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Design and Analysis of Flyover

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***Abstract*- Our project deals with design and analysis of flyover. The manual design of flyover consists of deck slab, longitudinal girder, cross girder, pier, pier cap, abutment, pile cap and pile based on code such as IS: 456-2000 and IRC: 21-2000. Here the structural analysis is carried out by using STAAD Pro V8i software.**

Keywords*: *Deck Slab; Girder; Pier; Pier Cap; Abutment; Pile; Pile Cap.

I. INDRODUCTION

Flyover may be referred as an overpass, a h high-level road bridge that crosses over a highway interchange or intersection. Flyover is a grade separated structure connects road at different levels for the purpose of reducing vehicle congestion. This project topic deals with the analysis of flyover at Kayamkulam – Haripad road using staad pro v8i and manual designing.

II. OBJECTIVE

- Analyse need of flyover at the proposed site.
- Creating model using STAAD Pro V8i.
- Designing the flyover manually and using STAAD PRO.

III. STRUCTURAL INFORMATION

The flyover consists of number of spans with columns (piers), deck slab, girders and abutments etc. Total hight of the structure provided as 6.25m. Grade of concrete and steel provided as M35 and Fe415 respectively. Diameter of the pier taken as 2m. Thickness of the deck slab 0.3m provided. Width of the carriage way is 7.5m

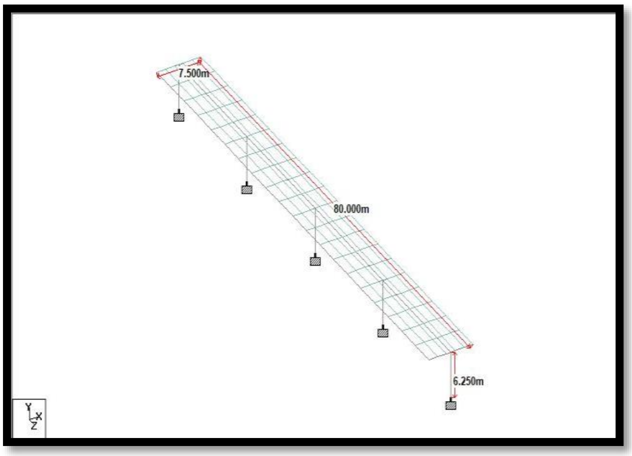

Fig. 1: Structural diagram

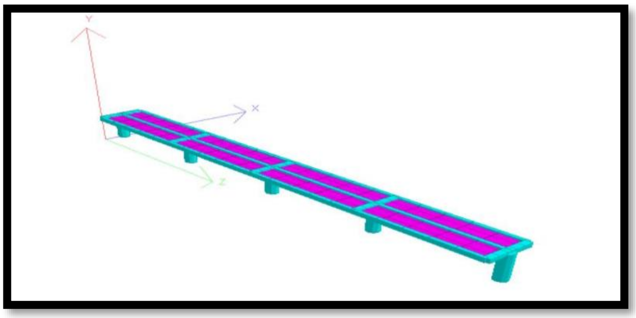

Fig. 2: 3D diagram of the structure

IV. LOAD SPECIFICATIONS

Dead load includes self-weight of the column, slab, girders, abutments, etc. Self-weight of the structure was automatically taken from the software. Different kinds of loads may be estimated by using respective Indian Standard Codes of practice.

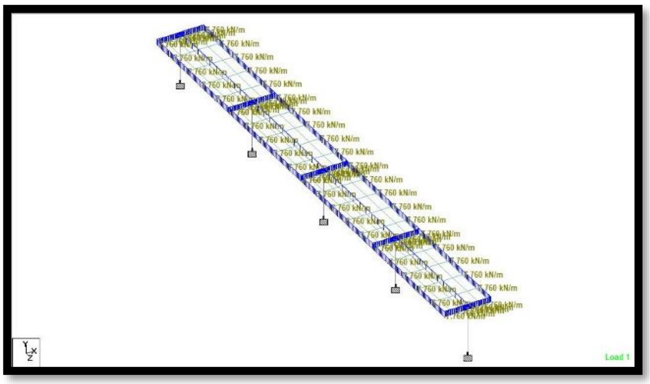

Fig. 3: Dead load diagram

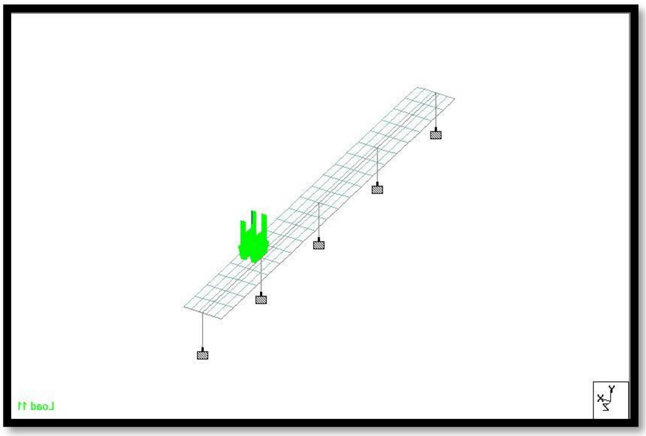

Fig.4: Loading diagram of moving load at mid span

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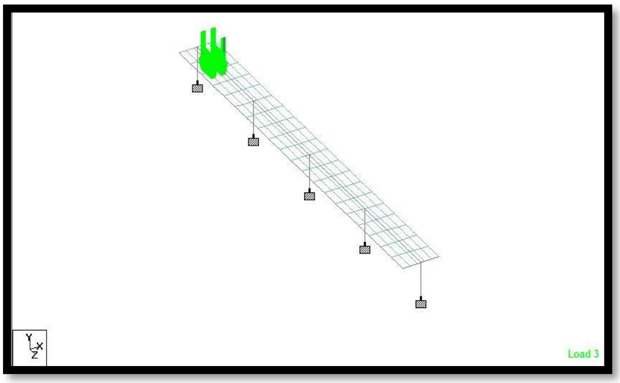

Fig. 5: Loading diagram of moving load at end span

V MANUAL DESIGN AND STAAD ANALYSIS

A. Design of Deck Slab

Centre to center span = 20m
Width of two-way carriage way = 7.5 m (IRC73:1980, clause 6.4
Width of crash barrier = 450mm (IRC5:1998, Page 50)
Wearing coat = 80mm
Loading = IRC Class AA Loading
Depth of slab in assumed to be =300mm
Wearing coat = 40mm and 20mm channel bar
There for, effective depth = 250
Effective Span =6.4m

1) Dead load calculation

Self-weight of deck slab =24$\times$ 0.3 $= 7.2\text{KN/m}^2$
Self-weight wearing coat =22$\times$ 0.08 $= 1.76$
Total dead load =7.5+1.76 =8.96KN/m^2
Maximum dead load of shear force

$= \frac{wl}{2} = 8.96 \times \frac{6.4}{2} = 28.67\text{KN}$

Maximum dead load for Bending Moment $=\frac{WL^2}{8}$

$$= 8.96 \times \frac{6.4^2}{8} = 45.88\ \text{KNM}$$

2) Live load calculation

Angle = 45°
Total width of d = 4.36m
Impact factor (IRC: 6:2016 clause20.6.3 page 50)
Impact factor = 25 - $\frac{25-10}{9.5}$
Impact factor =1.197
Effective width load
b = $\alpha \times\ a(1-\frac{a}{lo}) +\ b_l$
$l_o = 6.4$m
a = $\frac{lo}{2} = 3.2$m
b_l=0.85+2$\times$ 0.08 $= 1.01$mm
Width of slab; b = 8.4m
$\frac{b}{lo} = \frac{8.4}{6.4} = 1.31$
By interpolation, $\alpha = 2.728$
b = 2.728$\times$ 3.2 $\left(1-\frac{3.2}{6.4}\right) + 1.01 = 5.37$m
length = 2.075+2.05+2.685 =6.81m
average live load = $\frac{700\times1.19}{6.81\times4.36} = 28.224\text{kN/m}$
$K_1 = 61.52$kN
K_2 =61.52kN

Live load moment = 61.52$\times$ 3.2 $- 28.222 \times \frac{4.36}{2} \times \frac{4.36}{4}$
$= 130.13$ kNm
Shear force due to live load
For maximum shear at support the jec class AA
$b_0\delta$ = La (L$-\frac{a}{lg}$) + bl
A = $\frac{4.36}{2}$ = 2.18
l_0 = 6.4m
b_l= 1.01m
$b_0\delta = 2.728 \times 2.18 \times \left(1-\frac{2.18}{6.4}\right)$ +1.01 = 5.1m
Total width = 2.55 +2.05+2.55 =7.15m
Average Live Load = 700$\times \frac{1.197}{7.15} \times 4.36 = 26.89\ \text{kN/m}^2$
R_1 = 77.29 kN
R_2 = 40 kN
Maximum live load for shear force V_q = 77.24kN

3) Load Calculation

Design Bending Moment,$M_u = 1.35\ M_{g+1.5\ M_q}$
$= 1.35 \times 45.88 + 1.5\ \times 130.13$
= 257.133 kNm
Design Shear Force,$V_{u=}$ 1.35 $V_{g\ +1.5\ V_q}$
$= 1.35 \times 28.67 + 1.5\ \times 77.24$
= 154.56 kN

4) Design

$M_{u\ limit} = 0.138\ f_1 u\ bd^2$

d = $\sqrt{\frac{M_{u\ limit}}{0.138\ f_{ck}\ b}}$

d = $\sqrt{\frac{257.133\times10^6}{.138\times1.5\times1000}}$ = 230 < 300mm

Hence safe

$$M_u = .87\ f_y\ \text{Ast d}\left\{1-\frac{\text{Ast}\ f_y}{\text{b d}\ f_{ck}}\right\}$$

Ast = 2651.88 mm^2
Use 20mm Ø bars,
Ast = $\frac{\pi}{4} \times 20^2 = 314.15\ \text{mm}^2$
Spacing $= \frac{1000\times314.15}{2651.88} = 100\text{mm}$

$$\text{Ast}_{\text{provided}} = \frac{1000\ \times 314.15}{100} = 314.15\ \text{mm}^2$$

Moment = 0.3 $M_{ul} + .2M_{ud} = 72.3$ kNm

$$\text{Ast}_d = \frac{\text{Ast}_{\text{provided}}}{M_u} \times \text{Moment} = 883.2\text{mm}^2$$

Provide 12mm Ø bar @ 110mm c/c

- Check for ultimate flexural strength
  $M_u = .87\ f_y\ \text{Ast d}\left\{1-\frac{\text{Ast}\ f_y}{\text{b d}\ f_{ck}}\right\}$ = 298.1 kNm > M_u
- Check for ultimate shear strength
  $V_{rdc} = (.12\ \times K\ (80\ \rho_1 \delta_1\ U)^{.33})$bd
  = 197.13 > V_u

Hence safe

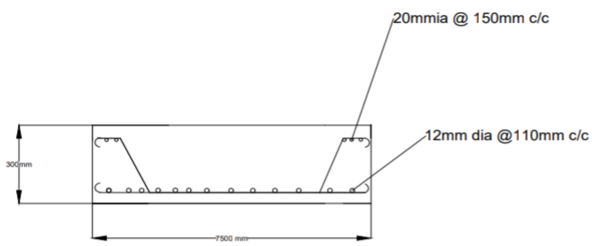

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Fig. 6: Cross section along shorter span

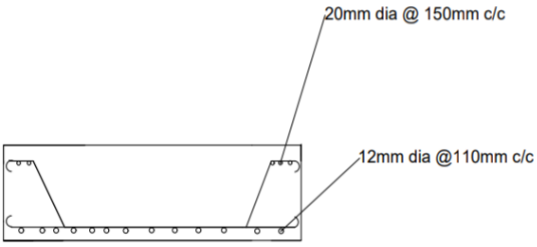

Fig.7: Cross section along longer span

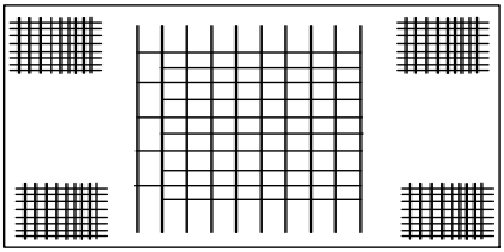

Fig.8: Bottom span

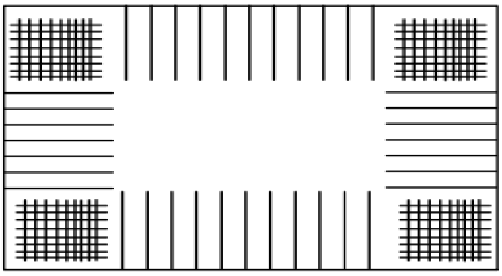

Fig. 9: Top span

B. *Design of Longitudinal Girder*

1) *Reaction factors*

Using Courbon's theory,

- Reaction factor at outer girder(A)

$$R_A = \frac{2W_1}{3}\left[1+\frac{3I\times2.5\times1.1}{2I\times2.5^2}\right]$$
$$= 1.107\ w_1$$

- Reaction factor at inner girder (B)

$$R_A = \frac{2W_1}{3}\left[1+\frac{3I\times2.5\times1.1}{2I\times\theta}\right]$$

If w = axle load + 700kN

Where, $w_1 = 0.5w$

$R_A = 1.107 \times .5w = 0.5536w$ [reaction factor =0.5536]

$R_B = \frac{2}{3}\times .5w = 0.3333w$

2) *Dead load from slab for girder*

Dead load of the deck slab

Parapet railing = 1kN/m

Wearing coat = (22×1.197×0.08) = 2.1KN/m

Deck slab = (24×1.197×0.3) = 8.61KN/m

Total load = 11.71 KN/m

Total dead load of deck = (2×11.71) + (8.96×5.3)
= 70.91 kN/m

Considering the girder to be rigid

$\frac{\text{Dead load}}{\text{girder}} = \frac{70.91}{3} = 23.64$ KN/m

3) *Live load bending moment in girder*

Span of the girder = 20m

Impact allowance 20m span above 9m =10%

The live load is placed centrally on the span.

Bending moment = $\left(\frac{5+4}{2}\right) \times 700 = 3185$ KNm

Area of one side = 0.5×1.8×(5+4.1) =8.19

Area of two side = 2× 8.19 =16.38m

M_{max} = (700/3.6) × 16.38= 3185KNm

Bending moment including impact and reaction factor for outer girder

= 3185 × 1.197× 0.5536 =2110.6KNm

Bending moment including impact and reaction factor of inner girder

B = 3185 × 1.197 × 0.3333 =1270.7KNm

1) *Live load shears in girder*

For estimating the maximum Live load shear in the girder, the IRC class AA loads

Reaction of w_2 on girder B = $\frac{(350\times0.45)}{2.5}$ = 63 KN

Reaction of w_1 on girder A = $\frac{(350\times2.05)}{2.5}$ = 287 KN

Total load on girder B = 350 + 63 =413KN

Total load on girder A = 350 - 63 =287KN

Maximum reaction in girder B

Reaction on girder B = $\frac{(470\times18.2)}{20}$ = 375.83 kN

Girder A

Reaction on girder A = $\frac{(287\times18.2)}{20}$ = 261.17 kN

Maximum Live load shear with impact factor in

Inner girder B = 375.83 × 1.197 =449.87 kN

Outer girder A = 261.71×1.197 = 313.27 kN

5) *Dead load moments and shear force in main girder*

The depth of girder is assumed as 1400mm

Depth of rib = 1.4m

Width of rib = 0.5m

Weight of rib/m = 24×1.4 ×0.5 = 16.8 kN/m

The cross girder is assumed to have the same cross section dimensions of the main girder.

Weight of cross girder = 16.8 kN/m

Reaction on main girder = 16.8 × 2.5 = 42 kN/m

Reaction from the deck slab = 22.64 kN/m

Total dead load / m on girder = 22.64 + 16.8 = 39.44 kN/m

Total downward load = (42×3) + (39.44×20) = 914.8 kN

$R_a = R_b$ = 914.8/2= 457.4 kN

M max at center = (457.4 ×10) - (39.44×10×(10/2)) - (42×5) = 2392 kN-m

Dead load shear at the support = $\frac{(39.44\times20)}{2}$ + 42 + (42/2) =457.4 KN

6) *Design moments and shear force*

Table 1: Design Moments and Shear Force

B.M	DL B.M	LL B.M	TOTAL B.M
Outer Girder Inner Girder	2392 kNm 2392 kNm	2110.6KNm 1270.7 kNm	4502.6 kNm 3662.7 kNm
S.F	DL S.F	LL S.F	TOTAL S.F
Outer Girder Inner Girder	457.4 kN 457.4 kN	313.27 kN 449.87 kN	770.67 kN 907.27 kN

Max B.M = 4502.6 kN-m

Max S.F = 907.27 kN

Effective depth = 1450mm

Approximate Lever arm = 1400-100 = 1300mm

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Ast = $(4502.6 \times 10^{\wedge}6)/(200 \times 0.9 \times 1300)$
= 19241.88mm^2
Provide 32mm ϕ bars
No of bars = 19241.88/((3.14×32^2)/4) = 24

7) Design of section for maximum B.M and S.F

Nominal shear stress, $\tau_V = \frac{V}{bd}$
τ_v = (907.27× 10^3)/ (500×1400)
τ_v= 1.3 N/mm
$\tau_{c=}$ 100As/bd = (100×19241.88)/ (500×1400) = 2.75
= 0.6 N/mm^2

$$\tau_V > \tau_c$$

Hence safe provide shear reinforcement
Assume 2 bars of 32mm Ø is bent up
$U_s = \frac{\sigma_{sv}A_{sv}d}{S_v}$ (sin ∝)
α = 45°
U_s = 200×2×$\frac{\pi}{4}$ × 32^2 × $\frac{1}{\sqrt{2}}$= 227.36KN
Balance shear = 907.27 - 227.36 = 679.91 KN
Using 10mm Ø, 4 legged stirrups

$$S_V = \frac{\sigma_{sv}A_{sv}d}{V}$$

= (200×4×(3.14/4)× (10^2)×1400)/679.91= 130mm
Provide 10mm Ø 4 legged stirrups at 120 mm c/c

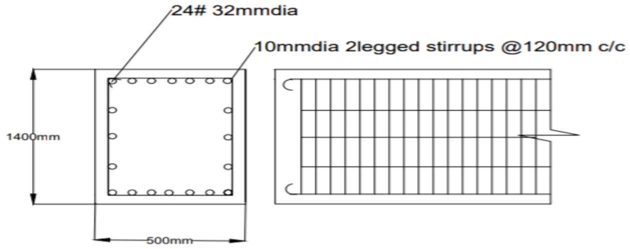

Fig. 10: Detailing of longitudinal girder

C. Design of Cross Girder

1) Dead load calculation

Self-weight of the cross girder (same as longitudinal girder size) = 16.8 KN/m Dead load from slab

= 2×(1/2)×2.5×1.25×8.96 = 28 KN

Uniformly distributed load = 28/2.5= 14 KN/m
Total load on cross girder = 16.8+14 = 30.8 KN/m
Assuming the cross girder to be rigid
Reaction on each cross girder = (30.8×5)/3 = 51.33 kN
Dead load shear = 51.33 kN

2) Live load calculation

Load coming on the cross girder =271.25KN
Assuming the cross girder as rigid, reaction on each longitudinal girder is = $\frac{2 \times 271.25}{3}$
Live load bending moment including impact
= 1.25x180.53x1.475
{for 5m span for c/s is 25%} =332.85KNm
Dead load bending moment at 1.475m from support
= 51.33×1.475-(30.8×1.475) × (1.475/2)
= 42.2 KNm
Total bending moment
= 332.85+42.2 = 375.05
kNm

Live load shear including impact = $\frac{2 \times 271.25}{3} \times 1.25$
(reaction on each longitudinal girder) = 226.04KN
Dead load shear = 51.33 kN
Total shear = 226.04+51.33
= 277.37 kN

3) Design moments and shear force

Maximum bending moment = 375.05 kN-m
Maximum shear force = 277.37 kN
Effective depth = 1400 mm
Approximate lever arm = 1300mm
Ast = $\frac{375.05 \times 10^6}{200 \times 0.9 \times 1300}$ = 1602.78mm^2

Provide 32 mm Ø bars
No of bars = $\frac{1602.78}{\frac{\pi}{4} \times 32}$= 2

4) Design of shear

Nominal shear stress, $\tau_v = \frac{V}{bd}$
= $\frac{277.37 \times 10^3}{500 \times 1400}$ (277.37×10^3)/ (500×1400) =0.4N/mm^2
100$\frac{As}{bd}$ = $\frac{100 \times 1602.78}{500 \times 1400}$) = 0.23
τ_c = 0.224 N/mm^2

$$\tau_v > \tau_c$$

Hence safe provide shear reinforcement
Assume 2 bars of 32mm Ø as bent up
Us= $\frac{\sigma_{sv}A_{sv}d}{s}$ (in ∝)
α = 45
Us = 200×2×$\frac{\pi}{4}$ × 32^2 × $\frac{1}{\sqrt{2}}$ = 227.36 KN
Balance shear = 277.37 - 227.36 = 50.01 KN
Using 10mm Ø, 4 legged stirrups

$$S_v = \frac{\sigma_{SV} \times A_{SV} \times d}{V}$$

$$= \frac{200 \times 4 \times \frac{\pi}{4} \times 10^2 \times 1450}{256.92} \left| \frac{200 \times 4 \times (\frac{3.14}{2}) \times (10^2) \times 1400)}{277.37 \times 10^3} \right| = 317\text{mm}$$

Provide 10mm Ø 4 legged stirrups at 300mm c/c

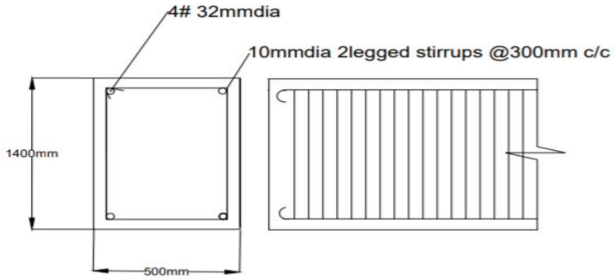

Fig.11: Detailing of cross girder

D. Pier cap

➢ Design Procedure

Design of hammer head portion over circular pier for the following details

Live load: IRC Class AA Tracked vehicle
Materials: M35 grade concrete and Fe 415 steel

➢ Data

Clear projection of cantilever slab = 3200mm
Thickness of wearing coat =80 mm
Materials: M35grade concrete and Fe 415 steel.

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Live load is IRC class AA tracked vehicle.

- Permissible stresses (IRC: 21):

For M35 grade concrete and Fe415 steel.

σ_{cb} = 11.67 N/mm, m =10, σ_{st}= 200N/mm, j = 0.9, Q = 1.93

1) Calculations of moments

Total dead load moment,

Mg = 70.56+105.84+4.2+15.52+30.24+508.032+432.36 =1166.752 kNm

- Live load moment

The live load is IRC class AA tracked vehicle. This is placed with its edge 1200 mm from the kerb.

Effective width of dispersion perpendicular to span is given by

b_e= 1.2x + b_w

x = 0.1m

b_w =[0.85+2x0.075] =1m.

Therefore be= (1.2x0.1) +1=1.12m.

Live load per meter width including impact =2110.6 KNm

2) Design moment

Design moment, M=1166.752+2110.6

Factored moment = 4916.028 KNm

3) Reinforcements

Effective depth required

$2Q_{bd}$ = maximum bending moment

d = $\left[\frac{(4916.028\times10^6)}{(1.93\times1000))0.5}\right]$ = 1595.98mm

Effective depth required =2200 - 50

= 2150 mm > 1595.98 mm

Hence adopted depth is adequate

Ast =$\left[\frac{(4916.028\times10^6)}{(200\times0.9\times2150)}\right]$ = 12702mm^2

Use 32 mm Ø bars

Ast = $\left[\frac{(3.14\times32^2)}{4}\right]$ = 804.24mm^2

No of bars = $\frac{12702}{804.2}$ = 16

However, provided more for effective reinforcement than required.

- Top reinforcement:
  Provide 30 numbers of 32mm Ø bars in 2 layers
- Side reinforcement:
  Provide 10 numbers of 16mm Ø bars on each face equally spaced
- Inclined reinforcement:
  Provide 10 numbers of 16mm Ø bars on each face equally spaced.
- Shear reinforcement:
  Provide reinforcement 12mm Ø 4-legged stirrups @ 150 mm/cc.

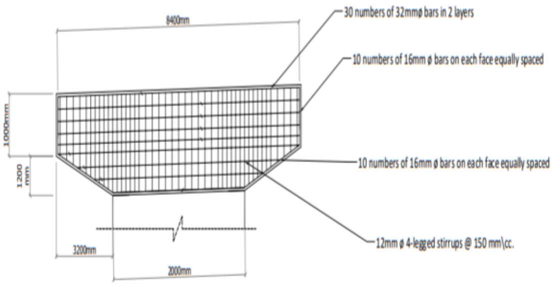

Fig. 12: Detailing of Pier cap

E. Pier

Live load: IRC Class AA tracked vehicle

Materials: M35 grade concrete and Fe 415 steel

- Calculation of loads

Data

Effective span of girder bridge = 20 m

Clear width of roadway = 7.5 m

Live load on bridge class AA

Height of pier = 6.250 m

Height of flood level = 2 m

Dead load of super structure per span equal to dead load coming outer girders and inner girders.

Assume weight of bearing, plate etc. as 10 KN

Dead load of pier cap:

The pier cap is divided into two cantilevers and one rectangular section Weight (moment)of two trapezoidal sections = area × unit weight of concrete =322.56 KNm

Dead load moment of circular pier

= $\left[\frac{3.14\times2^2}{4}\right]$ ×6.250×24=471.24 KNm

- Stresses Due to Live Load

Reaction due to live load class AA loading including impact = 1.197×70 = 83.79 kN

Maximum bending moment = 83.79× 0.5 = 41.895 KNm

Maximum and minimum stresses at the base due to this load will be

σ_c= 5.74 kN/m^2 and -0.916 kN/m^2

- Stresses Due to Longitudinal Force

Maximum longitudinal force will occur due to class AA loading = 14 kN

Moment at base of pier due to this force = 100 × 14 = 140 kNm

Stress at base =$\frac{140\times14}{I_{xx}}$ = 12.106 kN/m^2, Assume coefficient of friction as 0.25 at one bearing and 0.22 at other bearing Total resistance at one set of bearing with DL and LL

= 0.25 (166.85 + 1.197×700) = 234.212 kN

Total resistance at other set of bearings due to DL only

= 0.22 × 166.85=36.707 KN

Unbalanced force at top of the pier = 234.212 – 36.707 = 197.505 kN

Moment at base = $\frac{197.505\times1.4}{19.847}$ = 13.932 kN/m^2

- Stresses Due to Wind Loads

Exposed height of the structure

= depth of girder + thickness of slab + height of railing

= 1.4 + 0.23 + 1.45 = 3.05 m

Exposed area contributing wind pressure per pier

= span × height

= 20 × 3.141 = 62.831 m

Assume average height of about 5 m

The wind pressure from table is 76.96kg/m^2

i. Wind force on exposed surface = $\frac{62.831\times76.96}{1000}$

   = 4.83 kN

ii. The design wind load should not be less than 450 kg/m of the loaded chord.

Hence minimum design wind force = $\frac{20\times450}{1000}$ =9 KN

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iii. The wind force for design purposes should be less than 240 kg/m^2

For the unloaded structure = $\frac{62.831 \times 240}{1000}$ =15.079KN

Condition (iii) gives maximum wind force. Hence this is considered in design.

Assume this to act at mid height,

$\frac{3.141}{2}$ = 1.57m from the top of pier

Moment at base about Y axis Mw = 15.079 × (10 + 1.57)
= 174.964 kNm

The bending takes place about Y axis.

Maximum stresses at the base at end of straight portion.

$\sigma_{w1} = \frac{M_w}{I_y} \times 3.6 = \frac{174.464}{184.711} \times 3.6 = 3.345\ kN/m^2$

Maximum stress at the edge of the pier = $\frac{M_w}{I_y} \times 5 = \frac{174.464}{184.711} \times 5 = 4.722\ kN/m^2$

- Total Stresses
  Under dry conditions

Total stress = DL + LL + WL

At end of straight portion,

Maximum = 9.409 + 5.741 + 12.106 + 142.144 - 3.345 + 4.722 = 177.467 kg/m^2

Minimum = 9.409 - 5.741 - 12.106 - 142.144 - 3.345 - 4.722 = -158.649 kg/m^2

At end of pier,

Maximum = 9.409 + 4.722 = 14.131 kg/m^2

Minimum = 9.409 - 4.722 = 4.687 kg/m^2

Under wet conditions, Total stress = DL + LL + WL

At end of straight portion,

Maximum = 9.409 - 6.609 + 5.741 + 12.106 + 142.144 + 3.345 +1.373 = 1464.673 kg/m^2

Minimum = 9.409 - 6.699 - 0.916 - 12.106 - 142.144 - 3.345 - 1.373 = -157.174 kg/m^2

At end of pier,

Maximum = 9.409 - 6.699 + 4.722 - 1.907 = 9.339 kg/m^2

Minimum = 9.409 - 6.699 - 4.722 - 1.907 = -3.919 kg/m^2

Allowable compressive stress in 1:3:6 concrete is 2000 kg/m^2& 250 kg/m^2 intension. The stresses in pier are within these permissible limits.

Weight of IRC Class AA tracked vehicle is 700 KN

Total load = dead load + live load = 1929.389+700
= 20629.389 KN

Total load with impact = 20629.389 ×2= 5258.778kN

Factor of safety = 2

Factored load = 5258.778×2 = 10517.556 kN

Factored load P_u=10517.556 kN.

e is the eccentricity of the wheel load from center.

e = 1.1m

Live load = 700×2 = 1400 KN

Maximum moment = 1400×1.197 = 1675.8 KN

Moment with impact = 700×1.197= 837.9 KN

Factored moment = 1675.8 ×2.2 = 3686.76KN-m

Therefore, factored moment = Mu = 3686.76 KN-m

1) Non dimensional parameters

$\frac{P_U}{Fck\ d^2} = \frac{10517.553 \times 10^3}{35 \times 2000^2} = 0.1$

$\frac{M_U}{Fck\ 3} = \frac{3686.76 \times 10^6}{35 \times 2000^3} = 0.02$

Ratio $\left(\frac{d}{D}\right) = \frac{60}{2000} = 0.03$

Where D is the diameter of the circular pier = 2000 mm

d is the clear cover = 60 mm

By referring chart number 55 of SP 16

Where P is the percentage of steel reinforcement

P = 0.01 × 20 = 0.2

Area of steel = 0.2 $\pi \times \frac{2000^2}{4}$ = 6283.18 mm²

Use 25 mm Ø bar

Ast =$\frac{3.14 \times 25^2}{4}$ = 490.87 mm²

Number of bars = $\frac{6283.18}{490.87}$ = 14

However, provide 32 numbers of 25 Ø mm bars around the circular pier.

Using 10 mm Ø lateral ties

Spacing is the least of the following

1. Least lateral dimension = 2000 mm
2. 16×25 = 400 mm
3. 300 mm

Hence provide 10 mm Ø bars of lateral ties @ 300 mm c/c

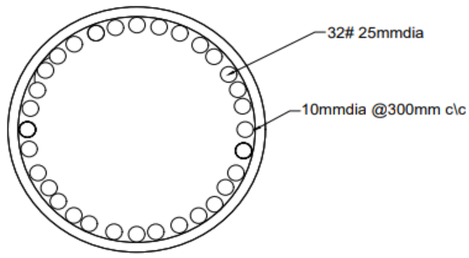

Fig.13: Detailing of pier

F. Design of Pile Cap

1. Data

Total Load on the column pu =10517.556 KN

Diameter of column = 2m

Pile diameter = 300mm

Allowable load on each pile = 1350KN

Grade of concrete for pile cap = M50

2. Size of pile cap

Number of piles required, N=$\frac{10517.5}{1350}$ = 8

Let us provide 8 nos of pile

Spacing of piles: (IS 2911:1/2, 2010)

S= 4×300=1200mm

3.Overhang portion of the pile cap

overhang = 2×300=0.6m

Thus, the total width and breadth of the pile cap is

B_{cap} = (2×1.2) +(2×0.6) =3.6 m

W_{cap}= B_{cap}

Hence let as assume pile cap size 3.6 m x 3.6 m

4. Thickness of pile cap based on shear

Pile reaction; R= $\frac{10517.556}{8}$ = 1314.69 kN

One way shear,

The critical section is at a distance d from the face of the column

Clear distance c = 2.4-2-0.3=0.1m

Considering tolerance of 50mm

Clear spacing = 100+50=150mm

r = $\frac{150+300-d}{300}$

V_{ul}=3×R×r = 3× $\left(\frac{150+300-d}{300}\right) \times 1.3 \times 10^6$

=5916105-13146.9d
Assuming percentage of steel 0.25& for M50 grade concrete T_C=0.38 N/mm2

a) One way shear resistance

V_{cl} = 0.38×3600×d = 1368d
$V_{cl} > V_{ul}$
D >407.588mm
therefore d = 408mm
Since, computed value of d
c=150 mm
c+d=450 mm
d > 487 mm
Hence assumption is correct

b) Two-way shear

The critical face is at a distance $\frac{d}{2}$ from the face of the column.
Let us assume that
$\frac{d}{2}$=c
Factored shear force.
V_{u_2}=8×1314.69 = 10517.52kN
Two-way shear resistance
K_s = 1
T_c= 0.25× √Fck=1.77 N/mm^2
V_{c_2}= Ks×Tc×(4×(2000+d) d) = 14160d+7.08d^2
$V_{c_2} > V_{u_2}$
d=577mm
$\frac{d}{2}$ > c
assumption that full pile reaction is not valid
V_{u_2} = R-(R×(1-r_1)×(1-r_2))
$r_1 = r_2 = \left(\frac{150+300-d}{300}\right)$
V_{u_2}=8×1.3×10^6×(1-(1- r_1)^2)
$V_{c_2} > V_{u_2}$
d = 651 mm

c) Around the pile punching shear

Factored shear force, V_{up}= 1000kN
Two-way shear resistance, V_{cp}= 1×1.77×(pi×(300+d)×d)
$V_{cp} > V_{up}$
d = 376 mm

d) Shear around the pile along ABCD

Vcp = 1×1.77× $\left(\frac{\pi+300+d}{4}\right)$+2×600)
d = 488 mm
hence the effective depth is considering critical case of two-way shear around the column d = 651 mm
D= 651+16+150 =817 mm
Let us assume D = 850 mm
d = 850-16-150=684 mm

d) Design of flexural reinforcement

Mu=3×R(c+$\frac{d_p}{2}$)= 1183.221 kNm
B= 3600 mm
D = 684 mm
$A_{st\ required} = f_{ck}\times b\times d\times\frac{(1-\sqrt[2]{1-4.598\times\frac{m_u}{f_{ck}\times b\times d^2}}}{2\times f_y}$
= 4871.48 mm^2
A_{min} = 0.0012×b×D = 2954.88 mm^2
$A_{assumed}$ = 0.0025×b×D = 6156 mm^2
Assuming 16 mm Ø bars
No of bars required =$\frac{6156}{P_i\times\left(\frac{16^2}{4}\right)}$ = 31
Spacing =$\frac{3600-(31\times16)-2\times75}{31-1}$ = 98.466 mm
S = 90 mm c/c

e) Development length.

T_{bd} = 1.9 N/ mm^2
$L_{d}=\frac{(0.87\times f_y)\times16mm}{4\times1.6\times T_{bd}}$ = 0.475 m
Length available; L =0.8 m
L_d < L

f) Transfer of force at column base

Column to pile cap intraction

- For column face.

$A_1 = A_2 = \frac{\pi}{4}$×2000^2
Fck_{column}= 50 N/mm2
$\left(\frac{A_1}{A_2}\right)$ = 1
$F_{br\ max\ column}$ = 0.45×50×1 = 22.5 N/mm^2

- For pile cap face.

A_1 =3600×3600
$A_2 = \frac{\pi}{4}$×2000^2
$Fck_{PILECAP}$= 50 N/mm^2
$\sqrt{\frac{A1}{A2}}$ () = 4.125 (limited to 2)
$F_{br\ max\ pilecap}$ = 0.45×50×2 = 45 N/mm^2
Evidently the column face governs
Limiting bearing resistance
F_{br} = 9232.5 kN
P= 10517.556 KN
Here, $F_{br} < p_u$
Excess force; P = 1285 kN
If column bars are extended into the footing, then, force per bar
F_{bar} = 1285/32= 40.15 kN
corresponding stress in bar = $\frac{40.15\times10^3}{\frac{\pi}{4}\times25^2}$ = 81.79 N/mm^2
development length required to take the stress
$l_d = \frac{81.79\times25}{4\times1.6\times1.25\times1.9}$ = 134.5 mm
Available vertical embedment in the footing d=654 mm more than Minimum
l_d= 135 mm
Astinterface_min=0.005×$\frac{\pi}{4}$× 2000^2= 15.7 ×10^3 mm^2
Ast interface provided = 32×$\frac{\pi}{4}$× 24^2 =31.41×10^3 mm^2
hence area provided is well over minimum interface reinforcement required.

g) Transfer of force at pile-to-pile cap

Factored reaction from pile; P_{upile} =1000kn
Column to pile cap interaction:
For pile face
$A_1 = A_2 = \frac{\pi}{4}$× 300^2
Fck_{pile} = 50 N/mm^2
root$\left(\frac{A1}{A2}\right)$= 1
$F_{br\ max\ pile}$ = 0.45×50×1 = 22.5 N/mm^2

h) Pile cap face

$A_1 = \frac{\pi}{4}$× 1200^2

$A_2 = \frac{\pi}{4} \times 300^2$
$Fck_{PILECAP}$ = 50 N/mm²
$root\left(\frac{A1}{A2}\right)$ = 4 (limited to 2)
$F_{br\ max\ pilecap}$ = 0.45×50×2 = 45 N/mm²
Evidently the column face governs
Limiting bearing resistance
F_{br} = 1590.43 kN
P= 10517.556 KN
Here, $F_{br} > p_u$
Hence Safe

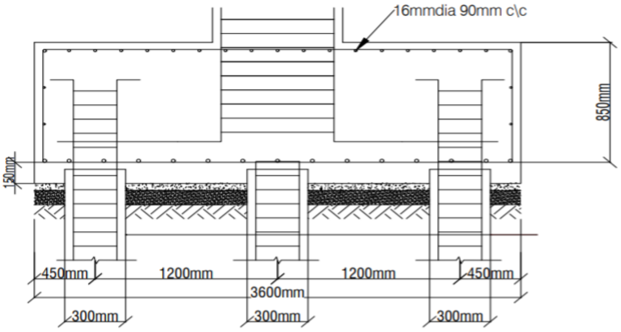

Fig.14: Detailing Pile and pile cap

E. *Design of Abutment*

Density of soil = 16.42 kN/m³
Coefficient of friction = 0.6
Angle of repose = 30°
Live load on bridge = IRC Class AA load
Angle of friction between soil and concrete = 18°
Longitudinal girder = 3 number of 1.4 m × 0.5 m
Deck slab = 0.3 m depth
Span of the bridge =20 m
Self- weight,
W_1 = ((0.5×1.24×.4) +(0.5×1.2×3.7) +(0.6×4.4) +(0.8×3.7)) ×24= 251.04 kN

➢ Dead load

$W_2 = L_1 + L_2$
L_1 = Load from longitudinal girder
= 3 × 1.4 × 0.5 × 20 × 24 = 1008 kN
L_2 = Load from slab and wearing course
= (0.3 × 8.4× 20 × 24) + (0.8 × 7.5 × 20 × 22) = 3849.6KN
W_2 = 4857.6 kN
Considering dead load (super structure) shared by an abutment and a pillar

$$= \frac{4857}{2} = 2428.8 \text{ kN}$$

Dead load per meter span of abutment $W_2 = \frac{2428}{84} =$
= 289.14 KN

Live load reaction = W × L
For Class AA tracked vehicle, W = 350 Kn

$R_{A=350 \times}\left(\frac{20-\text{Wheel load}}{20}\right)$

$R_{A=350 \times}\left(\frac{20-\frac{3.6}{2}}{20}\right)$

= 318 kN
Total load carrying on the abutment
$W = W_1 + W_2 + R_A$
= 251.04+ 289.14 + 318.5 = 858.68 kN

➢ Earth pressure

Lateral earth pressure k_a= 0.408
Horizontal component of earth pressure = total earth pressure × cos (δ + θ)
Total Earth pressure = $\frac{1}{2} \times \delta \times h^2 \times \cos\theta \times k_a$
=0.5×16.42×4.4² ×cos15.25×0.408 = 62.56 KN
Horizontal component = 62.56 cos (16.42 + 15.255)
= 53.24 kN
Vertical component = 62.56 sin (16.42 + 15.255) =32.85KN
R = $\sqrt{\sum V^2 + \sum H^2}$ = ((32.85²)+(53. 85²)) × 0.5 =63.07KN

➢ Overturning

Earth pressure is acting at 0.42 of height from base
= 0.42 × 4.4 = 1.848 m
Moments due to overturning = 1.848×53.24= 98.39 kNm
Restoring moments= (0.5×1.2×4.4×24×3.0) +(0.6×4.4×24×2.3) + (0.8×3.7×24×1.6) +(0.5×1.2×3.7×24×0.8) +(858.68×1.6) =1865.984 kNm
Factor of safety against overturning =1865.984/98.93
= 18.86

➢ Sliding

Factor of safety = (0.6×858.68)/53.24 = 9.6

➢ Base pressure

Distance of the resultant from the toe.

$$x = \frac{(\text{restoring moment} - \text{moment due to overturning})}{\text{resultant}}$$

$$= \frac{1865.986-98.39}{858.68} = 2.058\text{m}$$

Eccentricity of the resultant from the centre of the base.
$e = \frac{b}{2} - x = 0.022$
Maximum pressure, $P_{max} = \frac{858.68}{3.8} \times \left(1 + \frac{6\times.022}{3.8}\right)$ =233.82k
= 233.82km/m²
P_{min}= (858.68/3.8) (1-((6×0.022)/3.8) = 218.12 km/m²

VI CONCLUSION

- The proposed project could help rectify the traffic conjunction problems and improve safe driving.
- Project is designed manually and using STAAD Pro V8i.
- Amount of steel provided for the structure is economic.
- Structure is designed on the basis of IRC class AA loading.
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