2	0.0			
HI5	0.25			

OUTER SECTION DIMENSIONS

Label	Value (m)		Label	Value (m)
HO1	0.2		BO1	3.0
HO2	0.3		BO1-1	1.0
HO2-1	0.05		BO1-2	2.0
HO2-2	0.1		BO2	1.0
HO3	2.0		BO2-1	0.0
HO3-1	0.2		BO3	4.0

4.1.3 PYLON SECTION

Rectangular section is provided for the pylon

- **Lower pylon**
- Breadth = 4m
- Depth = 2m
- **Upper pylon**
- Breadth = 1m
- Depth = 4m

4.1.4 WELL CAP

- Solid round
- D = 8m

4.1.5 WELL STEINING TOP

- D = 8m
- Thickness of well = 1m

4.1.6 WELL STEINNING BOTTOM.

- D = 10m

- Thickness of well = 2m

4.1.7 ANCHOR CABLE

- D = 0.1m

4.1.8 WELL STEINING TAPERED

- D = 8m
- Thickness of well = 1m
- Bottom d = 10m and thickness of well 2m

4.2 MATERIALS USED

- Lower pylon concrete M60
- Upper pylon concrete M60
- Deck M60
- Well M40
- Well steining M30
- Anchor cable - steel
- Tendons - steel

5. LOADS CALCULATION

5.1 DEAD LOAD

- Density of reinforced concrete – 2kn/m3
- Density of steel for cables and tendons is 7850 kg/m^3
- Density of wearing course is 2300 kg/m^3
- Railings 1.5 kN/m
- Light towers 1.5 kN per unit
- Median 6 kN/m
- Footpath 4.0 kN/m^2

5.2 VEHICLE LOADING

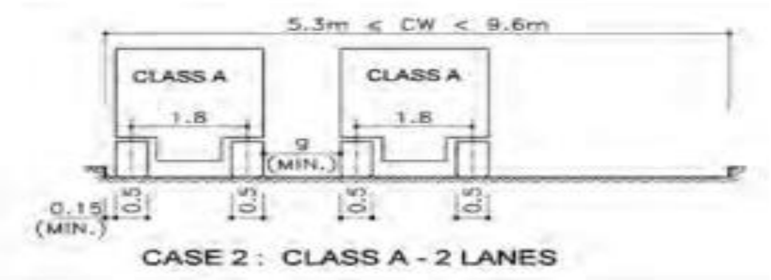
- Width of the PSC box girder =16m
- Centre of the box girder from the edge -=8m
- Width of the carriage way = 16 – 0.5 – 0.5 – 0.5 = 14m
- Width of the crash barrier = 0.5m
- Width of the median = 0.5m
- Thickness of the wearing coat = 0.075m

5.2.1 FOR CLASS A LOADING WITH TWO LANES

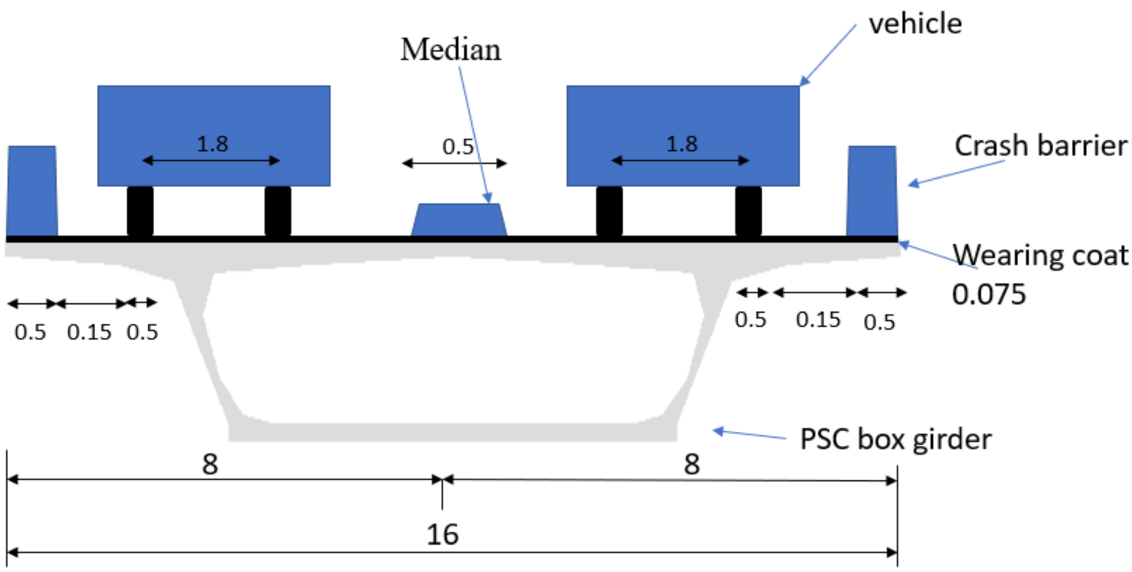
Table 6 A - IRC 6 2017

Wheel spacing is 1.8m

Minimum distance between the crash barrier and outer wheel of the vehicle is 0.15m



5.2.2 CLASS A LOADING



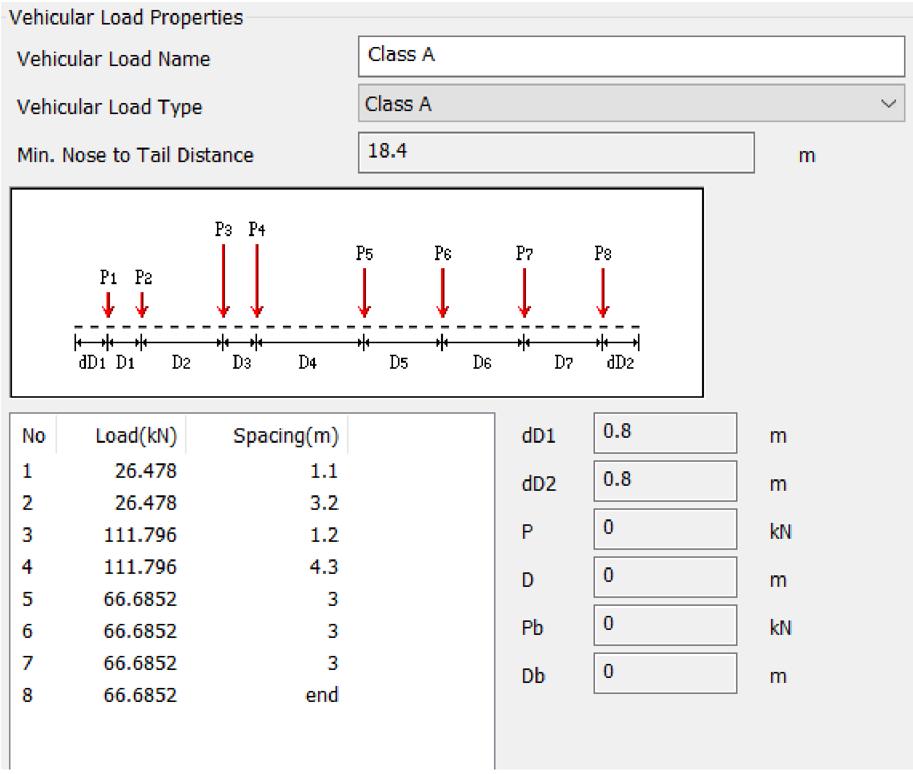
- Eccentricity E= 0.5 + 0.15 + 0.25 + 0.9 = 1.8m

  4.25 – 1.8 = 2.45 m

- c/c between the center of the vehicle and the centre of median

  0.15 + 0.25 + 0.9 = 1.3m

  c/c distance 8 – 1.3 = 6.7 m

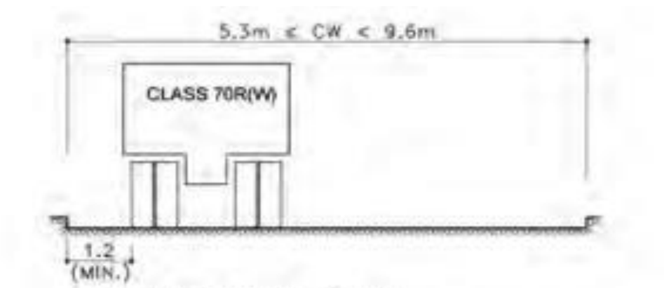


5.2.3 FOR CLASS 70R LOADING WITH ONE LANES

TABLE 6 A - IRC 6 2017

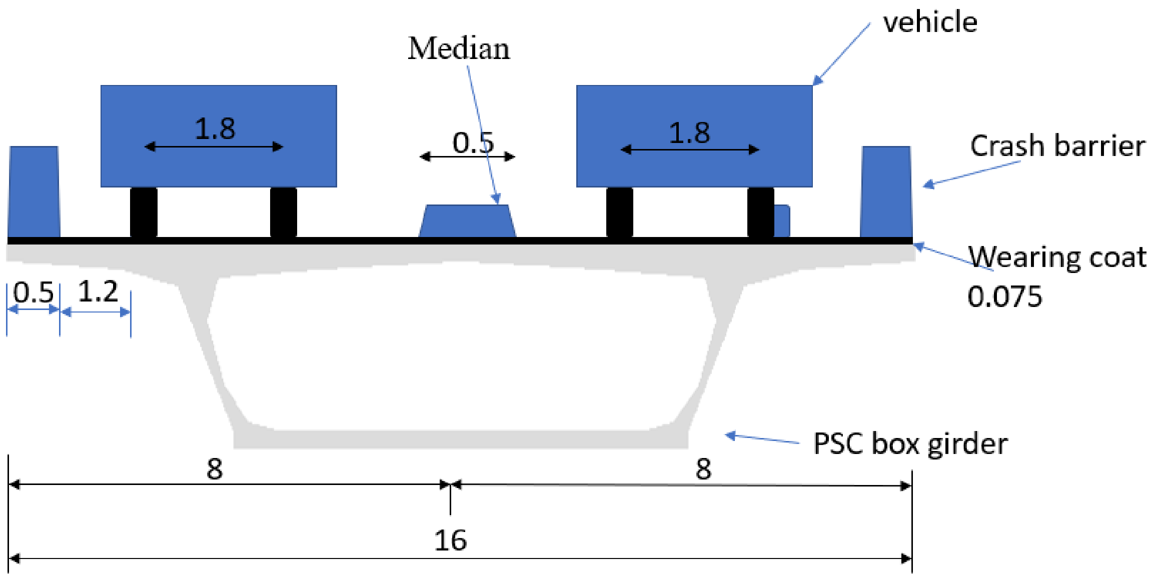
Wheel spacing is 1.8m

Minimum distance between the crash barrier and outer wheel of the vehicle is 0.15m



CASE 1 : CLASS 70R (W)

5.2.4 CLASS 70 R LOADING



- Eccentricity =0.15 + 0.25 +1.8 + 0.25 + 1.2 +0.25 +0.9 = 4.8 m

  8-4.8 = 3.2m

- c/c between the centre of the vehicle and the centre of median

  = 1.2 + 0.42 + 1.03 = 2.65 m

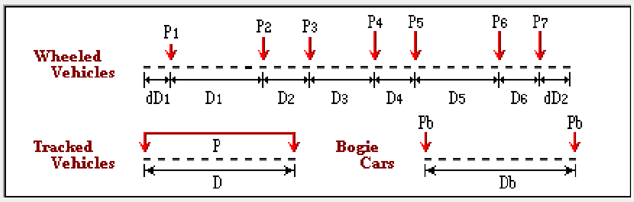
  = 8 – 2.65 = 5.35 m

Standard Name

IRC:6 Standard Load

Vehicular Load Properties

Vehicular Load Name	Class 70R
Vehicular Load Type	Class 70R
Min. Nose to Tail Distance (Wheeled)	30 m
Min. Nose to Tail Distance (Tracked)	90 m



No	Load(kN)	Spacing(m)
1	78.4532	3.96
2	117.68	1.52
3	117.68	2.13
4	166.713	1.37
5	166.713	3.05
6	166.713	1.37
7	166.713	end

dD1	0.61	m
dD2	0.91	m
P	686.4655	kN
D	4.57	m
Pb	196.133	kN
Db	1.22	m

5.2.5 BREAKING FORCE

- From IRC 6 2017 CI 209.6
- Breaking force is taken as 20 percentage of live load
- Maximum live load on the bridge is 166.13 KN
- Breaking force = 0.2 x 166.13 = 33.226 KN

5.3 WIND LOAD

Wind load calculations (Without live load) IRC 6 2017 CI 209.3.5

5.3.1 Wind Load Without Live Load

5.3.2 Transverse Wind Load (Ft)

$$Ft = Pz \times A \times G \times CdFt$$

- **Pz**: Hourly mean wind pressure in N/m^2 at deck height (from Clause 209.3.2)
- **A**: Exposed area of the superstructure in elevation (m^2 per meter length)
- **G**: Gust Factor (from Clause 209.3.3) – usually taken as 2.0 for highway bridges
- **Cd**: Drag Coefficient (typical value = 1.28 for standard deck shapes)

$$Ft = 508.11 \times 4.05 \times 2 \times 1.28 = 5268 N/m = 5.27 kN/m$$

5.3.3 Longitudinal Component (Fl)

- Per IRC:6-2017, Clause 209.3.5:
- $Fl = 0.25 \times Ft = 1.32 kN/m$

5.3.3 VERTICAL/UPLIFT LOAD (FV)

- From **Lift formula**:
- $Fv = Pz \times Av \times G \times CL$
- Where:
- Av: Plan area (deck width × unit length)
- CL : Lift coefficient (0.75 for standard decks
- $Fv = 508.11 \times 16 \times 2 \times 0.75 = 12.2 kN/m$

5.3.4 WIND LOAD WITH LIVE LOAD

Live load (vehicles) must be considered when wind speed is **below 36 m/s**. Vehicles are subject to transverse wind pressure and drag as well.

- Deck height = 14.55 m → Pz = 480 N/m²
- Deck height = 16.0 m → Pz = 510 N/m²

Superstructure Wind Load:

- $F_t = 480 \times 4.05 \times 2 \times 1.28 = 5 kN/m$
- $F_l = 25\% \times F_t = 1.25 kN/m$
- $F_v = 510 \times 16 \times 2 \times 0.75 = 12.24 kN/m$

5.3.5 WIND LOAD ON VEHICLES (LIVE LOAD)

$$F_t = P_z \times A \times G \times C_d$$

Given:

- Pz = 510 N/m²
- A = 1.45 m²/m (projected area of vehicles per meter)
- G = 1 (since live load gust considered normal)
- Cd = 1.28

$$F_t = 510 \times 1.45 \times 1 \times 1.28 = 1.893 \approx 1.9 kN/m$$

$$F_l = 25\% \times F_t = 0.475 \approx 0.5 kN/m$$

5.3.6 WIND LOAD ON SUBSTRUCTURE (PIERS/COLUMNS)

Clause 209.3.6 gives method for **wind on substructures**, such as piers, columns.

Given:

- P = 463.7 N/m² (wind pressure at lower height)
- A = Area of pier (varies)
- Cd = 1.05 – 1.425 (drag coefficient depending on shape)
- G = 2.0

Example Calculations:

Ft (substructure vertical face):

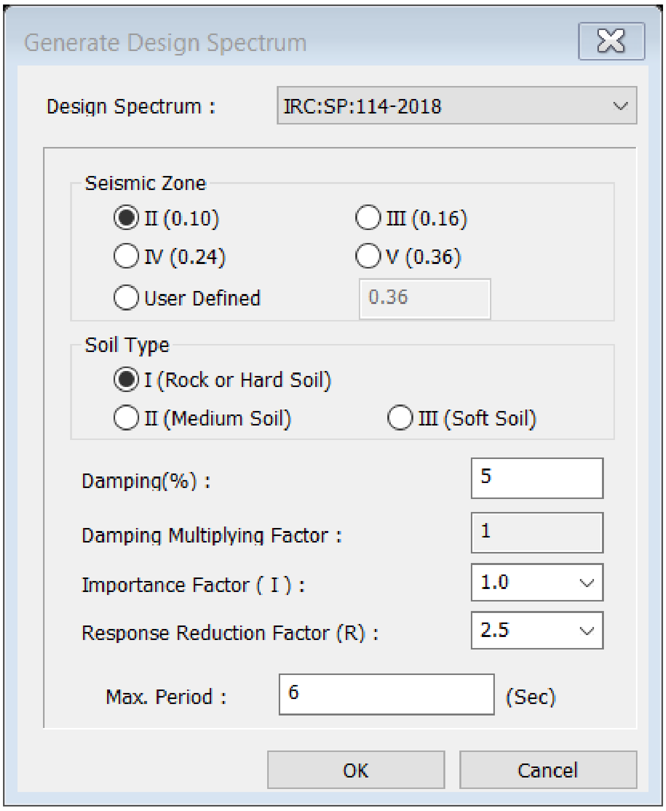
$$F_t = 463.7 \times 2 \times 1.05 \times 2 = 1.947 \approx 1.95 kN/m$$

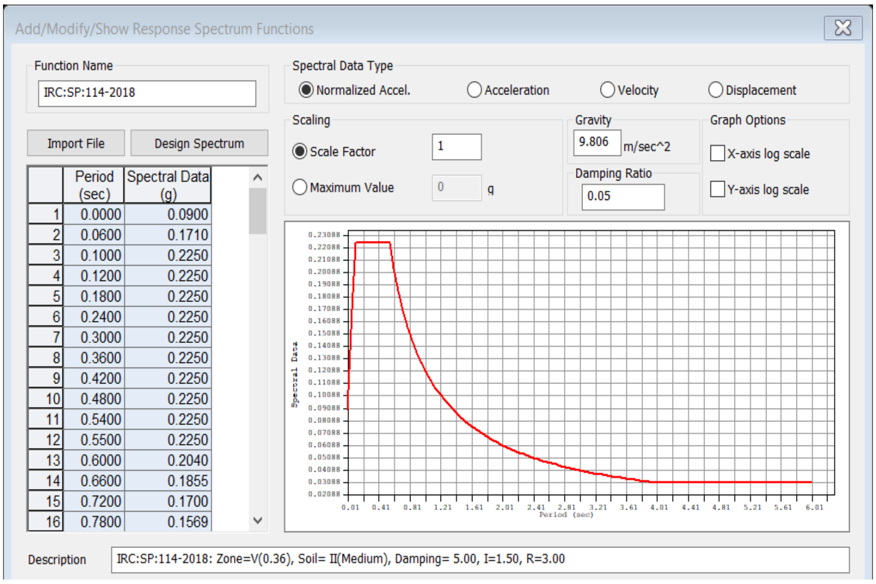
Fl (larger pier face): $F_t = 463.7 \times 4 \times 1.425 \times 2 = 5.28 kN/m$

5.4 SEISMIC LOAD

- **Design Spectrum**: IRC:SP:114-2018
- **Seismic Zone**: II (Zone factor, Z = 0.10)
- **Soil Type**: Type I (Rock or Hard Soil)
- **Damping (%)**: 5%
- **Damping Multiplying Factor**: 1.0
- **Importance Factor (I)**: 1.0
- **Response Reduction Factor (R)**: 2.5
- **Maximum Period**: 6 seconds

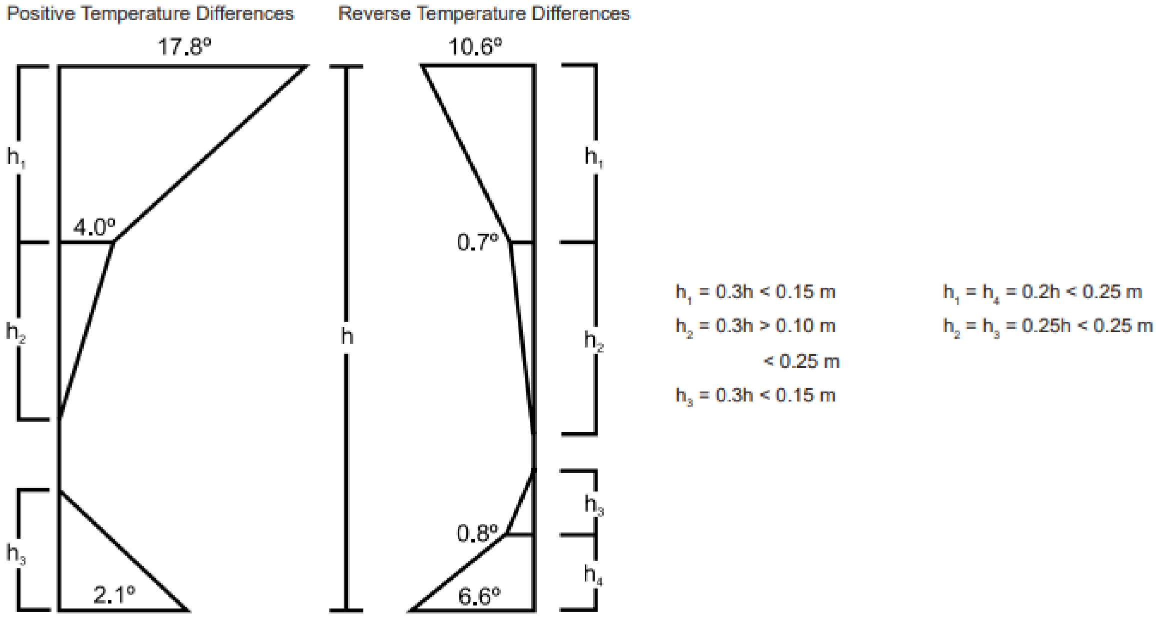
5.4.1 Seismic definition in Midas civil.





5.5 TEMPERATURE LOAD ON THE GIRDER

From IRC 6 2017 CI 215.4



Typical depth of segment =2.5m

Assumed Parameters:

- Typical depth of segment: h=2.5m
- For positive temperature difference:

$h1=0.3h<0.15m \Rightarrow h1=0.15m$

$h2=0.3h=0.75m >0.10m$ and $<2.5m \Rightarrow h2=0.25m$

$h3=0.3h<0.15m \Rightarrow h3=0.15m$

For reverse temperature difference

$h1=h4=0.2h=0.5m>0.25m \Rightarrow h1=h4=0.25m$

$h2=h3=0.25h=0.625m>0.25m$

Temperature Assumption for MIDAS Input:

- Max Temp = 50∘C
- Min Temp = 20∘C

Mean Temp =

- Mean=T_{max}+T_{min}/2 =250+20= 35∘C
- Temperature for input in Midas (°C → °F):
- T∘F=T∘C×59+32
- Top Temperature Input
- $T_{umax}=T_{max}-T_{mean}+10=50-35+10=25$∘C
- Bottom Temperature Input
- $T_{umin}=T_{min}-T_{mean}-10=20-35-10=-25$∘C

5.5.1 IN FC IN MIDAS CIVIL TG+

- h_1 0 h_2 0.15 T1 63.68F T2 39.2F
- h1 0.15 h20.4 T139.2 T20F
- h1 0 h20.15 T1 35.78 T2 F
- h1 0 h2 0.25 T1 10.6 T2 0.7 TOP
- h1 0.25 h2 0.25 T1 6.6 T2 0.8 TOP
- h1 0 h20.25 T16.6 T20.8 BOTTOM
- h1 0.25 h2 0.5 T1 0.8 T2=0 BOTTOM

5.7 CREEP AND SHRINKAGE.

Concrete Grade / Name:

- M60

Code:

- INDIA (IRC:112-2011)

Compressive Strength of Concrete at 28 Days:

- 60 kN/m²

Relative Humidity of Ambient Environment:

- 70%

Notional Size of Member (h): 1.0 m

$$h = \frac{2\times A_c}{u}$$

Creep Function Data Type:

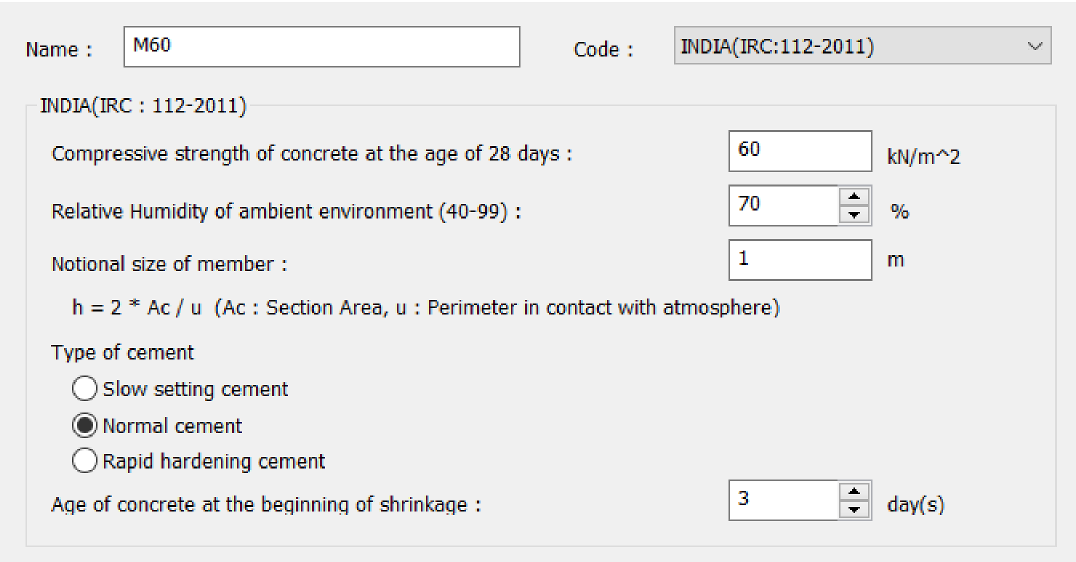
- Creep Coefficient (selected option)
- Other option: Shrinkage Strain

Start Loading Time:

- 10 days

End Loading Time:

- 10,000 days



6.0 LOAD COMBINATIONS.

6.1. LIMIT STATE OF STRENGTH.

Table 4 of IRC:6-2017

Load Type	Leading ($\gamma f1$)	Accompanying ($\gamma f2$)
DL, SIDL	1.35	1.0
LL	1.5	1.15
Wind	1.5	0.9–1.5 (case based)
Temp	0.9–1.5	
Braking Force	1.5	
Seismic Load	1.2	

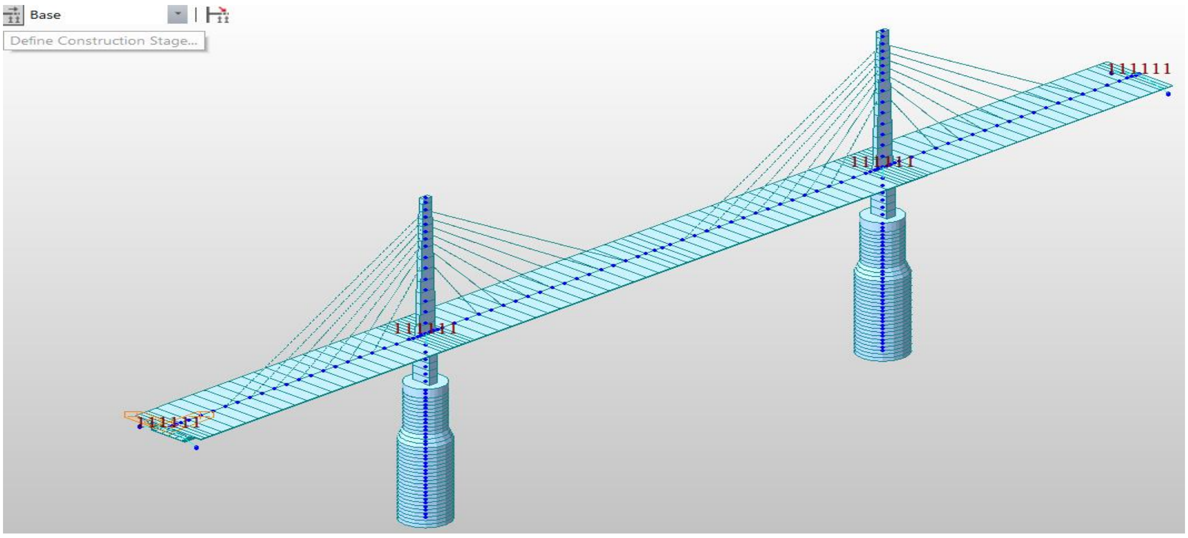
6.2 LIMIT STATE OF SERVICEABILITY.

Serviceability Load Factors (IRC:6-2017 Table 4)

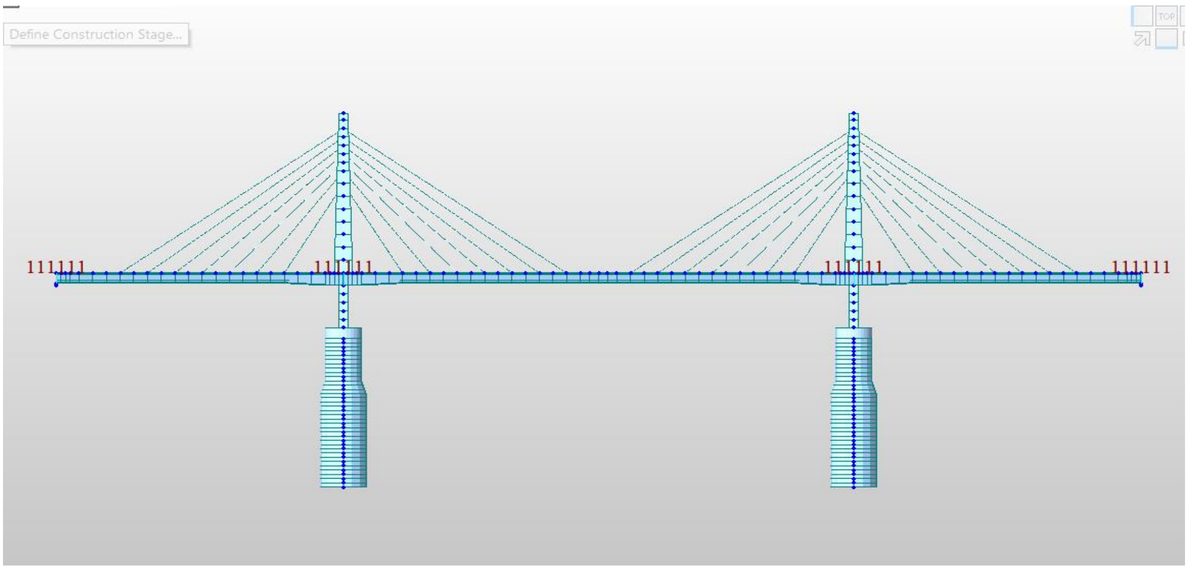
Load Type	Partial Safety Factor (γf)
Dead Load (DL)	1.0
SIDL Fixed/Var.	1.0
Live Load (LL)	1.0
Wind Load (WL)	1.0
Temperature	1.0
Braking Force	1.0
Seismic (EQ)	1.0

7.0 MODELING OF CABLE BRIDGE IN MIDAS CIVIL

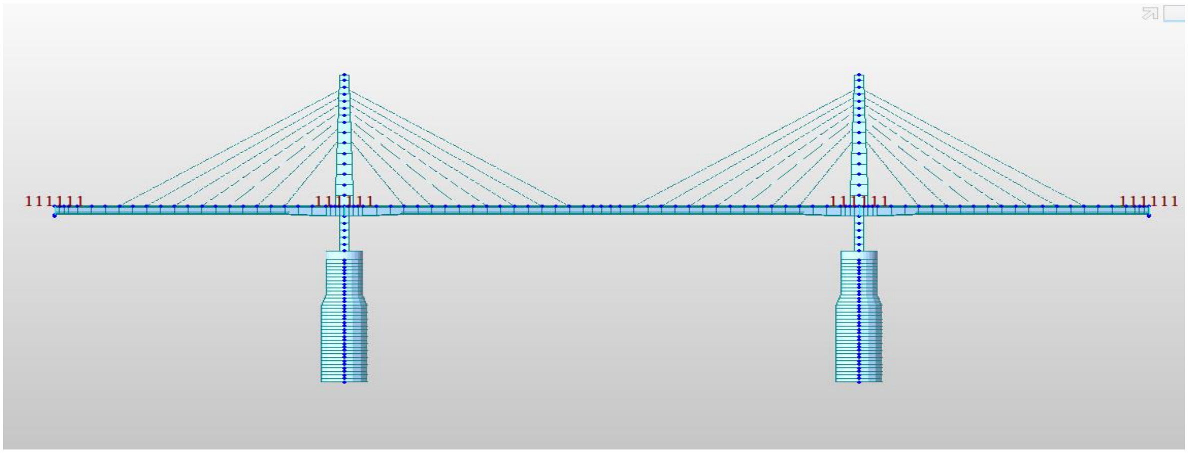
ISOMETRIC VIEW



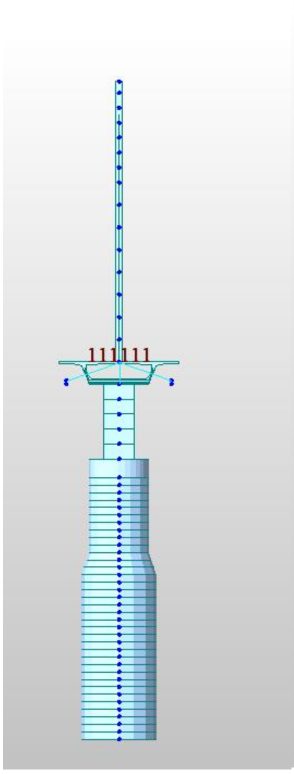
BACK VIEW

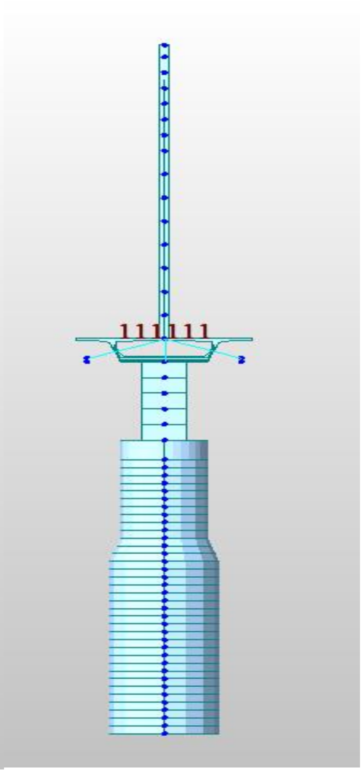


FORNT VIEW

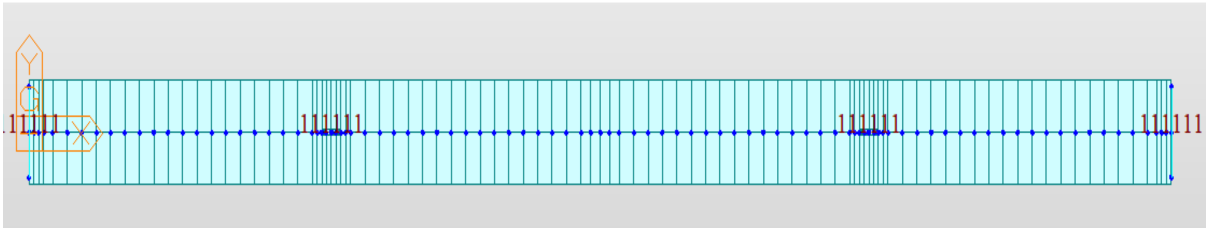


LEFT SIDE VIEW RIGHT SIDE VIEW



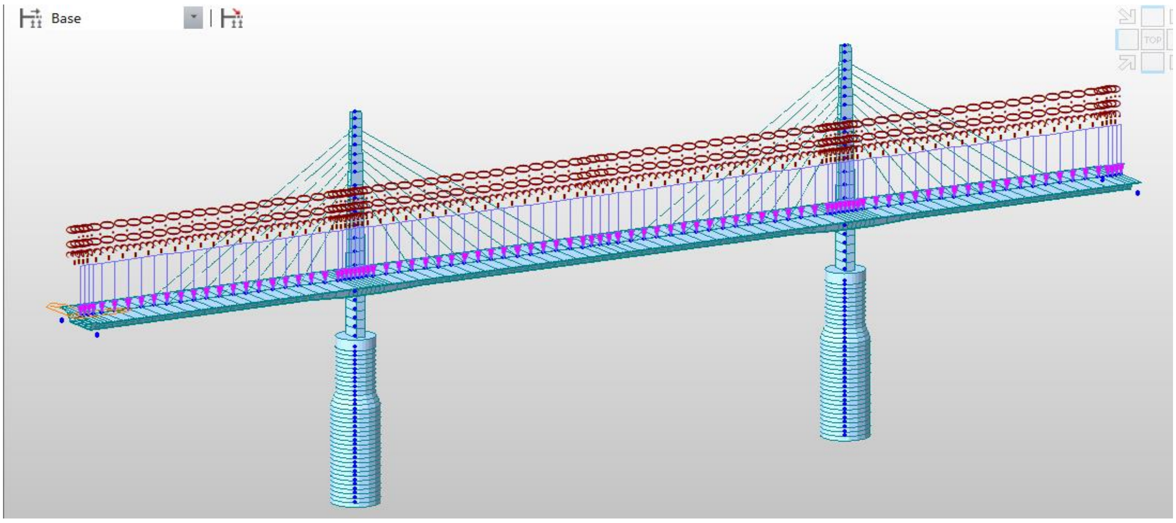


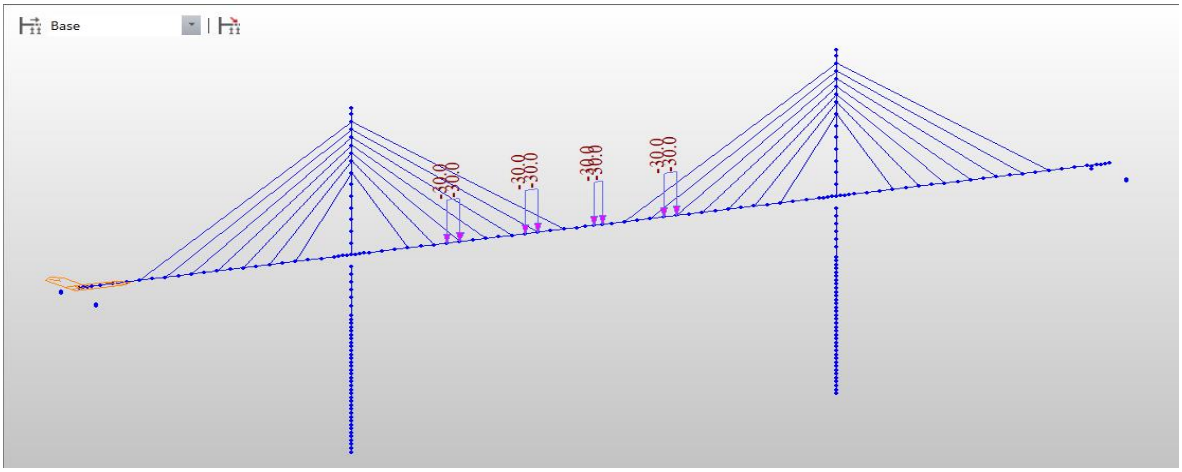
TOP VIEW



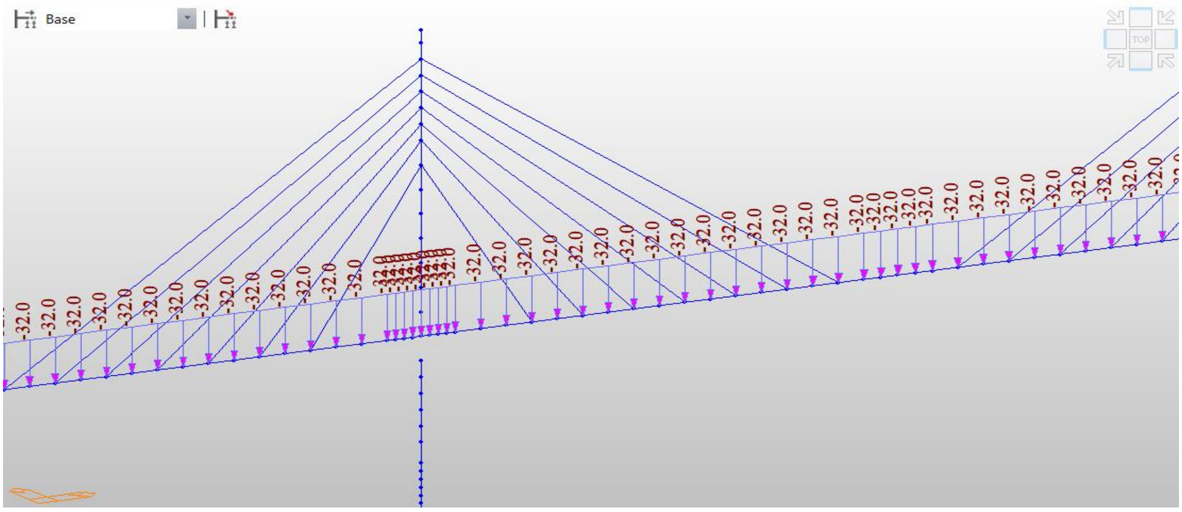
7.2 APPLICATION OF LOADING ON MODEL IN MIDAS CIVIL

SIDL FIXED

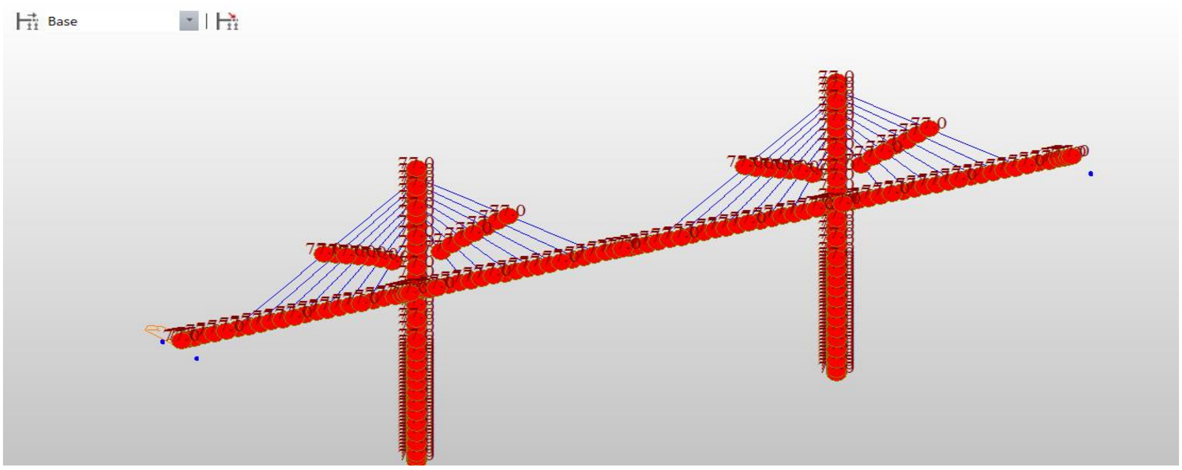




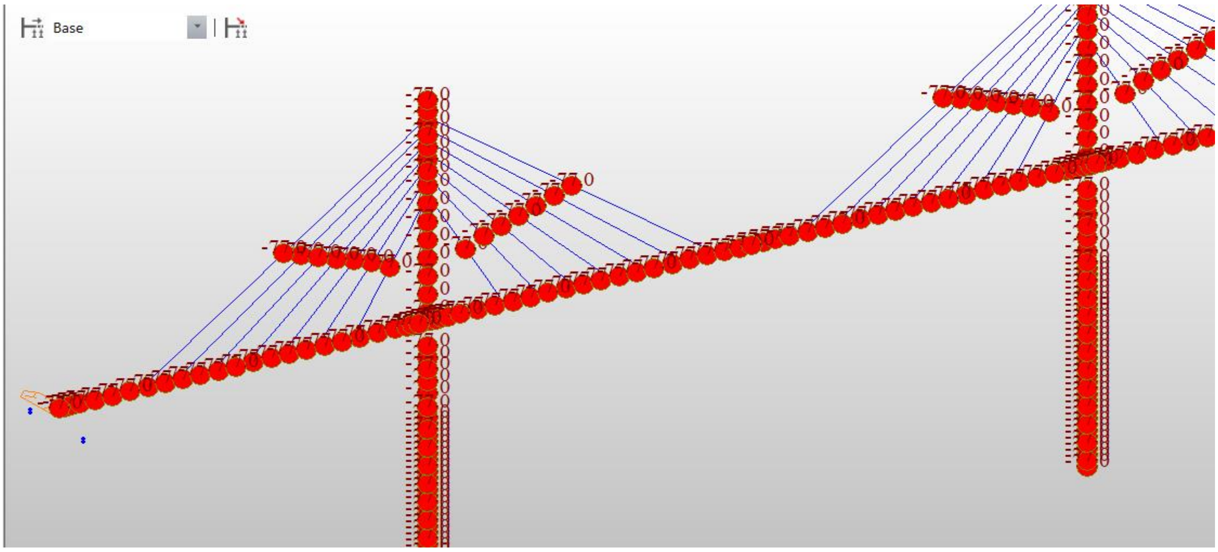
SIDL VARIABLE



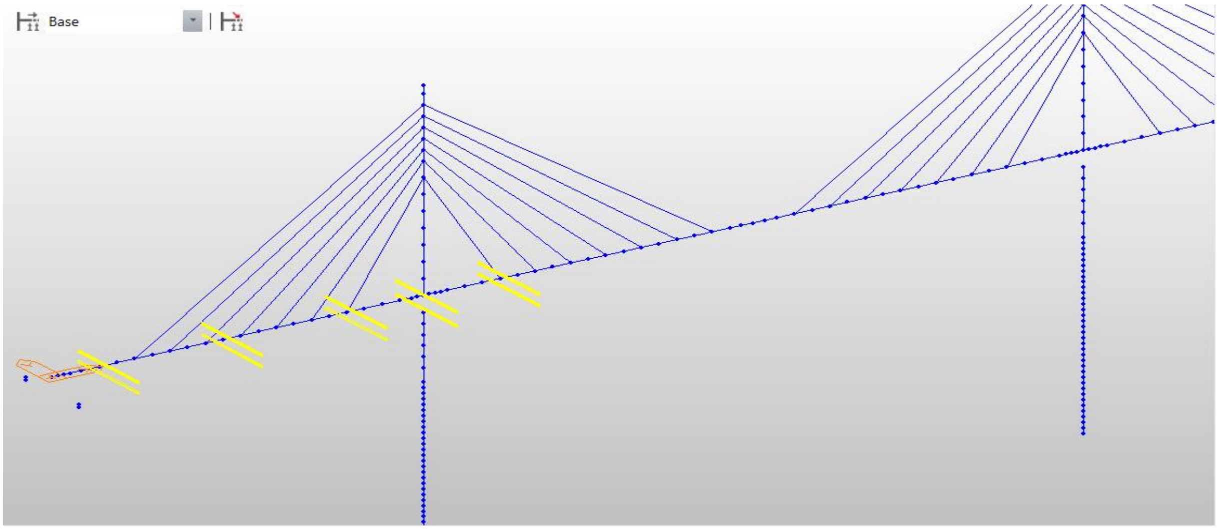
TU+



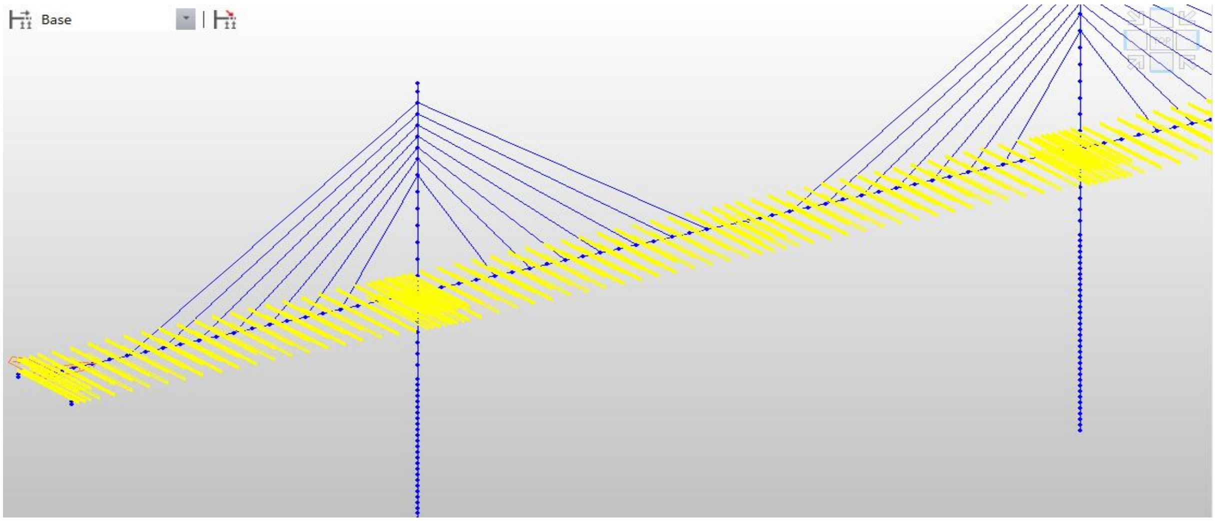
TU-



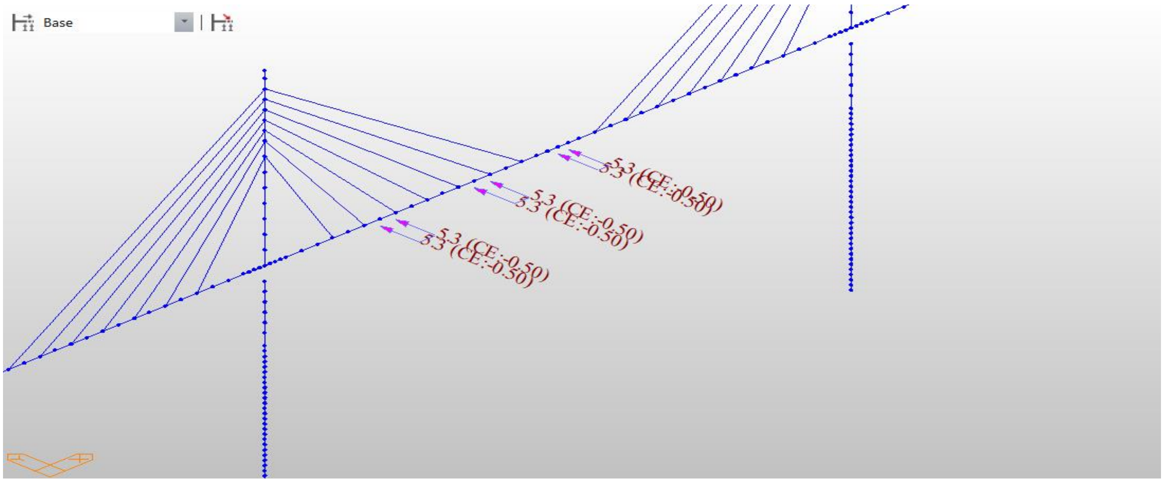
TG+



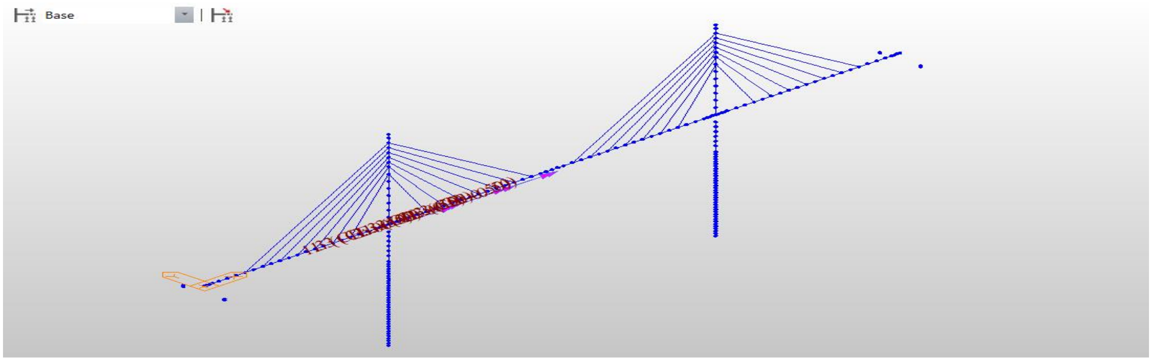
TG-



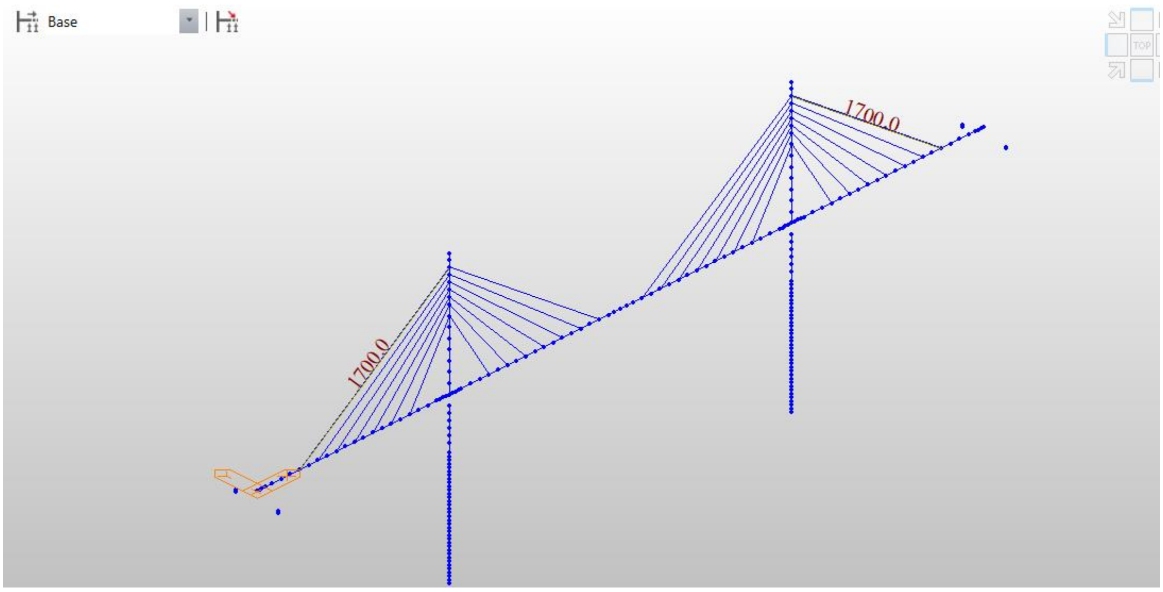
WT SUP WOLL



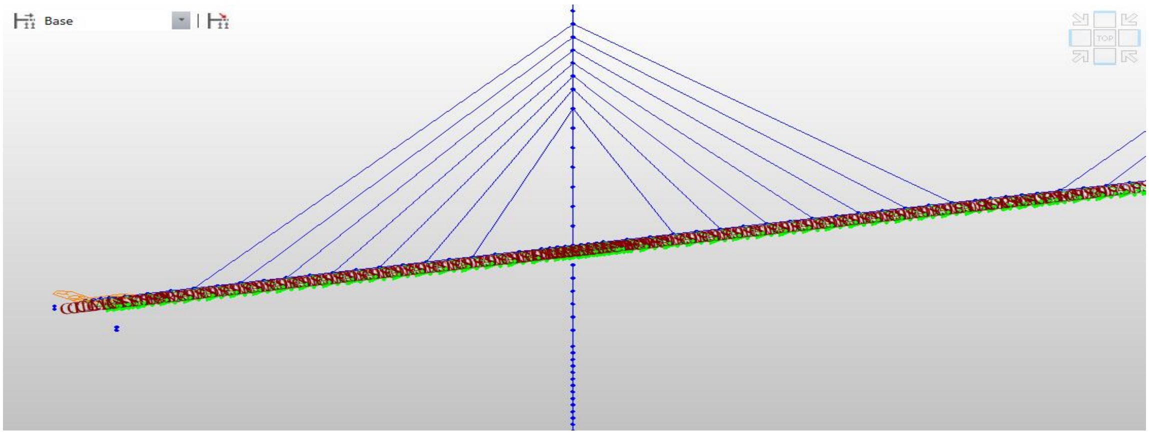
WL SUP LOL



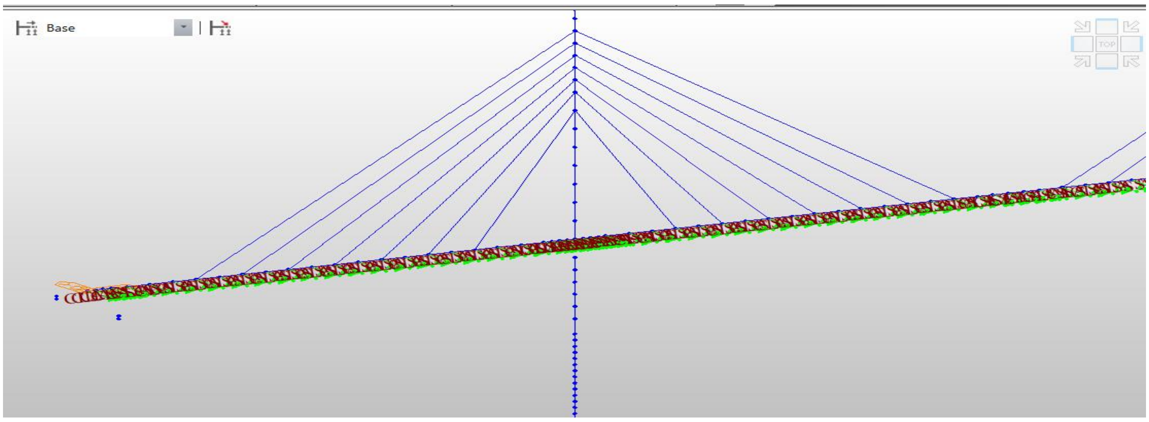
PRETESNIO ON TENDON

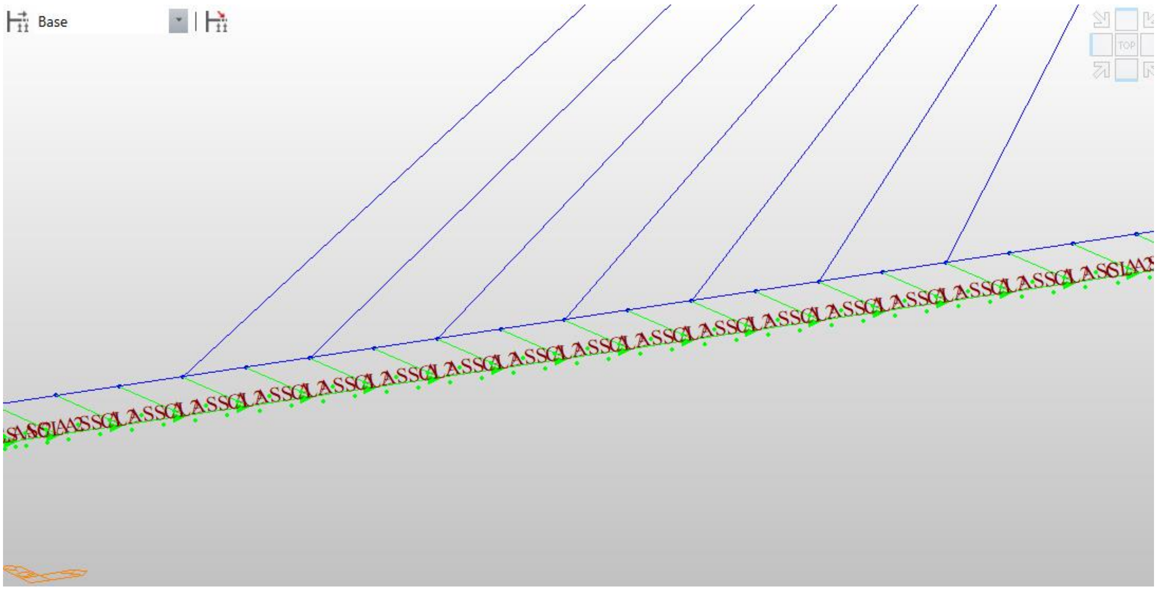


TRAFFIC LINE LANE CLASS 70R



LANE 1 CLASS





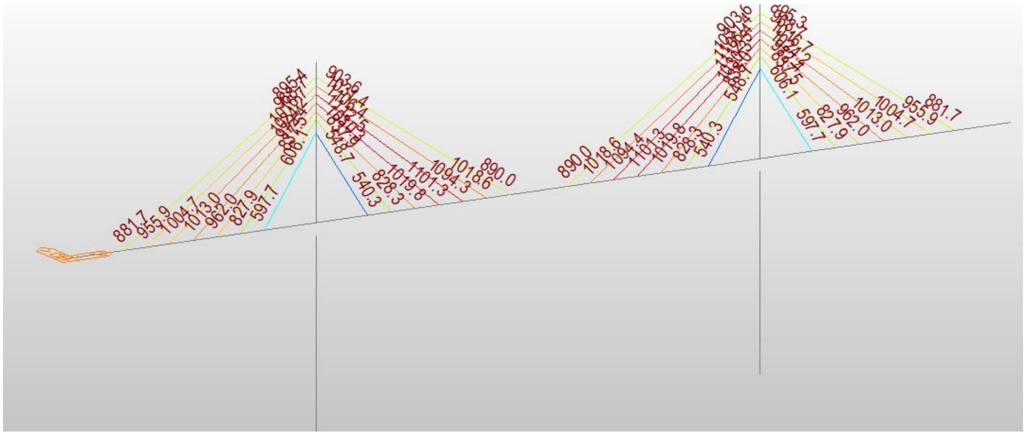
8.0 ANALYSIS AND RESULTS

- STATIC ANALYSIS
- STATIC NON LINEAR – PUSHOVER ANALYSIS.
- DYNAMIC – RESPOSNSE SPECTRUM ANALYSIS.
- CONSTRUCTION STAGE ANALYSIS OF THE BRIDGE

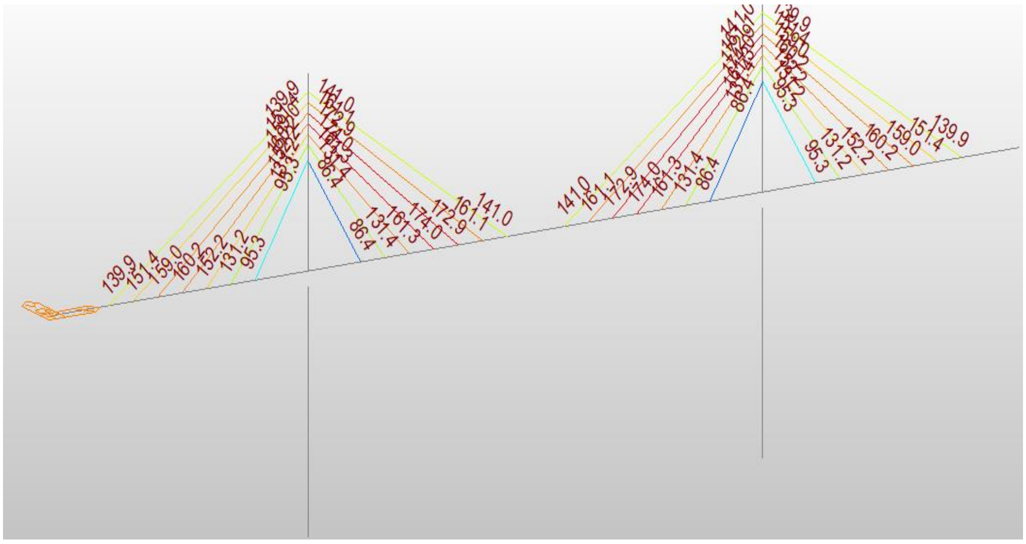
Response spectrum analysis and results.

11.1 Forces in cables

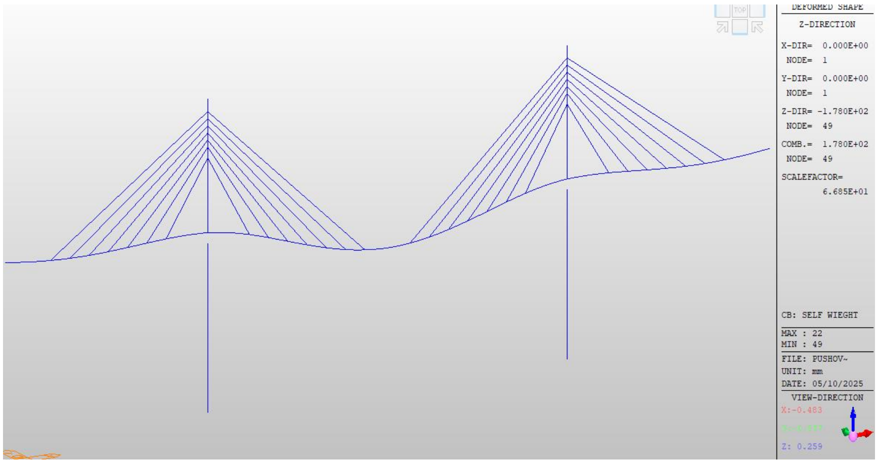
Self weight



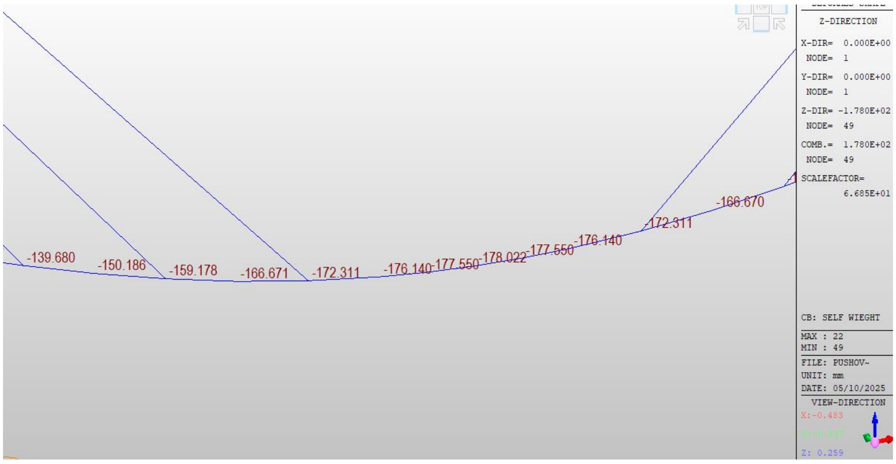
SIDL Fixed.



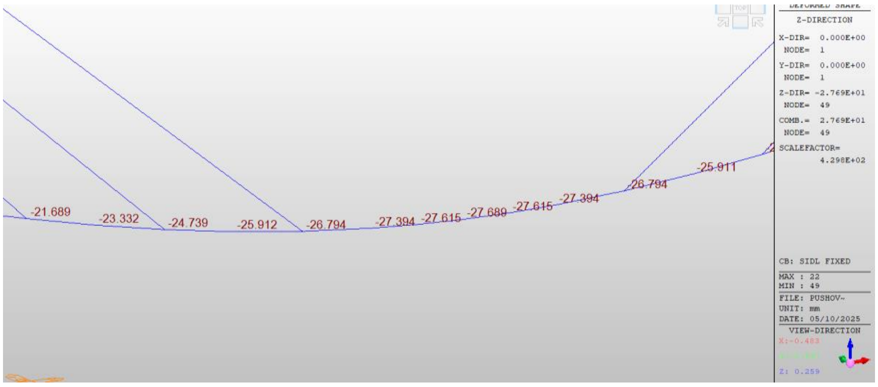
8.2 DEFLECTIONS IN CABLE BRIDGE



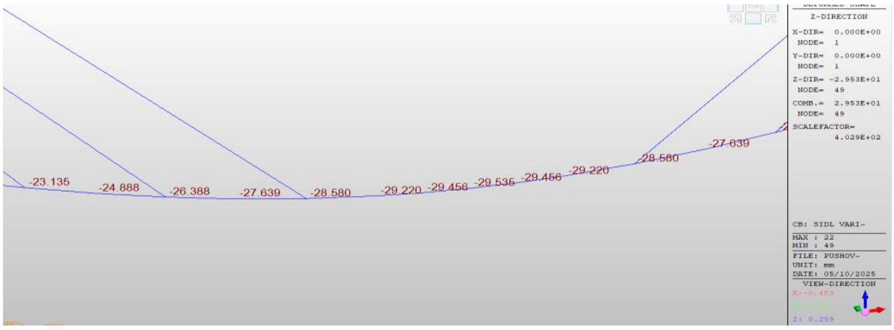
Self weight



SIDL fixed

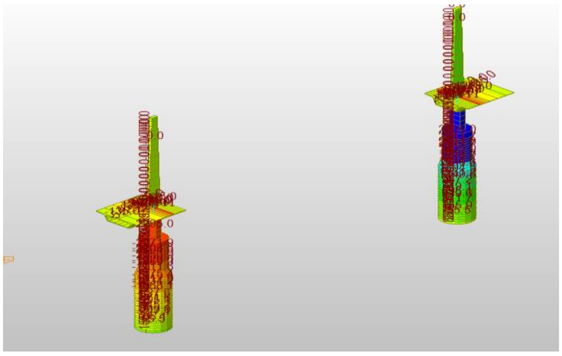


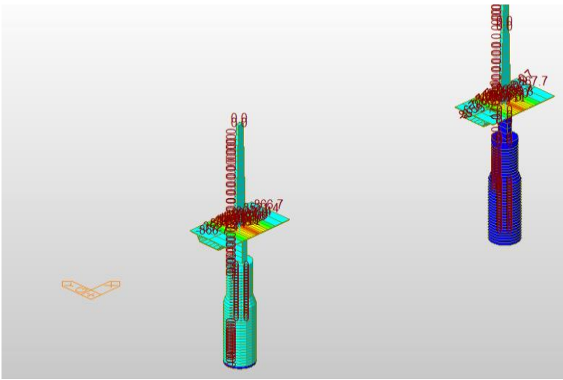
SIDL variable

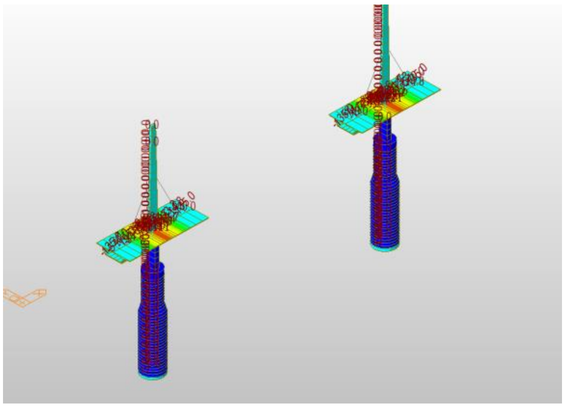


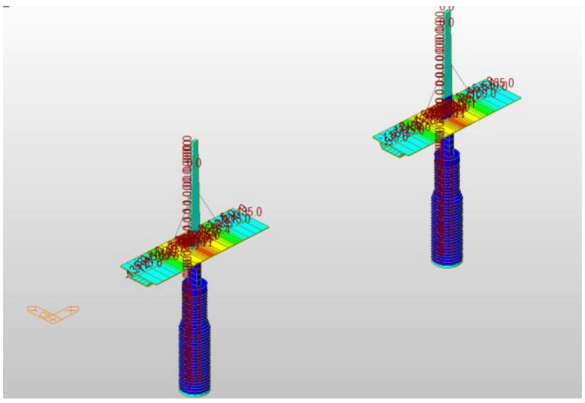
8.1 Construction stage analysis

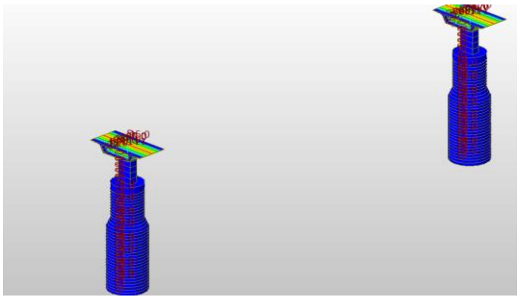
Total 22 stages are considered in construction stage analysis.

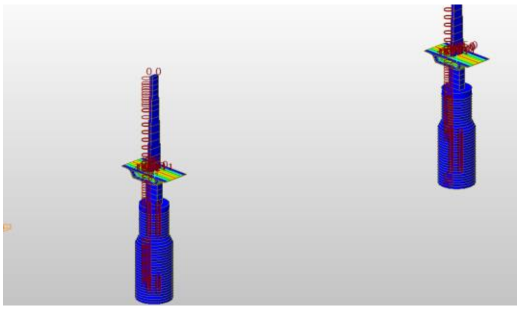


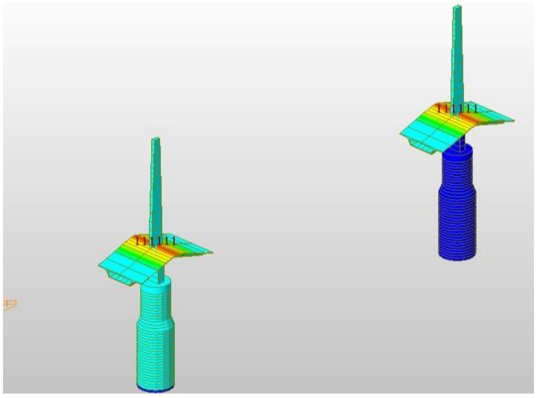


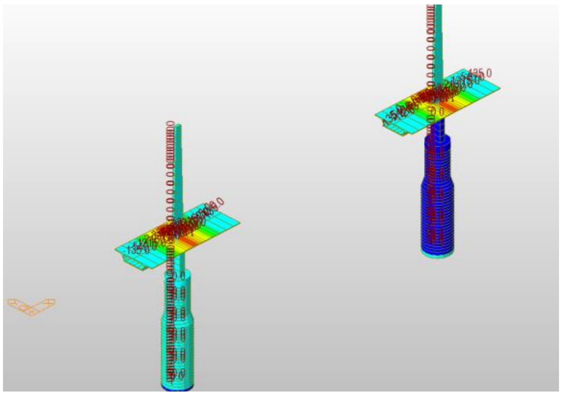


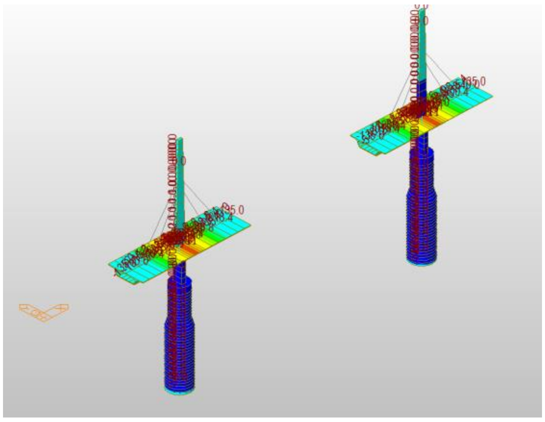


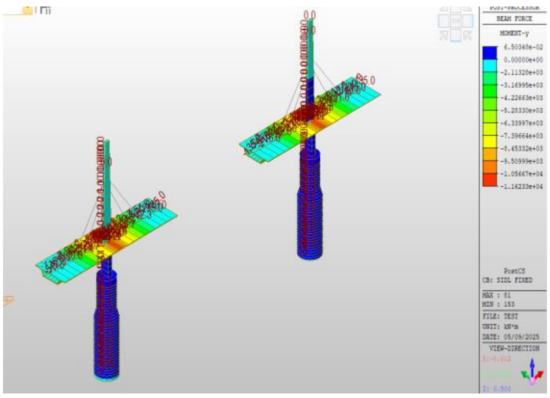








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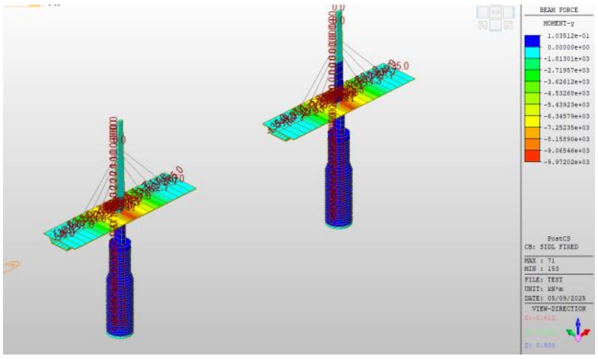
0.00000e+00
-2.11328e+03
-3.16995e+03
-4.22663e+03
-5.28330e+03
-6.33997e+03
-7.39664e+03
-9.50999e+03
-1.05667e+04
-1.16233e+04
PostCS
CB: SIDL FIXED
MAX : 81
MIN : 153
FILE: TEST
UNIT: kN*m
DATE: 05/09/2025
VIEW-DIRECTION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-6.34579e+03
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-9.06546e+03
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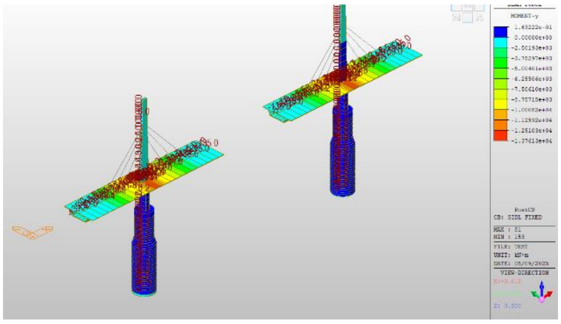
PostCS
CB: SIDL FIXED
MAX : 71
MIN : 153
FILE: TEST
UNIT: kN*m
DATE: 05/09/2025
VIEW-DIRECTION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-8.75715e+03
-1.00082e+04
-1.12592e+04
-1.25103e+04
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PostCS
CB: SIDL FIXED
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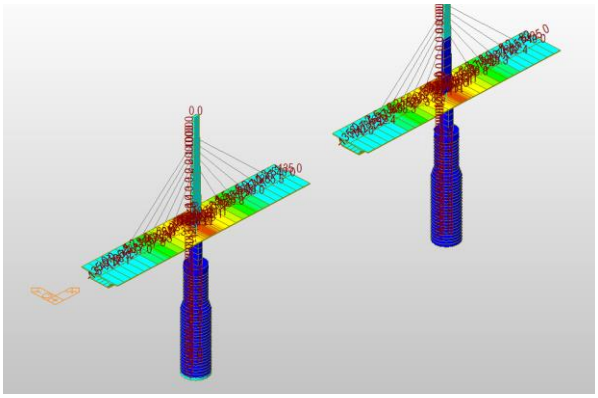
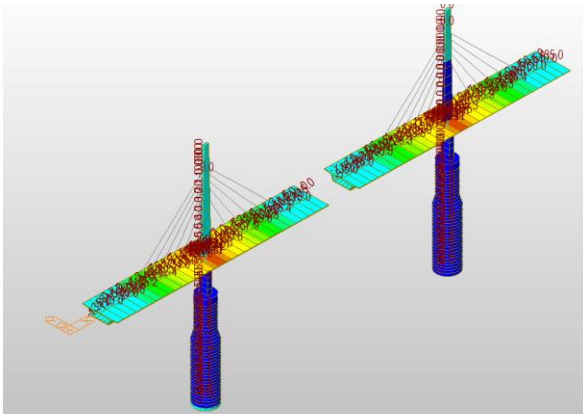
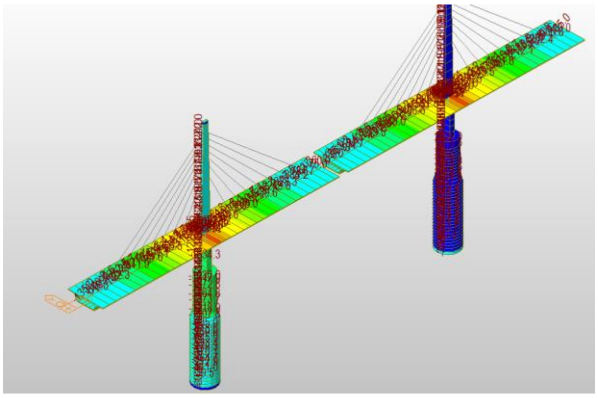
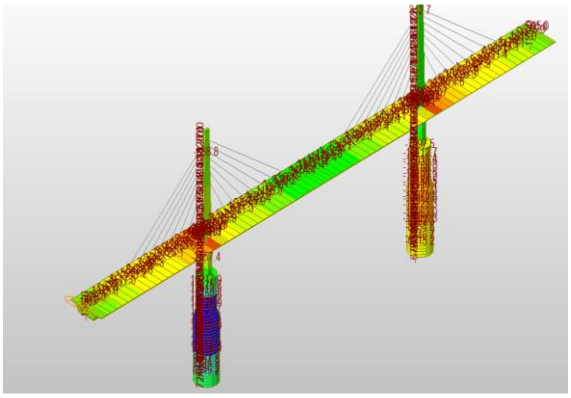
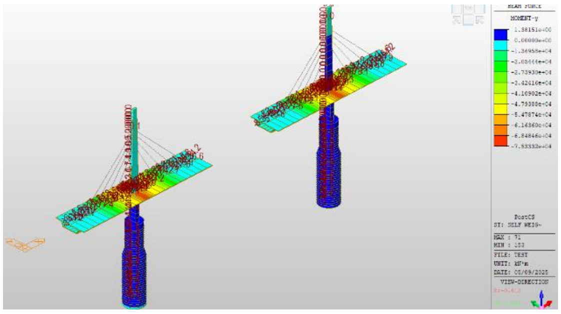
UNIT: kN*m
DATE: 05/09/2025
VIEW-DIRECTION

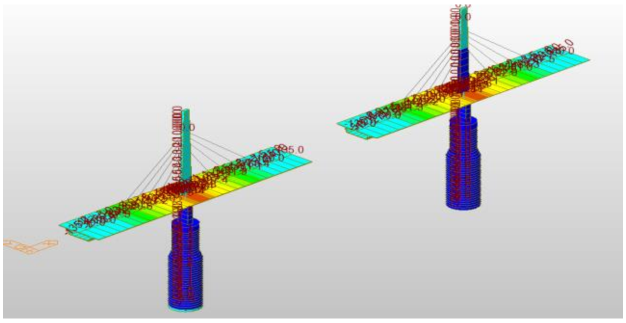


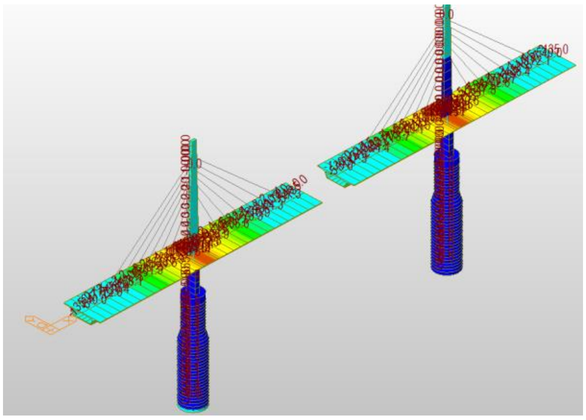


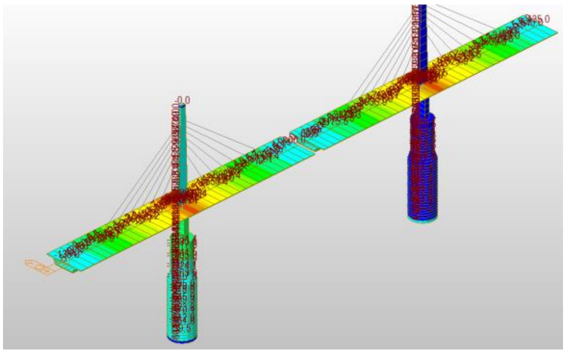


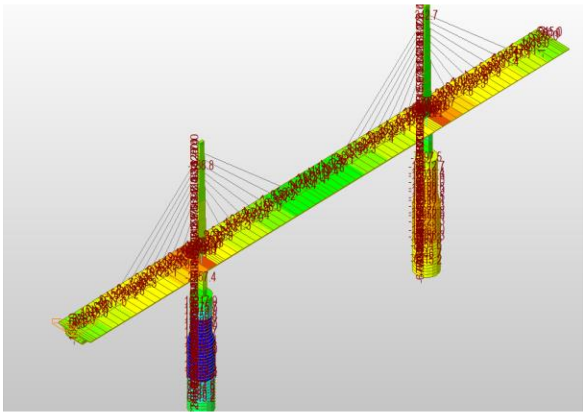



BEAM FORCE
MOMENT-y
1.38151e+00
0.00000e+00
-1.36958e+04
-2.05444e+04
-2.73930e+04
-3.42416e+04
-4.10902e+04
-4.79388e+04
-5.47874e+04
-6.16360e+04
-6.84846e+04
-7.53332e+04
PostCS
ST: SELF WEIG-
MAX : 71
MIN : 153
FILE: TEST
UNIT: kN*m
DATE: 05/09/2025
VIEW-DIRECTION



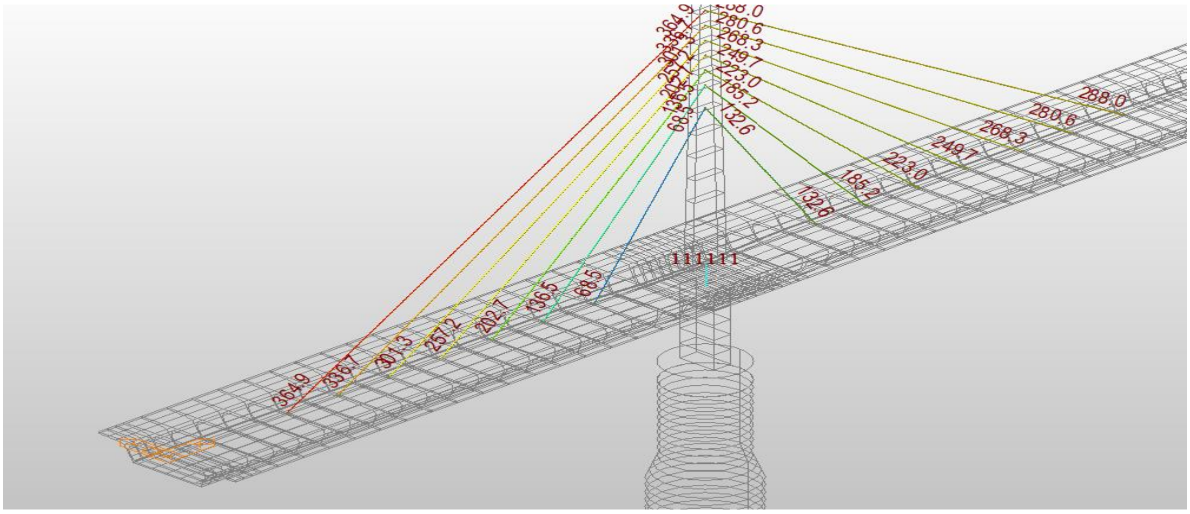


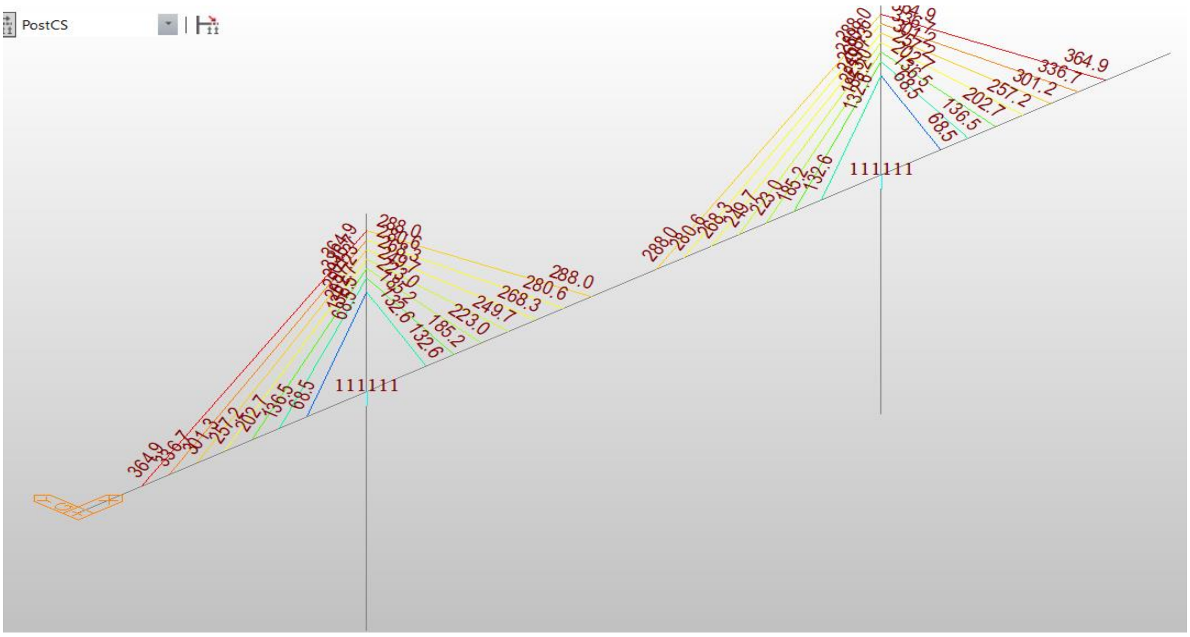




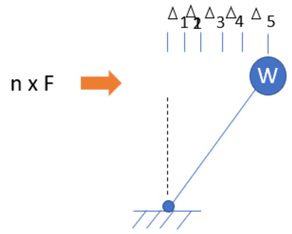
Results of the construction stage analysis

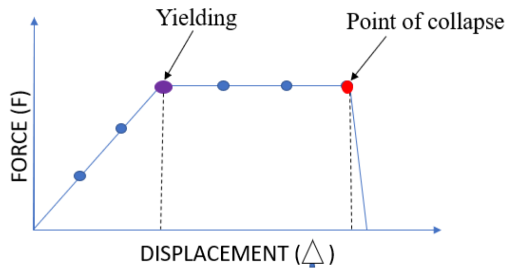
Cable forces after the construction stage analysis



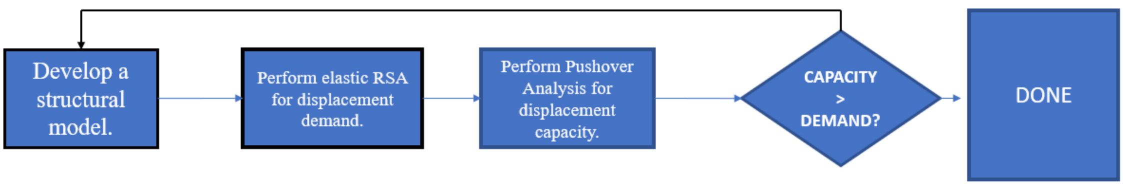


8.2 PUSH OVER ANALYSIS





OVERALL PROCEDURE



8.2.1 PLASTIC HINGE LENGTH CALCULATION

The height of the lower pylon = 10m

Uls permanent longitudinal moment at top = 360026 kn.m

Uls permanent longitudinal moment at = 35393 kn.m

Distance between points of contra flexure at bottom L1

$$L1 = \frac{36026\ x\ 10}{36026+3539} = 5.04\ m$$

Distance between points of contra flexure at top L2

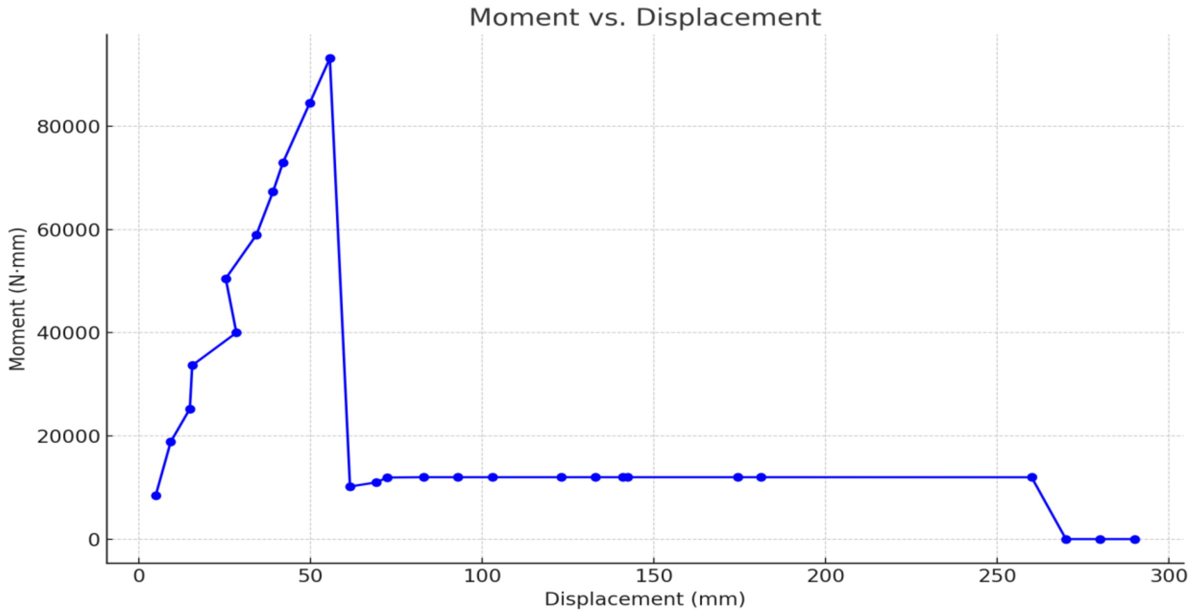
$$L1 = \frac{35398\ x\ 10}{36026+3539} = 4.96m$$

Length of plastic hinge at bottom Lp1 = (0.1 x 5040) + (0.015 x 500 x 36) = 774 mm

Length of plastic hinge at top Lp2 = (0.1 x 4960) + (0.015 x 500 x 36) = 766 mm

Lp = 766 = 0.766 m = 0.776/2 = 0.383 x 3.616 = 1.392

Lp = 774 = 0.774 = 0.774/2 = 0.387



9.0 CHECKS

9.1 LIMIT STATE OF SERVICEABILITY

Demand displacement from the response spectrum analysis = 176.40 mm

Maximum displacement from pushover analysis = 260mm

Demand displacement is less than maximum displacement hence it is safe.

- Deck displacement less or equal $\frac{L}{540}$

112000/540 = 207.407mm

Hence the deflection is well with in the limit.

10.0 DESIGN OF STRUCTURAL ELEMENTS FOR BRIDGE

10.1 ELASTOMERIC BERING DESIGN

Design of Reinforced Elastomeric Bearing as per IRC:83

4.1 Bearing Dimension

Length l_o =	**600** mm	Eff. length (l_e)=	**584** mm
Breadth b_o =	**400** mm	Eff. breadth (b_e)=	**384** mm
Side cover c =	**8** mm	Eff area (A_e)=	**224256** sqmm
Internal layer h_i =	**18** mm	Shape Factor(S_i)=	**6.44**
Outer layer h_e =	**6** mm	Shore Hardness =	**60**
Steel thk. t_s=	**6** mm	Shear Modulus (G)=	**0.90** N/sqmm
No. of Steel plate n =	**5** nos		
Total Elastomer. (h)=	**84** mm		
Total Height of Bearing (t) =	**114** mm		

t_{tot}=[2*t_0 + (n-1)*t_i + n*t_s]

4.2 Loads on bearing :

	V_{max}(KN)	V_{min}(KN)	H_{ld} (KN)	H_{bd}(KN)
Load Case 1 : Max. Reaction Case	**2998.26**	**1091.00**	**5.86**	**33.99**
Load Case 2 : Min. Reaction Case	**2998.26**	**1091.00**	**5.86**	**33.99**
Load Case 3 : Max. H_L Case	**2004.90**	**1377.60**	**150.95**	**49.35**
Load Case 4 : Max. H_T Case	**2869.46**	**1219.80**	**5.86**	**177.62**

4.3 Movement of Bearing

	D_{bd} (in mm)	D_{ld} (in mm)
Movement due to Temperature	28.74	0.9
Movement due to Shrinkage	9.50	0.7
Movement due to Creep of conc.	0.00	0.0
Displace ment caused by lateral force	62.83	20.5
Total Longitudinal Movement	**101.07**	**22.10**

4.4 Rotation of Bearing

	a_{bd} (radian)	a_{ld} (radian)	Additional rotation
Rotation due to Dead Load & SIDL+Prestress	0.001	0.000	
Rotation due to Live Load	0.001	0.000	
Rotation due to Installation Inaccuracy	0.000	0.000	
Total Rotation	**0.00271**	**0.00020**	0.00071
a_d =	**0.00302**		

4.4 Maximum Design Strain

At any point in the bearing the sum of the strains ($\varepsilon_{t,d}$) due to the design load effects (E_d) is given by the expression:

$\varepsilon_{t,d}$ = KL ($\varepsilon_{c,d}$ + $\varepsilon_{q,d}$ + $\varepsilon_{a,d}$) < 7

$\varepsilon_{c,d}$ - Design strain due to compressive design loads

$\varepsilon_{q,d}$ - Design shear strain due to design traslatorey movements

$\varepsilon_{a,d}$ - Design and strain due to compressive design loads

K_L is type of loading factor. The vlue of KL is normally considered as 1.

Design stain due to compressive load

$\varepsilon_{c,d}$ = 1.5 F_{zd} / G A_r S

where, Ar = A1 (1 - v_{xd}/a' - $v_{y,d}$/b') = 156748 mm2

	V_{max}(KN)	V_{min}(KN)	$\varepsilon_{c,d}$	$\varepsilon_{c,d}$
Load Case 1 : Max. Reaction Case	**2998.26**	**1091.00**	**4.95**	**1.80**
Load Case 2 : Min. Reaction Case	**2998.26**	**1091.00**	**4.95**	**1.80**
Load Case 3 : Max. H_L Case	**2004.90**	**1377.60**	**3.31**	**2.28**
Load Case 4 : Max. H_T Case	**2869.46**	**1219.80**	**4.74**	**2.02**

Design shear strain

$\varepsilon_{q,d}$ = $v_{xy,d}$/T_q

$v_{xy,d}$ is maximum resultant horizontal relative displacement of parts of the bearing obtained by vectorial addition of

Hence ok

4.5 Reinforcing plate due to angular rotation

Thickness of steel plate $t_s = K_p F_{z,d} (t_1 + t_2) K_h \gamma_m / (A_r f_y)$

t1	=	18.00
t2	=	18.00
f_y	=	250.00
K_h	=	1.00
γ_m	=	1.00
K_p	=	1.30
ts	=	2.34

Hence ts should be minimum 3.00 mm

i) $Ev_{z,d} = E (F_{zd} t_1 / A1) (1/5\ G\ S_1^2 + 1/E_b)$

Bulk Modulus Eb = 2000.00 MPa

$Ev_{z,d}$ min= 2.05 0.00 = 2.05

ii) $(a'\ \alpha_{a,d} + b'\ \alpha_{b,d})/K_{r,d}$= 0.39

Kr,d is rotational factor which is considered as 3

Evz,d max= 5.647 0.000 = 5.647

i) - ii) = 1.67 > 0

Section is ok

Buckling stability (ULS)

For laminted bearings, the pressure, Fz,d/Ar shall satisfy the expression

$F_{z,d}/Ar$ = **12.49** --iii)

2 a' G S1/ 3 Te = **17.65** --iV)

$F_{z,d}/Ar$ < 2 a' G S1/ 3 Te

Section is ok

Non sliding condition (ULS)

For non anchored bearing

$\mu_e = 0.1 + 1.5\ K_f/\sigma_m$

K_f= 0.60 for concrete

$F_{z,d\ min}$ (KN)	H_{ld} (KN)	H_{bd}(KN)	$F_{xy,d}$	σ_m	μ_e	μ_e Fz,dmin
1091.00	**5.86**	**33.99**	**34.49**	4.55	0.30	325.10
1091.00	**5.86**	**33.99**	**34.49**	4.55	0.30	325.10
1377.60	**150.95**	**49.35**	**158.81**	5.74	0.26	353.76
1219.80	**5.86**	**177.62**	**177.72**	5.08	0.28	337.98

Section is OK

Under the permanent load

$\sigma_{cd\ min} = F_{z,d\ min}/Ar$

$F_{z,d\ min}/Ar$ = 1479.01 kN

$\sigma_{cd\ min}$ = 6.16 > 3

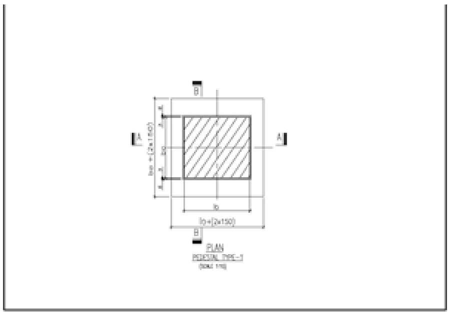
Section is ok

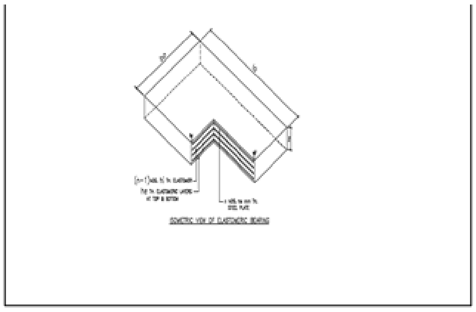
4.7 Forces, moments and deformations exerted on the structure pressure on the contact surface

Elastomeric bearing exert a non-uniform pressure on the contact surface with the structure. It is sufficinet to ensure that mean pressure does not exceed the streght of the supporting materail.

The force Rxy exerted on the structure by the bearing resisting traslatory movement,

$R_{xy} = A\ G\ V_{xy}/T_e$ = 266022.27 = 266.02 kN





4.5 Check for Bearing Geometry as per Cl. 916.3.3

a)	l_o/b_o =	1.500		OK
d)	h =	84	mm	please modify h
e)	h_i =	18	mm	NOT OK, please modify hi
d)	Shape Factor =	6.44		OK

4.6 Check for Translation as per Cl. 916.3.4

		Load Case 1	Load Case 2	Load Case 3	Load Case 4
a)	D_{bd}/h	1.203155654	1.203155654	1.203155654	1.203155654
b)	t_{bd}	0.0290	0.0290	0.7479	0.0290
c)	g_{bd}	1.2322	1.2322	1.9511	1.2322
d)	D_{ld}/h	0.26	0.26	0.26	0.26
e)	t_{ld}	0.1684	0.1684	0.2445	0.8801
f)	g_{ld}	0.4315	0.4315	0.5076	1.1431
g)	g_d	1.3056	1.3056	2.0160	1.6808
	Status	Not OK, Please modify plan area			

4.7 Check for Rotation as per Cl. 916.3.5

a)	n =	4	
b)	α_{bi},max =	0.00565951	
c)	β =	1.336982596	
d)	α_d =	0.00302	
e)	$\beta.n.\alpha_{bi}$,max =	0.030266667	OK

4.8 Check for Friction as per Cl. 916.3.6

a)	$\sigma_{m(max)}$ =	13.370		Greater than 10, Hence Not OK
b)	$\sigma_{m(min)}$ =	4.865	Mpa	OK

	V_{min}(KN)	s_m (Mpa)	$0.2+0.1s_m$	g_d
Load Case 1 : Max. Reaction Case	1090.9964	4.8650	0.68650	1.3056
Load Case 2 : Min. Reaction Case	1090.9964	4.8650	0.68650	1.3056
Load Case 3 : Max. H_L Case	1377.6039	6.1430	0.81430	2.0160
Load Case 4 : Max. H_T Case	1219.8009	5.4393	0.74393	1.6808
STATUS			NotOK	

4.9 Check for Total Shear Stress as per Cl. 916.3.7

a)	τ_c =	3.1164	Mpa	
b)	τ_y =	2.0160	Mpa	
c)	τ_α =	1.2879	Mpa	
d)	Total $\tau_c+\tau_a+\tau_\alpha$ =	6.4203	Mpa	Not Ok

4.10 Check for Stress in concrete below Bearings

Stress below Bearing, σ_{bb} =	12.49	Mpa
Length of Pedestal (Across Bridge Axis) =	900	
Breadth of Pedestal (Along Bridge Axis) =	700	
A1 =	630000	
A2 =	224256	
A1/A2 =	2	

Allowable Stress, σ_{cc} = 14.14 Mpa OK

10.2.1 CABLE DESIGN CALCULATION

Force in cable 260.4 KN

Choose cable profile DB-P37 with no of strands 37

Area = 140 mm2

Stress = 1860 n/mm2

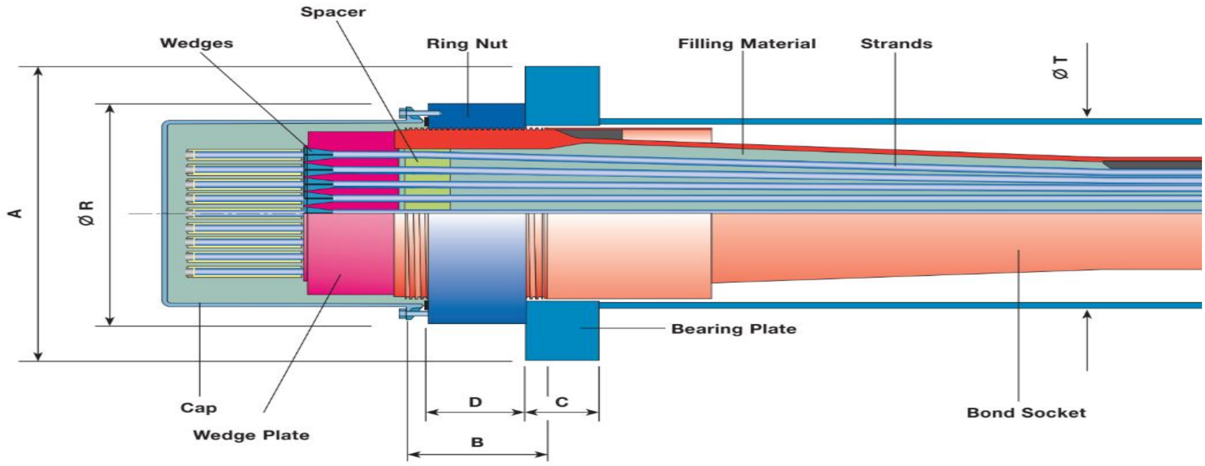
Stress 45 percentage = 1860 x 0.45 = 837 mpa

35 dead load 10 live load 1860 x 0.1 = 186 mpa

1860 x 0.35 = 651 mpa

= 651 x 5100 mm2 = 3300 KN

DYWIDAG Multistrand Stay Cable Systems



DYNA Bond® Anchorage
(forces calculated with strands 0,62" St 1620/1860)

Cable type		DB-P12	DB-P19	DB-P27	DB-P37	DB-P48	DB-P61	DB-P75	DB-P91	DB-P108
No. of strands		12	19	27	37	48	61	75	91	108
Forces [kN]										
ultimate load (GUTS)		3.348	5.301	7.533	10.323	13.392	17.019	20.925	25.389	30.132
working load (0,45 x GUTS)		1.507	2.385	3.390	4.645	6.026	7.659	9.416	11.425	13.559
Dimensions [mm]										
bearing plate	A	300	370	430	500	580	640	715	780	855
bearing plate	C	50	60	70	80	90	100	110	120	130
thread*	B	160	170	180	190	205	220	240	260	280
ring nut	D	90	100	110	120	135	150	170	190	210
ring nut	Ø R	244	287	326	378	434	480	536	584	636
recess pipe	Ø T	219	245	299	324	394	419	470	508	559
HDPE sheathing	Ø P	110	125	160	180	200	225	250	280	315

* standard length, larger lengths can be provided upon special request

Subject to modification

fig. properties of cable profile

10.3.1 DESIGN OF WELL FOUNDATION

Span Arrangement=	112	meter
Deck Level	98.5	meter
HFL =	80.67	meter
LWL =	78.37	meter
Well Cap Bottom Level	89.8	meter
Back fill Soil Properties.		
Ka =	0.297	
Bulk Density of soil =	2	t/cum.
Submerged Density of Soil	1	t/cum.
Angle of internal friction=	30	degree
Angle of wall friction	21	degree
Width of Well Cap	8	meter
[redacted]	13	
	14	
	97.000	
	96.000	
Thickness of dirt wall	0.3	meter
Thickness of Well Cap	1.8	meter
Thickness of abutment shaft	1.5	meter
Thickness of side wall	0.4	meter
Seismic Zone	V	
Seismic Coefficient =	0.27	
Overhang of Superstructure from C/L of bearing =	0.6	meter
Expansion Gap	0.05	meter
[redacted]	4	
Thickness of bearing beam (Rect.)	0.6	meter
Thickness of bearing beam (Trapz.)	0.4	meter
Abutment cap width	1.9	meter
Max. Scour Level	89.8	meter
Top Level of earthfilling in well	89.5	meter
Average Ground Level	89.8	meter
Thickness of bearings inclluding pedestals	0.5	meter
Outer Dia of Well	8	meter
Internal dia of well	7	meter
Thickness of Top Plug	0.3	meter
Dia of Well Curb	7	meter
Foundation Level	70	meter
Thickness of steining	1	meter

SUMMARY OF LOADS OF WELL FOUNDATION

Wt.of Intermediate plug	21.2058	ton
Wt.of Well Cap	400.5351	ton
Dry Wt. Of Well Steining	54.978	ton
Sub. Wt. Of Well Steining	32.9868	ton
Dry. Wt. Of Pocket Filling	56.5488	ton
Dry.. Wt. Of Well Curb and Bottom Plug	250.1499	ton
Kp	6.105	

SUMMARY OF LOAD AND MOMENTS

NON SEISMIC CASE(LWL CONDITION)

Live load max.

S.No.	Description	Vertical Load	HL ton	HT ton	ML tm	MT tm
1	Dead load of superstructure	13332			0	
2	Live load on superstructure	153.17			0	399
3	Footpath live load Reaction	48.029			0	
4	Snow Load	144.33				
5	Dead load of substructure	1306.6			-1514	
6	Due to earth pressure		372.86		1418.5	
7	Horizontal force at bearing Level		691.67		5325.9	
	Total	14984	1064.5		5230.1	399

MR = 5245

II Live load Min.

S.No.	Description	Vertical Load	HL ton	HT ton	ML tm	MT tm
1	Dead load of superstructure	13332			0	
2	Live load on superstructure	46.828			0	54.09
3	Dead load of substructure	1306.6			-1514	
4	Due to earth pressure		372.86		1418.5	
5	Horizontal force at bearing Level		683.96		5266.5	
	Total	14686	1056.8		5170.7	54.09

MR = 5171

iii No Superstructure

S.No.	Description	Vertical Load	HL ton	HT ton	ML tm	MT tm
1	Dead load of superstructure	0			0	
2	Live load on superstructure	0			0	0
3	Dead load of substructure	1306.6			-1514	
4	Due to earth pressure		292.24		1067.8	
5	Horizontal force at bearing Level		0		0	
	Total	1306.6	292.24		-446.4	0

MR = 446.4

NONSEISMIC CASE(HFL CONDITION)

Live load max.

S.No.	Description	Vertical Load	HL ton	HT ton	ML tm	MT tm
1	Dead load of superstructure	13332			0	
2	Live load on superstructure	153.17			0	399
3	Footpath Live load Reaction	48.029			0	
4	Dead load of substructure	1306.6			-1514	
5	Due to earth pressure		372.86		1418.5	
6	Horizontal force at bearing Level		691.67		5325.9	
	Total	14840	1064.5		5230.1	399

MR = 5245

DESIGN OF PIER WELL

Loads and moments upto Abutment shaft Bottom level

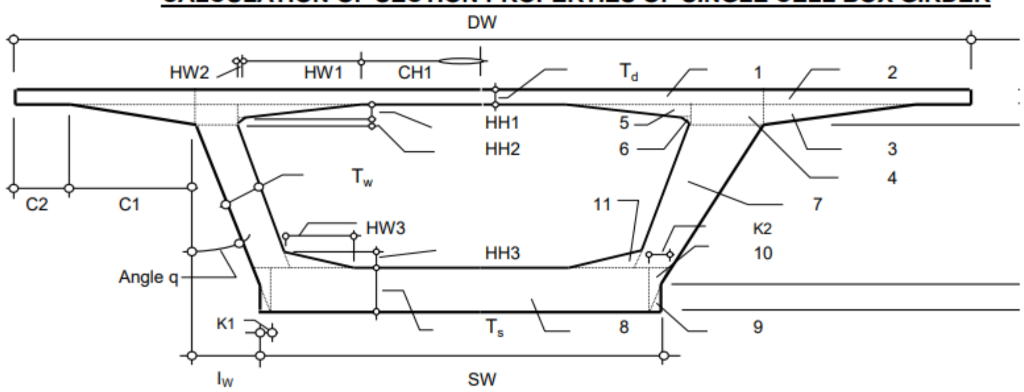
Case	Vertical Force KN	Horizontal Force (KN)		Moment (KNm)	
		HL	HT	About Transverse Axis ML	About Long. Axis MT
Normal Case , LWL, Max. LL	145838.6	10645	0	52301.4	3990.1
Normal Case , LWL, Min. LL	144395.3	10645	0	52301.4	3990.1
Seismic Case, LWL, Max. LL	143170	73535	0	522391	798.03
Seismic Case , LWL, Min. LL	142785.7	73535	0	522391	798.03

10.4.1 DESIGN OF PSC BOX GIRDER

Basic Design Data

Overall Span (C/C spacing of exp. joint)	=	48.750	m
Effective Span (C/C spacing of Bearing)	=	47.750	m
Distance between C/L of Brg. and C/L of Exp. Joint	=	0.500	m
Girder end to bearing centre line	=	0.500	m
Expansion gap	=	0.040	m
Width of deck	=	16.000	m
Depth of Box Girder	=	4.000	m
Grade of Concrete of Girder	=	60	Mpa
Age of concrete for at transfer		14	days
Maturity of concrete for at transfer		87	%
Strength concrete at the time of transfer		52.2	Mpa
Age of girder at the time of casting of SIDL		56	days
Maturity of girder at the time of casting of SIDL		100	%
Extra time dependent loss to be considered		20.0	%
Wearing coat thickness	=	0.075	m

CALCULATION OF SECTION PROPERTIES OF SINGLE CELL BOX GIRDER



NOTE : 1. ALL DIMENSIONS ARE TO BE GIVEN IN METERS
2. CELLS SHADED ARE NOT BE GIVEN ANY INPUT

SECTION		Support	D away from support	L/8	L/4	3L/8	L/2
Web Inclination, θ(deg)		**26.5651**	**17.6501**	**17.6501**	**17.6501**	**17.6501**	**17.6501**
Total Depth	D	4.000	3.000	3.000	3.000	3.000	3.000
Top Flange	DW	16.000	9.750	9.750	9.750	9.750	9.750
	T_d	0.200	0.500	0.500	0.500	0.500	0.500
cantilever	C1	1.000	1.500	1.500	1.500	1.500	1.500
	C2	1.000	0.000	0.000	0.000	0.000	0.000
	T_{tip}	0.200	0.200	0.200	0.200	0.200	0.200
	T_f	0.500	0.300	0.300	0.300	0.300	0.300
Web	l_W	1.000	0.700	0.700	0.700	0.700	0.700
	D1	2.000	2.200	2.200	2.200	2.200	2.200
	T_w	0.800	0.800	0.800	0.800	0.800	0.800
soffit Slab	SW	8.000	6.750	6.750	6.750	6.750	6.750
	T_s	0.200	0.500	0.500	0.500	0.500	0.500
	D2	0.200	0.500	0.500	0.500	0.500	0.500
	K1	**0.1000**	**0.1591**	**0.1591**	**0.1591**	**0.1591**	**0.1591**
	K2	**0.1000**	**0.1591**	**0.1591**	**0.1591**	**0.1591**	**0.1591**

10.4.2.1 DESIGN OF TENDON IN PSC BOX GIRDER AND TENDONS.

Calculation of Prestressing Force & Its Effects at Various Sections

A. CONSTRUCTION PROGRAM & PRESTRESSING STAGES

	ACTIVITY	*DAY AFTER CASTING*		fcj (MPa)
i)	Completion of casting of Box Girder	0	day	
ii)	1st Stage prestress	14	day	52.20
iii)	Completion of wearing coat, crash barrier	56	day	60.00

B. TENDON PARTICULARS

1)	Nominal Diameter	D	15.2	mm
2)	Nominal Area	A	140	sq.mm
3)	Nominal Mass	Pu	1.1	Kg/m
4)	Yield Strength	Fy	1670	MPa
5)	Tensile Strength	Fu	1860	MPa
6)	Minimum Breaking Load	Pn	260.7	KN
7)	Young's Modulus of Elasticity	Eps	15.2	Gpa
8)	Jacking Force at Transfer (% of Breaking Loa	Pj	195	%
9)	Slip at Jacking end	s	76.5	mm
10)	Coefficient of Friction	µ	6	per radian
11)	Wobble Friction Coefficient	k	0.17	per metre
12)	Relaxation of prestressing steel at 70% uts	Re1	0.0	Mpa
13)	Relaxation of prestressing steel at 50% uts	Re2	35	MPa
14)	Age of concrete for 1st Stage prestressing	t_{d1}	14	days
15)	Dia of Prestressing Duct	q_d	0	mm
16)	Concrete Grade	Fcu	60	MPa
17)	Modulus of Elasticity of Concrete (28 days)	Ec	38729.8	Mpa

C. FORCES AFTER FRICTION SLIP (For Friction & Slip calculation refer next few sheets)

Section					Support Section	Daway from support	1/8th span section	1/4th span section	3/8th span section	Mid span
Distance from Left support					0.00	2.50	5.97	5.97	5.97	5.97
COMPONENT				UNIT						
Cable No. 1	No. of Cables	2	Ecc. From sofit	m	1.672	1.505	1.274	0.876	0.478	0.130
	P_x (per Cable)			t	326.5	335.8	343.1	347.4	350.7	333.6
Cable No. 2	No. of Cables	1.052	Ecc. From sofit	m	0.000	0.000	0.000	0.000	0.167	0.130
	P_x (per Cable)			t	303.4	304.1	318.6	338.6	339.5	339.5
Cable No. 3	No. of Cables	2	Ecc. From sofit	m	0.522	0.312	0.130	0.130	0.130	0.130
	P_x (per Cable)			t	317.0	325.0	336.5	343.1	342.0	340.9
Cable No. 4	No. of Cables	1.052	Ecc. From sofit	m	0.248	0.130	0.130	0.130	0.130	0.130
	P_x (per Cable)			t	0.0	0.0	0.0	307.3	338.6	339.5
Cable No. 5	No. of Cables	2	Ecc. From sofit	m	0.522	0.312	0.130	0.130	0.130	0.130
	P_x (per Cable)			t	328.7	330.6	338.4	341.4	339.7	335.6
Cable No. 6	No. of Cables	2	Ecc. From sofit	m	0.248	0.130	0.130	0.130	0.130	0.130
	P_x (per Cable)			t	317.0	325.0	336.5	343.1	342.0	340.9
Cable No. 7	No. of Cables	1.052	Ecc. From sofit	m	0.248	0.130	0.130	0.130	0.130	0.130
	P_x (per Cable)			t	338.2	345.3	347.0	350.0	353.0	356.0
TOTAL NO. OF CABLE					10.1	10.1	10.1	11.2	11.2	11.2
TOTAL Px (STAGE-1)				t	3253	3316	3409	3798	3834	3791
Cg from Bottom				m	0.638	0.474	0.356	0.251	0.196	0.130
Ecc. From cg				m	2.074	1.171	1.290	1.394	1.449	1.515
TOTAL PRIMARY BM				t.m	6748	3882	4397	5293	5556	5744

10.4.3.1 DESIGN OF PSC SECTION

Check for Ultimate Shear

Grade of Concrete	60	Mpa	
Perm. direct shear stress ; τ_v	5.80	Mpa	(As per relevant standard)
Perm. direct shear stress ; τ_{tv}	0.42	Mpa	(As per relevant standard)
Perm. shear stress in combined shear & torsion; τ_{tu}	5.81	Mpa	(As per relevant standard)

COMPONENT / Section	UNIT	"d" away from Support	L/8	L/4	3L/8	L/2
Ultimate Shear Capacity of Section uncracked in Flexure (As per relevant standard)						
Overall Width, bo	m	1.679	1.679	1.679	1.679	1.679
Overall Depth, d	m	4.000	4.000	4.000	4.000	4.000
Area of Section	m^2	11.409	11.433	11.519	11.519	11.519
Dia of duct, ϕ	m	0.000	0.000	0.000	0.000	0.000
Effective Width, b = bo-2/3ϕ	m	1.679	1.679	1.679	1.679	1.679
Maximum Principal Tensile Stress, $f_t = 0.24(fck)^{0.5}$	Mpa	1.86	1.86	1.86	1.86	1.86
Horizontal Component of prestress after all losses	KN	3293.1	3385.8	3771.7	3807.2	3764.9
Cg of cable from sofit, Yord	m	0.474	0.356	0.251	0.196	0.130
Comp. Stress due to prestress, f_{cp}	Mpa	0.289	0.296	0.327	0.331	0.327
Effect of Vertical Prestress, V_{pr}	KN	0.0	0.0	0.0	0.0	0.0
Shear Capacity, $V_{co} = 0.67*b.d.(f_t^2+.8f_{cp}*f_t)^{0.5}$	KN	8869.7	8882.4	8935.2	8940.4	8934.3
$V_c = V_{co} + V_{pr}$	KN	8869.7	8882.4	8935.2	8940.4	8934.3
Ultimate Shear Capacity of Section cracked in Flexure (As per relevant standard)						
Effective Width, b (m)	m	1.679	1.679	1.679	1.679	1.679
D1 = (D - Yord)	m	3.526	3.644	3.749	3.804	3.870
D2 = 0.8*D	m	3.200	3.200	3.200	3.200	3.200
Depth , d_b	m	3.526	3.644	3.749	3.804	3.870
Stress due to prestress. f_{pt}	KN/m^2	751.6	820.1	954.4	988.6	1007.2
Distance of extreme fibre from centroid, y_b	m	1.645	1.646	1.645	1.645	1.645
Second Moment of Area, I	m^4	13.700	13.718	13.795	13.795	13.795
Cracking Moment, Mt = $(0.37*(f_{ck})^{0.5}+0.8*f_{pt}).I/y$	KNm	28874.9	29351.3	30431.8	30661.5	30786.3
Ult. Applied Shear Force, V_{ult}	KN	7515	6291	4556	2179	256
B. Moment corresponding to Ult. Shear Force, M_{ult}	Knm	21283	44216	74538	92488	97614
Shear Capacity, Vcr = $0.037*b*d_b*(f_{ck})^{0.5}+M_t/M*V$	KN	11892.4	5929.3	3664.0	2552.8	1943.0
Design Shear Capacity, Vc	KN	8869.7	5929.3	3664.0	2552.8	1943.0

Check for Limiting Shear for Outer Girder (As per relevant standard)						
Ultimate Shear, V_u	KN	7515.0	6290.8	4556.2	2178.5	256.0
$V_u - V_{pr}$	KN	7515.0	6290.8	4556.2	2178.5	256.0
Depth , d_b	m	3.526	3.644	3.749	3.804	3.870
$\tau=V_u/(b.d_b)/1000$	MPa	1.269	1.028	0.724	0.341	0.039
Status		OK	OK	OK	OK	OK
Provision of Shear Reinforcement (As per relevant standard)						
Is V less than $V_c/2$?		No	No	No	No	Yes
$V-V_c$ (in KN)	KN	-1354.7	361.6	892.2	-374.3	-1687.0
Minimum reift., $A_{sv} / S_v = 0.4*b/(.87*f_{yv})$	mm^2/m	1860.2	1860.2	1860.2	1860.2	
Shear Reinft. Due to ultimate loads, $A_{sv}/S_v = (V-V_c)/(0.87*f_y*d_t)$	mm^2/m	0	257	634	0	0
Design Shear Reinforcement	mm^2/m	1860.2	1860.2	1860.2	1860.2	0.0

REINFORCEMENT REQUIRED FOR TORSION						
Ult. Applied Torsional Moment, T_{ult}	Knm	4354.77	3585.09	2314.36	1025.25	339.24
Area enclosed on C/L of Box, Ao	m^2	10.70	10.70	10.70	10.70	10.70
Perimeter of Ao	m	13.90	13.90	13.90	13.90	13.90
Deck Thickness	m	0.225	0.225	0.225	0.225	0.225
Sofit thickness	m	0.260	0.260	0.260	0.260	0.260
Total Shear Stress due to torsion : Web	MPa	0.24	0.20	0.13	0.06	0.02
Total Shear Stress due to torsion : deck	MPa	0.90	0.74	0.48	0.21	0.07
Total Shear Stress due to torsion: sofit	MPa	0.78	0.64	0.42	0.18	0.06
Status		R/F Reqd.	R/F Reqd.	R/F Reqd.	OK	OK
Asv/Sv for torsion/web	mm^2/m	563.9	464.2	299.7	0.0	0.0
Tot Asl for torsion						
Asl for deck	mm^2/m	563.9	464.2	299.7	0.0	0.0
Asl for web	mm^2/m	563.9	464.2	299.7	0.0	0.0
Asl for sofit	mm^2/m	563.9	464.2	299.7	0.0	0.0
EFFECT OF COMBINED TORSION & SHEAR						
Total Shear Stress : Web	MPa	1.51	1.23	0.85	0.40	0.06
Status		OK	OK	OK	OK	OK
Total Asv/Sv/web due to torsion & shear	mm^2/m	564	592.6	616.5	0.0	0.0
Minimum Asv/Sv/web	mm^2/m	930	930	930	930	#VALUE!
Asv/Sv/web required	mm^2/m	930	930	930	930	#VALUE!

Check for Ultimate Moment of Midspan Section (Which is more critical)

I) Failure by Yield of Steel	
Area of high tensile steel, As, (mm^2)	29674.96
Distance of cg of tendons from compression fibre, db, (mm)	3870.0
Ultimate tensile strength of steel, fp, MPa	1860
Ultimate Moment capacity of steel, Ms_{ult}, kNm = 0.9*db*As*fp	192245.7
II) Failure by Crushing of Concrete	
Width of web, b, (mm)	1679
Width of flange, Bf, (mm)	16000
Thickness of flange, t, (mm)	225
Ultimate Moment capacity of concrete, Mc_{ult}, (kNm) = 0.176*b*db^2*fck + 2/3*0.8*(Bf-b)*(db-t/2)*t*fck	652989.5
III) Ultimate Moment Capacity of Section	192245.7
IV) Applied Ultimate Moment	97614
Status	OK

10.5.1 PYLON SECTION DESIGN

Component 1

Material type: Concrete

Material grade: M85

Profile: STD R 2500 3000

Reinforcement

Cover

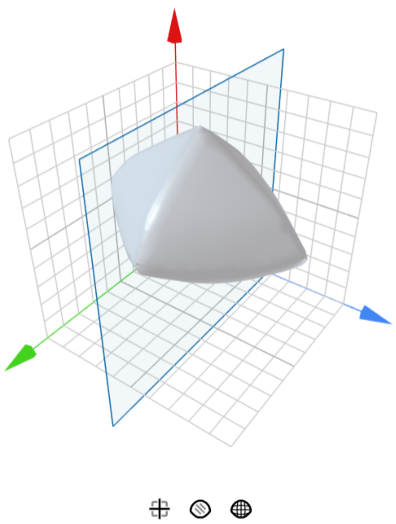
- (●) Uniform — All faces (mm): 80
- ( ) Variable

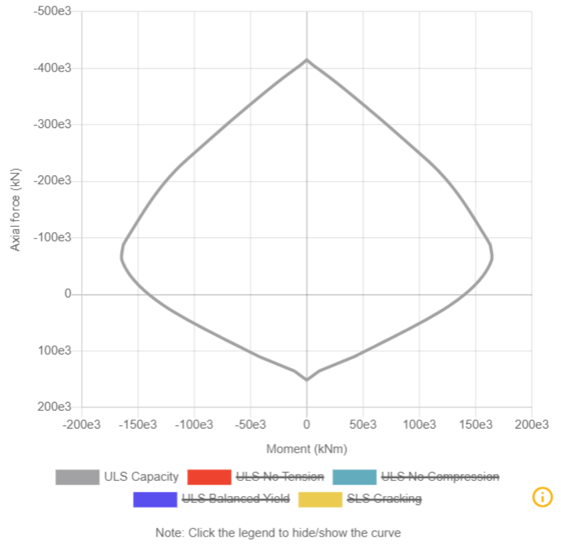
ADD REINFORCEMENT GROUP | ADD INDIVIDUAL BARS...

Group	Type	Description	Position(s) (mm, °)	Pre-Load Type
1	Line	24"FE500"50	(-1417,1167) (-14	None
2	Line	24"FE500"50	(1417,1167) (141	None
3	Line	24"FE500"50	(-1317,1167) (131	None
4	Line	24"FE500"50	(-1317,-1167) (13	None
5	Line	16"FE550D"50	(-1217,967) (-121	None
6	Line	16"FE550D"50	(1217,967) (1217,	None
7	Line	16"FE550D"50	(-1117,967) (1117	None
8	Line	16"FE550D"50	(-1117,-967) (111	None

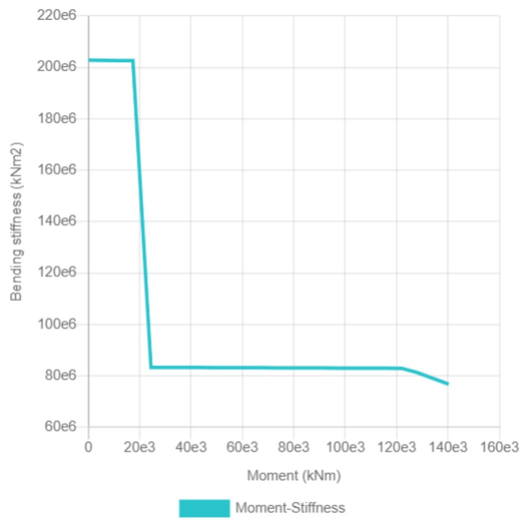
TABLE CASE DETAILS INTERACTION DIAGRAM MORE CHARTS

Case	Load			ULS Status	Utilisations		M/Mu	Deformation			Stiffness		
	F_x (kN)	M_{yy} (kNm)	M_{zz} (kNm)		Load	Deformation		ε_x (‰)	κ_{yy} (‰/m)	κ_{zz} (‰/m)	EA (kN)	EI_{yy} (kNm²)	EI_{zz} (kNm²)
1	29714	17314	10692	✓	32.42%	5.7%	0.1791	0.4774	0.332	0.14	350.4e6	200.38e6	289.26e6
2	39168	673	21417	✓	37.36%	0.97%	0.1619	0.6234	0.013	0.289	351.93e6	186.73e6	288.37e6
3	35615	14746	10692	✓	34.63%	1.03%	0.1645	0.5668	0.288	0.144	351.32e6	200e6	288.54e6
4	29714	17314	10692	✓	32.42%	5.7%	0.1791	0.4774	0.332	0.14	350.4e6	200.38e6	289.26e6
5	36015	23208	10664	✓	40.31%	7.19%	0.2335	0.5785	0.446	0.138	351.21e6	200.13e6	288.48e6
6	36606	40040	-10618	✓	51.69%	16.96%	0.3789	0.6321	0.716	-0.119	349.92e6	200.43e6	289.18e6
7	31312	48917	21472	✓	56.47%	28.82%	0.4738	0.6104	0.816	0.226	346.77e6	200.81e6	290.24e6
8	36045	-23208	-10664	✓	40.33%	6.88%	0.2335	0.582	-0.442	-0.139	351.65e6	200e6	288.55e6
9	37049	10552	21423	✓	37.99%	3.75%	0.1992	0.5904	0.205	0.288	351.81e6	199.47e6	288.52e6
10	39108	673	-21417	✓	37.32%	0.97%	0.1618	0.6224	0.013	-0.289	352.04e6	190.75e6	288.3e6

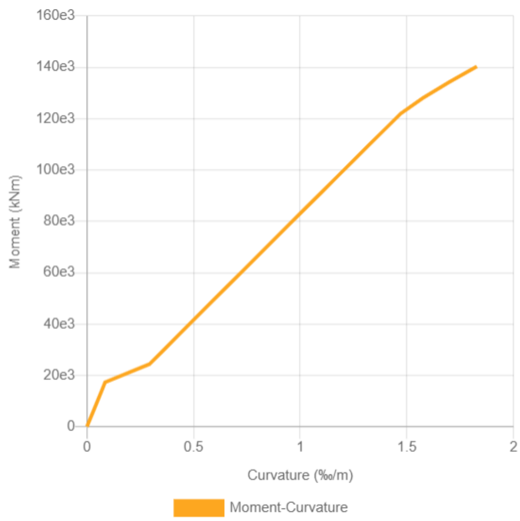




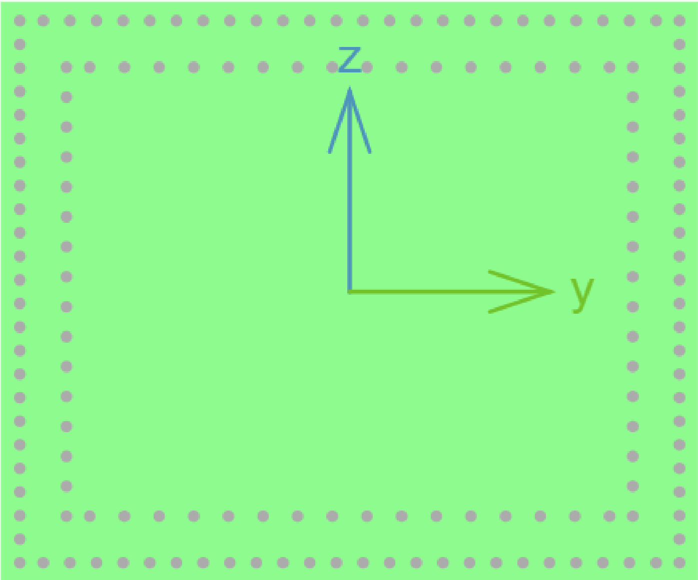
Moment-Stiffness Chart



Moment-Curvature Chart



4.19% reinforcement



· 12.0 REFERENCES

IRC: 112- 2020, *Code of Practice for Concrete Road Bridges (Limit State Design)*, Indian Roads Congress, New Delhi.

· **IRC:136**, *Guidelines for the Design, Construction and Maintenance of Cable-Stayed Bridges*, Indian Roads Congress, New Delhi.

· **IRC:6-2017**, *Standard Specifications and Code of Practice for Road Bridges, Section II – Loads and Stresses*, Indian Roads Congress, New Delhi.

· **IRC:18-2010**, *Design Criteria for Prestressed Concrete Road Bridges (Post-tensioned System)*, Indian Roads Congress, New Delhi.

· **IS: 1343- 2012**, *Code of Practice for Prestressed Concrete*, Bureau of Indian Standards, New Delhi.

· **IRC:83 (Part I & II)**, *Specifications and Guidelines for Bearings for Road Bridges*, Indian Roads Congress, New Delhi.

· **IRC: SP:114-2018**, *Guidelines for Seismic Design of Road Bridges*, Indian Roads Congress, New Delhi.

· **IRC:45-1972**, *Recommendations for Estimating the Resistance of Soil Below the Maximum Scour Level in the Design of Well Foundations*, Indian Roads Congress, New Delhi.

· **IS: 3955- 1967**, *Criteria for Design of Well Foundations*, Bureau of Indian Standards, New Delhi.