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Analysis and Design of T-Beam Bridge Over Rangiya Nala

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Abstract

Bridges serve as the means of transportation over river or valley or to provide ease of transportation over crossings. They act as catalyst in booming the local economy. Design of bridges has a special importance in the field of civil engineering. Bridges range from timber to pre-stress. However, the design of T-beam bridge is always a better choice where underneath transportation is prohibited during construction and the span of the bridge is restricted to 25 meters. T-beam bridge decks are one of the principal types of cast-in place concrete decks with concrete slab integral with girders. The analysis procedure also changes with time i.e. initially from working stress method to ultimate limit state method. Different countries have adopted different methods whereas in India the limit state method of design is prevalent due to its economy and predictability. IRC: 112, the code of practice for concrete and pre stressed bridges, restricts the use of working stress method of analysis in the design of bridges. The proposed bridge has two piers against four piers of the existing bridge in order to allow more waterways. Also due to reduction in number of piers, piles and pile caps, better economy is achieved in terms of approximate 30% material reduction when the number of piers is halved. Under-reamed pile foundation is provided keeping the soil strata in mind as well as from reference of National Highway Division.

Keywords: T-beam bridge, limit state method, working stress method, pile caps, IRC

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INTRODUCTION

The obstacle to be crossed may be a river, a road, railway or a valley. Bridges range in length from a few meters to several km. They are among the largest structures built by man. The demands on design and on materials are very high. A bridge must be strong enough to support its own weight as well as the weight of the people and vehicles that use it.

The structure also must resist various natural occurrences, including earthquakes, strong winds, and changes in temperature. Most bridges have a concrete, steel, or wood framework and an asphalt or concrete road way on which people and vehicles travel. The T-beam Bridge is by far the Most commonly adopted type in the span range of 10–25 M. The structure is so named because the main longitudinal girders are designed as T-beams integral with part of the deck slab (Figure 1), which is cast monolithically with the girders. Simply supported T-beam span of over 30 m are rare as the dead load then becomes too heavy.[1, 2]

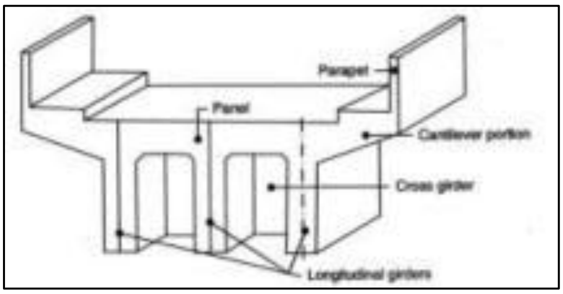

Fig. 1. *Components of T-Beam Bridge.*

Area under Study

The catchment area from the topographic map has been presented in Figure 2. An area of catchment is approximately 212 km^2.

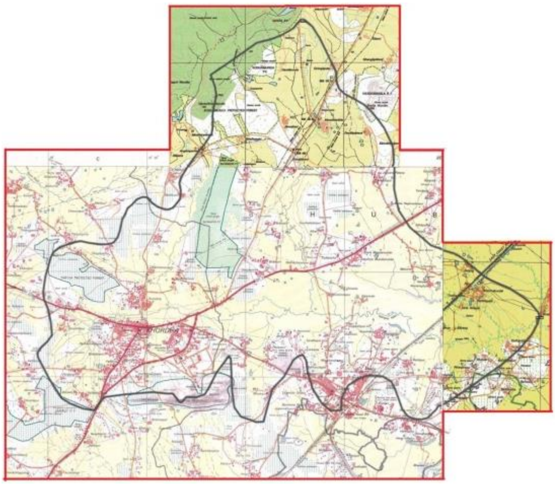

Fig.2. *Catchment of the Study Area.*

The study area is located in the outskirt of Bhubaneswar City and the bridge that is to be designed is located on Rangiya Nala near the village Birabandha some 10 kms away from Bhubaneswar airport.

METHODOLOGY

The most commonly used method (Pigeaud's method) has been adopted to design the deck slab with cantilevers. The detail method has been elaborated as follows:

Pigeaud's method for the design of bridge slabs consists of a series of charts that are used for determining the longitudinal and transverse bending moments in a bridge slab due to a wheel load occupying a small rectangular area.

The sides of the loaded area are assumed to transmit themselves at an angle of 45° through the non-structural surfacing. From the geometry of the tyre pressure area the width 'u' and the length 'v' are calculated. According to the geometry of the slab and beam system, the slab's span between diaphragms 'a' and the slab's width between the longitudinal beams 'b' are calculated. The ratio of a/b determines which chart to use, and plotting the values of u/a and v/b result in a moment M_1 or M_2 which are function of 'a' and 'b' respectively are determined.

From the results of M_1 and M_2, M_a and M_b are calculated from the following equation where

$M_a = (M1 + 0.20M_2)$ P in kN-m
$M_b = (M2 + 0.20M_1)$ P in kN-m
P is the concentrated load or wheel load.

M_a and M_b are the transverse and longitudinal moments per unit width respectively and can be considered as positive at the mid span of the panel and negative over the supports. Pigeaud did make recommendation of multiple wheel loads on slab but did not take into account the continuity of the slab over the supports.

Once the moments have been calculated the slab can be designed with ordinary reinforced concrete fundamentals for the beam of unit width. Pigeaud's method is of most use in slabs where the width is less than 1.8 times the span.[3,4,5]

Limitation
Only loads placed at the center can be considered. In practice, a number of wheel loads may occur in a single slab panel. While one load can be placed at the center and the others are non-central. Some approximations are to be used while considering the non-central loads.

Tracked vehicle gives maximum effect along short span/direction but along long span, the wheeled vehicle gives critical effect.[6]

The following load positioning gives the worst effect among other combinations (Figure 3).

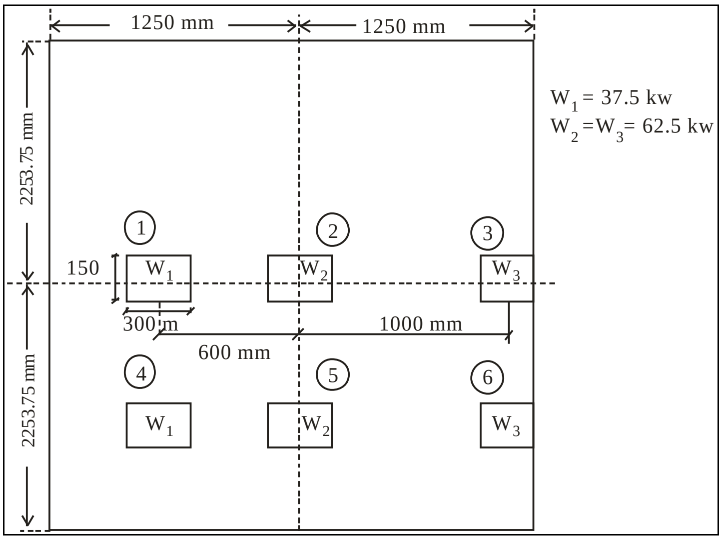

Fig.3. *Loading of Wheeled Vehicle for Critical Effect.*

As per IRC-6:2014 specifications, no vehicle is allowed to occupy the same panel during the above arrangement.[7]

RESULTS AND DISCUSSION

Bending Moment due to Wheel Load-1
Since Pieguad's curve is applicable to only symmetrical loading pattern, it cannot be used directly for unsymmetrical loading. In order to calculate the moment, a dummy load having equal magnitude will be arranged at equal distance as per real load from the center of slab or from the vertical axis of the slab. The arrangement is shown in Figure 4.

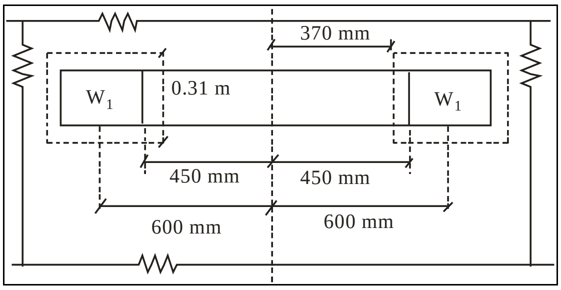

Fig.4. *Bending Moment Calculation due to Wheel Load-1.*

So u = 2(u_1 + x) = 2(0.46 + 0.37) = 1.66m and v = 0.31 m
Load intensity = 37.5/(.31×.46) = 262.973 kN/m^2
K = B/L = 0.55, u/B = 0.664 and v/L = 0.069
Hence m_1 = 0.12 and m_2 = 0.14
M_B = (0.12+0.2×0.14)262.973×1.66×0.31 = 20.03 kN-m
Similarly M_L = 22.193 kN-m
Now u = 2x = 2×0.37 = 0.74 and v = 0.31
K = 0.55, u/B = 0.296 and v/L = 0.069

From Pieguad's curve, m_1 = 0.184 and m_2 = 0.189
M_B' = 13.38 kN-m and M_L' = 13.622 kN-m
Hence total moment is given by,
M_{B2} = (20.03 – 13.38) ×0.5 kN-m = 3.325 kN-m
M_{L2} = (22.193 – 13.622) ×0.5 kN-m = 4.285 kN-m

Bending Moment calculation due to Wheel Load-2:
Tyre contact dimension: 300 mm × 150 mm
So u = 0.3+2×0.080 = 0.460 m and v = 0.15+2×0.080 = 0.310 m
The value of u/B = 0.184, v/L = 0.069 and B/L = 0.55
Using Pieguad's curve, m_1 = 0.22 and m_2 = 0.20
Hence M_{B1}= 62.5 (0.22+0.2×0.2) = 16.25 kN-m and M_{L1} = 62.5(0.2+0.22×0.2) = 15.25 kN-m

Bending Moment due to Wheel Load-3:
Now in this case u = 2(0.46+0.77) = 2.46m, v = 0.31 m
Load intensity is 62.5/(.31×.46) = 438.3 kN/m^2
K = 0.55, u/B = 0984 and v/L = 0.069

From Pieguad's curve, m_1 = 0.09 and m_2 = 0.09
M_B = 36.10 kN-m and M_L = 36.10 kN-m
Taking u = 2x = 2×0.77 = 1.54 and v = 0.31,
K = 0.55, u/B = 0.616 and v/L = 0.069
Hence m_1 = 0.125 and m_2 = 0.14
M_B' = 32.01 kN-m and M_L'= 34.52 kN-m
M_{B3} = [36.10 - 32.01] ×0.5 = 2.045 kN-m
M_{L3} = [36.10 - 34.52] ×0.5 = 0.79 kN-m

Bending Moment due to Wheel Load at-4:
u = 2(u_1+x) = 2(0.46+0.37) = 1.66m and v = 2(v_1+y) = 2(0.31+1.045) = 2.71m
K = 0.55, u/B = 0.664 and v/L = 0.601

Coefficient $m_1 = 0.09205$ and $m_2 = 0.027$
After multiplying with $(u_1+x)(v_1+y)$ with both coefficients,
$m_1 = 0.1015$ and $m_2 = 0.0304$
$u = 2x = 2\times0.37 = 0.74m$, $v = 2y = 2\times1.045 = 2.09m$
$K = 0.588$, $u/B = 0.296$ and $v/L = 0.464$
So $m_1 = 0.134$ and $m_2 = 0.0405$
Multiplying with xy,
we get $m_1' = 0.052$ and $m_2' = 0.016$
$u = 0.74m$, $v = 2.71m$
$K = 0.55$, $u/B = 0.296$ and $v/L = 0.601$
$m_1 = 0.12$ and $m_2 = 0.03$ and multiplying with $x(v_1+y) = 0.5013$,
$m_1' = 0.060$ and $m_2' = 0.015$
$u = 1.66$ m and $v = 2.09m$
$K = 0.55$, $u/B = 0.664$ and $v/L = 0.464$
$m_1 = 0.095$ and $m_2 = 0.038$
Multiplying with $y(u_1+x) = 0.87$,
$m_1' = 0.083$ and $m_2' = 0.033$
So $m_1 = [(0.1015+0.052)-(0.060+0.083)] = 0.0105$, $m_2 = [(0.0304+0.016)-(0.015+0.033)] \approx 0$
$M_{B4} = \frac{37.5}{0.46\times0.31}[0.0105+0.2\times0] = 2.761$ kN-m, $M_{L4} = \frac{37.5}{0.46\times0.31}[0.2\times0.0105+0] = 0.552$ kN-m

Bending Moment due to Wheel Load at-5:
$u = 0.46m$, $v = 2(v_1+x) = 2(0.31 + 1.045) = 2.71m$
Load intensity $= 62.5/(0.46\times0.31) = 438.3$ kN/m^2
$K = 0.55$, $u/B = 0.184$, $v/L = 0.60$
So $m_1 = 0.125$ and $m_2 = 0.0306$
$M_B = 71.57$ kN-m and $M_L = 30.35$ kN-m
$u = 0.46m$ and $v = 2x = 2\times1.045 = 2.09m$
$K = 0.55$, $u/B = 0.184$ and $v/L = 0.464$
From Pieguad's curve, $m_1 = 0.1441$ and $m_2 = 0.043$
$M_B' = 64.30$ kN-m and $M_L' = 30.25$ kN-m
So $M_B = 3.635$ kN-m and $M_L = 0.05$ kN-m

Bending Moment due to Wheel Load at-6:
$u = 2[u_1+x] = 2[0.46+0.77] = 2.46$ m and $v = 2[v_1+x] = 2[0.31+1.045] = 2.71$ m
$K = 0.55$, $u/B = 0.984$ and $v/L = 0.601$
$m_1 = 0.068$, $m_2 = 0.02$
$[u_1+x][v_1+y] = 1.23\times1.355 = 1.67$
$m_1' = 0.113$ and $m_2' = 0.0334$
2 $u = 2x = 1.54m$ and $v = 2y = 2.09m$
$K=0.55$, $u/B = 0.616$ and $v/L = 0.464$
$m_1 = 0.097$, $m_2 = 0.038$
$xy = 0.804$

Hence $m_1' = 0.078$ and $m_2' = 0.0305$
$u = 2[u_1+x] = 2.46m$ and $v = 2y = 2.09m$
$K = 0.55$, $u/B = 0984$ and $v/L = 0.464$
$m_1 = 0.074$ and $m_2 = 0.03$
$y(u_1+x) = 1.285$, $m_1' = 0.095$ and $m_2' = 0.038$

$u = 2x = 1.54$m and $v = 2(v_1+y) = 2.71$m
$K = 0.55$, $u/B = 0.616$ and $v/L = 0.6012$
$m_1 = 0.092$ and $m_2 = 0.025$

Multiplying $x(v_1+y) = 1.043$ with above coefficients,
$m_1' = 0.096$ and $m_2' = 0.026$

So final coefficients are,
$m_1 = [(0.113+0.078)-(0.095+0.096)] = 0$
$m_2 = [(0.0334+0.0305)-(0.038+0.026)] \approx 0$

The resulting moment is given by,
$M_{B6} = M_{L6} = 0$ kN-m

Total bending moment is given by,
$M_B = 16.25+3.325+2.045+3.635+2.761 = 28.016$ kN-m
$M_L = 15.25+4.285+0.79+0.05+0.552 = 20.93$ kN-m
Applying continuity and impact,
$M_B = 28.016 \times 0.8 \times 1.18 = 26.45$ kN-m
$M_L = 20.93 \times 0.8 \times 1.18 = 19.76$ kN-m

The impact factor is taken as 18% as per clause-208.4 of IRC 6: 2014.[7] It can be seen that the moment along short span for tracked load is greater while the wheel load bending moment along the longer span is severer. Hence the moment for tracked load will be taken along shorter direction and moment along longer direction will be considered from wheel load in the design of deck slab.

Shear Force Calculation for Interior Slab Panel
Wheel Load Shear
Following IRC-112:2011 (B-3.3, Annex- B-3), the dispersion of load through wearing coat & slab will be at 45°.[8]
Hence dispersion of load is given by 0.85+2(0.08+0.25) =1.51m.

For maximum shear to occur, the load dispersion should be within face of girder.
So wheel load will be kept at least 1.51/2=0.755 m from the longitudinal girder face as shown in Figure 5.

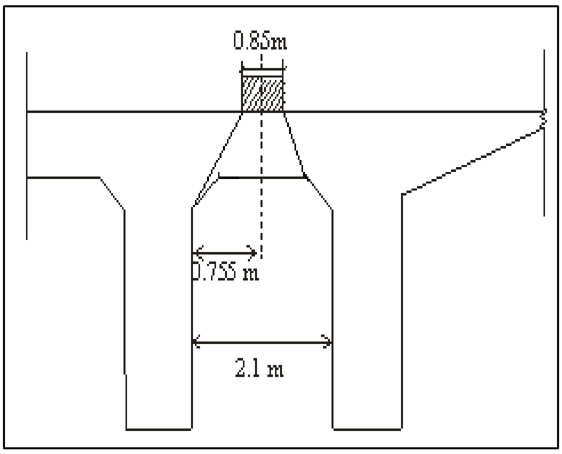

Fig.5. *Load Dispersion through Deck Slab.*

Referring IRC-112:2011, clause-B 3.2(page-278),
$b_{ef} \ =\alpha a(1-a/l_0)+b_1$
$b/l_0 \ =\frac{4.2075}{2.1} = 2.003 > 2.00$
$\alpha = 2.6$, $l_0 = 2.1$
$a = 0.755m$, $b_1 = 3.6+2\times0.08 = 3.76m$
$b_e = 2.6\times0.755(1-0.755/2.1)+3.76 = 5.017m$
Load per meter width is = $\frac{350}{5.017} = 69.763$
Maximum shear force = $\frac{69.763\times(2.1-0.755)}{2.1} = 44.681$ kN/m
Shear force with impact due to tracked vehicle = 49.15 kN/m

Dead Load Shear
Total dead weight= 8.46 kN/m^2
Total dead load shear=$\frac{8.46\times2.1}{2} = 8.883$ kN/m

Bending Moment and Shear Force
Total M_B = 3.81+28.58=32.39 kN-m, M_{l} = 1.824+19.76=21.584 kN-m
Total shear force=(8.883+49.15) kN/m=58.03 kN/m
Since we have considered limit state method the above load values will be 1.5 times that of calculated as per Anex A2 of IRC: 112-2011
So $M_B = 48.585 \approx 50$ kN-m
$M_l = 32.376 \approx 33$ kN-m
$V_u = 87.045$ kN/m

Minimum effective depth as per maximum bending moment is given by

$$d = \sqrt{\frac{50\times10^6}{(0.36\times35\times0.48\times10^3\times(1-0.416\times0.48))}} = 101.635 \text{ mm}$$

As per table – 14.2, clause –14.3.2.1 (page 142) of IRC–112: 2011,
Clear cover =40+8=48 mm
So, d = 250−48 =202 mm > 101.63 mm (Safe)
Using 16 mm-Ø bars, cover for short span = 210−8 =202mm and cover for long span =202−16=186 mm.

Determination of Area of Steel for Interior Slab Panel
Area of Steel along Short Direction

$$(A_{st})_B = \frac{0.5\times35}{415}\times\left[1-\sqrt{1-\frac{4.6\times50\times10^6}{35\times10^3\times202^2}}\right]\times10^3\times202$$

As per clause 16.6.1.1 of IRC: 112-2011

$$(A_{st})_{min} = 0.26\times\frac{f_{ctm}}{f_{yk}}\times b_t d$$

From Table 6.5, (Page-38) of IRC: 112-2011
F_{ctm}=2.8 N/mm^2, F_{yk}= 415 N/mm^2, B_t=1000 mm, D = 202 mm, $(A_{st})_{min}$=354.351 mm^2
Also same clause specified
$(A_{st})_{min} = 0.0013 b_t d = 262.6$mm^2
$(A_{st})_{max} = 0.025A_c = 0.025\times250\times1000 = 6250$mm^2

Spacing of bars as per clause-16.6.1.1(4) of IRC-2011 is,
$s_{max} < 2h = 2\times250 = 500$ mm
In our case, spacing of 16 mm Ø bars is
$S = \frac{1000\times\pi/4\times16^2}{716.008} = 280.811\text{mm} > 250$ mm

Also as per clause15.2.1 (2) of IRC: 112-2011,
The clear distance between the parallel main reinforcing bars should not be less than $d_g+10=20+10=30$ mm and 20 mm , whichever is greater
D_g= aggregate size = assumed 20mm for slabs, providing 16 mm-Ø bars @ 225mm c/c
$((A_{st})_B)_{provided} = \frac{1000\times\pi/4\times16^2}{225} = 893.61$ mm^2

Also as per clause -12.2.2, page – 120 of IRC: 112-2011, under rare combination of loads, the maximum tensile stress limits to 0.8 f_y is to avoid inelastic strain, undesirable cracking/deformation of structure and also to account for long term creep.[8]

We have calculated the steel area taking $f_{yd}=0.87f_{yk}$ (as per Cl-15.2.3.3 IRC: 112-2011)
Considering the worst case i.e rare combination of loads, additional steel area required $=1-\frac{0.8}{0.87} = 0.080 = 8\%$ of required steel area.
So 8% of $(A_{st})_{required} = \frac{8}{100}\times716.005 = 57.28$ mm^2
$A_{st}$to be provided=716.005+57.28=773.28mm^2<893.61mm^2 (Safe)
$(A_{st})_{provided}$=893.61 mm^2

Area of Steel along Long Direction
Providing 12mm-Ø bars,
$(A_{st})_L = \frac{0.5\times35}{415}\times\left[1-\sqrt{1-\frac{4.6\times33\times10^6}{35\times10^3\times190^2}}\right]\times10^3\times190 = 480.53$ mm^2
$(A_{st})_{min}=0.26\times\frac{f_{ctm}}{f_{yk}}\times b_f\times d$ (Cl-16.6.1, IRC:112-2011)
F_{ctm}= 2.8 N/m^2 (table-6.5 of IRC: 112-2011)
$(A_{st})_{min} = 0.26\times\frac{2.8}{415}\times1000\times190 = 333.301$ mm^2
Also as per same clause,
$(A_{st})_{min} = 0.0013b_td = 247$ mm^2
$(A_{st})_{max} = 0.025A_c = 0.025\times250\times1000 = 6250$ mm^2
Spacing of bars (as per Cl-16.6.1, IRC: 112-2011),
$S_{max} < 2h = 2\times250 = 500$ mm or 250mm (smaller value is taken)
$S = \frac{1000\times\pi/4\times12^2}{480.53} = 235.36$mm
Hence providing spacing of bars @175 mm c/c & using 12mm Ø bars
$((A_{st})_l)_{provided} = \frac{1000\times\pi/4\times12^2}{175} = 646.30$ mm^2
Also as per clause-15.2.1(2) of IRC: 112-2011, the clear distance between the parallel main reinforcing bars should not be less than $d_g+10=20+10=30$ mm or 20 mm (larger value is taken).

Also as per clause-12.2.2, P-120 of loads, the max tensile stress in steel is limited to $0.8f_{yk}$ to avoid inelastic stain, undesirable cracking/deformation of structure & also to account for long term creep.

Hence more area required is = $(1-\frac{0.8}{0.87})$ $(A_{st})_{required}$ =0.08×480.53=38.44 mm^2
$(A_{stl})_{required\ for\ creep\ cracking}$ =480.53+38.44=518.97 mm^2< 646.30 mm^2
$(A_{st})_{L.prov}$=646.30 mm^2

Shear Check in Interior Deck Slab Panel
V_u87.045 KN/m = $V_{Ed.}$
$V_{rdc} = [0.12k(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}]b_w d$ (Cl-10.3.2, IRC: 112-2011)
$V_{Rdc} = (V_{min}+0.15\sigma_{cp})b_w d$ (minimum)
$K=1+\sqrt{\frac{200}{d}} = 1+\sqrt{\frac{200}{250}}$=1.894 < 2.0 (Safe)
$V_{min}= 0.031K^{3/2}f_{ck}^{1/2} = 0.031\times 1.995^{3/2} \times 35^{0.5}$=0.478 and $\sigma_{cp} = 0$
$\rho_1 = \frac{A_{st}}{b_w d} \leq 0.02$
$A_{sl} = (A_{st})_B /2$ =893.61/2=446.80 mm^2
b_w =1000, d = 202
$\rho_1 = 2.21\times 10^{-3}$=0.00221 < 0.02 (Safe)
$V_{Rdc}=[0.12 \times 1.894 \times (80 \times 0.00221 \times 35)^{0.33}] \times 1000 \times 250$ =103.684 kN/m
$(V_{Rdc})_{min}$=119.50 kN/m
So minimum shear resistance isV_{Rdc}= 119.5 kN/m ≫ V_{Ed} = 87.045 kN/m
No shear reinforcement is necessary in slabs.

Design of Cantilever Slab

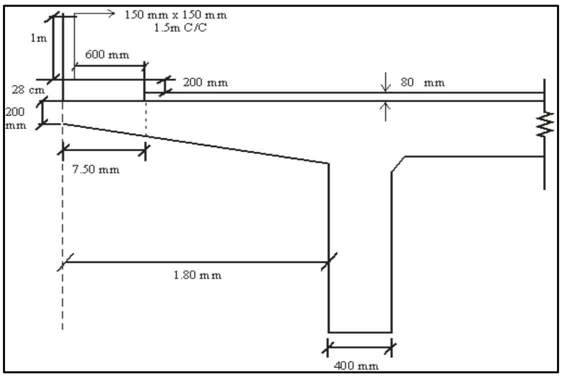

Fig. 6. *Cantilever Portion of Slab.*

Dead Load Moment (Figure 6)
Hand rail (lump sum) = 2 KN, Lever arm = 1.725m
M_{la} = 2×1.725 =3.45 kN-m
R.C.C post = $0.15\times 0.15 \times 1 \times 25$ =0.562 kN, Lever arm = 1.725m, M_{RP}= 0.97 kN-m
Kerb = $075\times 0.28 \times 1 \times 25$ =5.25 kN, Lever arm=1.425 m, M_{kerb}=7.48 kN-m
R.C.C slab =$0.3\times 1.8 \times 25$=13.5 kN
Lever arm=0.9 m, M_{RS}=12.15 kN-m
Wearing coat = $0.080\times 1.05 \times 22$=1.85 kN, Lever arm=0.525m, M_{wc}=25.02 kN-m
Total dead load moment M_{dc}=25.02 kN-m

Live Load on Kerb
L.L =400 kg/m^2=4 kN/m^2
Lateral load due to live load is =750 kg/m=7.5 kN/m

L.L=4×0.6=2.4 kN/m
Lateral L.L =7.5 kN/m
L.L moment =2.4×1.35= 3.24 kN-m
Lateral L.L moment=7.5× 0.58 = 4.35 KN-m (Cl-209 of IRC: 6-2000)[9]

Moment Due to Wheel Load
As per IRC-6:2010,[10] only IRC-class A & IRC-class-B
Vehicles can come to the cantilever portion, since it can have a minimum distance 150 mm. (Figure 7).

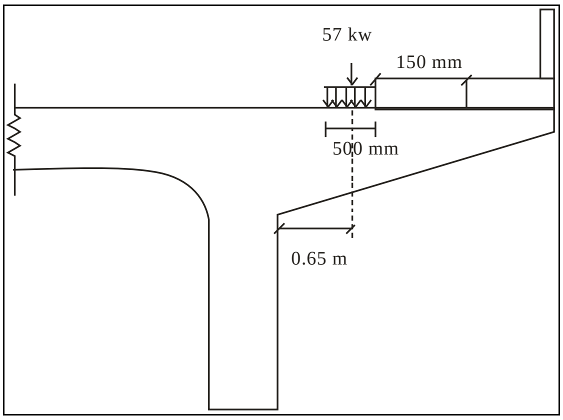

Fig.7. *Wheel Load on Cantilever Slab.*

Using IRC-112:2011, Annexure: B-3(2), Page-279,
Effective depth (B_{ef}) = 1.2a+b_1
a=0.65 m
b_1=0.25 + 2 × 0.080 =0.41 m
B_{ef}=1.2× 0.65 + 0.41 =1.19 m

Live load per meter width including impact = (0.57× 1.5)/1.19=71.85 kN-m
Wheel load moment (M_{wc}) = 71.85× 0.65 = 46.70 KN-m

$d_{required} = \sqrt{\frac{119\times10^6}{0.36\times35\times0.48\times10^3\times(1-0.416\times0.48)}}$ =156.80 mm

Providing 40 mm clear cover &16 mm Ø bars, Effective depth provided is
$d_{provided}$ =400-(40+8)=352 mm ≫156.80 (Safe)

Reinforcement in Cantilever Slab
Main reinforcement is given by

$(A_{st})_{main} = \frac{0.5\times35}{415}\left[1-\sqrt{1-\frac{4.6\times119\times10^6}{35\times10^3\times352^2}}\right]\times10^3\times352$ =968.4056 mm^2

Spacing of 16 mm-Ø bars is given by
S =1000× $\frac{\pi}{4}\times16^2$ =207.6 mm
Providing 16 mm-Ø bars @ 190 mm c/c,
$(A_{st})_{main}$ = 1058.22 mm^2 (Fig.8)
Distribution moment is given by
M_{dc} = 1.5× [0.3 × 56.043 + 0.2 × 25.02] =31.941 kN-m
Distribution reinforcement is
$(A_{st})_{dc}$ = 349.2 mm^2; Providing 12mm-Ø bars @ 175 mm c/c

$[(A_{st})_{dc}]_{provided}$ =646.27 mm^2> 349.2 mm^2 (Safe)

Check for Shear in Cantilever Portion

Total shear=dead load shear + live load shear = $23.162+1.5\times \frac{5.7}{1.19} + 2.4 = 97.411$
(where1.19= b_{ef} (Annex-B3, IRC:112-2011))
Design shear = 1.5× 97.41= 146.115 kN=V_{Ed}
As per clause-10.3.2(2) of IRC: 112-2011,
Shear resistance of a structure is given by
$V_{Rdc} = [0.12k(80\rho_1 f_{ck})^{1/3} + (0.15\sigma_{cp})]b_w d$
Subject to min $V_{Rdc}= (V_{min}+0.15\sigma_{cp})b_w d$
$K=1+\sqrt{\frac{200}{d}} =1+\sqrt{\frac{200}{400}} = 1.7< 2.0$ (Safe)
$V_{min}=0.031k^{3/2}f_{ck}^{1/2} =0.031\times 1.71^{3/2} \times 35^{1/2} =0.41$
$\rho_1 =\frac{Asl}{b_w d} =\frac{1058.22}{1000\times400} = 2.645\times 10^{-3}$
$V_{Rdc}=[0.12\times 1.71 \times (80 \times 2.645 \times 10^{-3} \times 35)^{1/3}]\times 10^3 \times 400$ =158.93 kN
$(V_{Rdc})_{min}=0.41\times 1000 \times 400$ =164 kN
$\therefore V_{Rdc}$ =164 kN $>V_{Ed}$ =146.15 kN (Safe)
Also IRC: 112-2011,Cl-10.3.2(5) specified the following criteria.

$V_{ed} \leq 0.5b_w dvf_{cd}$

$V=0.6[1-\frac{f_{ck}}{310}] = 0.6[1-\frac{35}{310}] = 0.532$
So $0.5b_w dvf_{cd}= 0.5\times 1000 \times 400 \times 0.532 \times 0.36 \times 35$ =1340.64 kN > V_{Ed} (Safe)

The live load applied is 0.65 m from the edge of support. The same clause specifies that is the applied load is at a_v i.e 0.5d to 2d (200mm to 800mm),then there will be reduction factor multiplied to V_{ed}. So in our case the dead load shear will be 23.162 kN.
But live load shear will be $(1.5\times \frac{57}{1.19}) \times \beta$
β = reduction factor=$a_v/2d$
a_v= 650 mm, $\beta =\frac{650}{2\times400}=0.8125$
$V_{ls}=1.5\times \frac{57}{1.19} \times 0.8125$ =58.377kN
Total shear=84 kN
V_{ed}=126 KN, V_{Rdc}=164 kN >126 kN (Safe)
It may be noted that the downward wind force will be only 5×1.8=9 kN against live load 86.22 kN.
Hence, there is no need of combination of loads taking wind effect into account.

Deflection Check for Cantilever Slab

The deflection will be checked as per IS: 456:2000, since different loading at different positions are accorded, hence separate calculations are necessary.

First Trail

(Annex-C of IS: 456-2000)**:**

Short Term deflection

$(I_{gr})_{end}=\frac{b\times d^3}{12} = \frac{1000\times200^3}{12} = 6.67 \times 10^8 mm^4$

$(I_{gr})_{mid} = 22.5 \times 10^8 mm^4$, $(Igr)at\ 0.65m = 29.35 \times 10^8 mm^4$

$F_{cr} = 0.7\sqrt{f_{ck}} = 4.141\ N/mm^2$

$(M_r)_{end} = \frac{4.141 \times 6.67 \times 10^8}{100} = 27.6kN - m$

$(M_r)_{mid} = \frac{4.141 \times 22.5 \times 10^8}{150} = 62.115kN - m$

$(M_r)_{0.65m} = 74.158$ kN-m

$E_c = 5000\sqrt{f_{ck}} = 2.958 \times 10^4\ N/mm^2$

$E_s = 2 \times 10^5 \frac{N}{mm^2}, m = \frac{Es}{Ec} = 6.76$

Transformed area of compression steel = $(m-1) \times A_{sc} = 3427.93mm^2$

Transformed area of tension steel=$m \times A_{st} = 7152.08mm^2$

Let "x" be the depth of neutral axis

<u>At end:</u>

Or $1000 \times x \times \frac{x}{2} + 3427.93 \times (x - 46) = 7152.08 \times (152 - x)$

Or $500x^2 + 10580.01x - 1459363.34 = 0$, Or x=44.47mm

<u>At mid</u>:

$500x^2 + 10580.01x - 1960008.94 = 0$

X=52.92mm

<u>At 0.65m</u>:

$500x^2 + 10580.01x - 2158693.7 = 0$, X=55.97mm

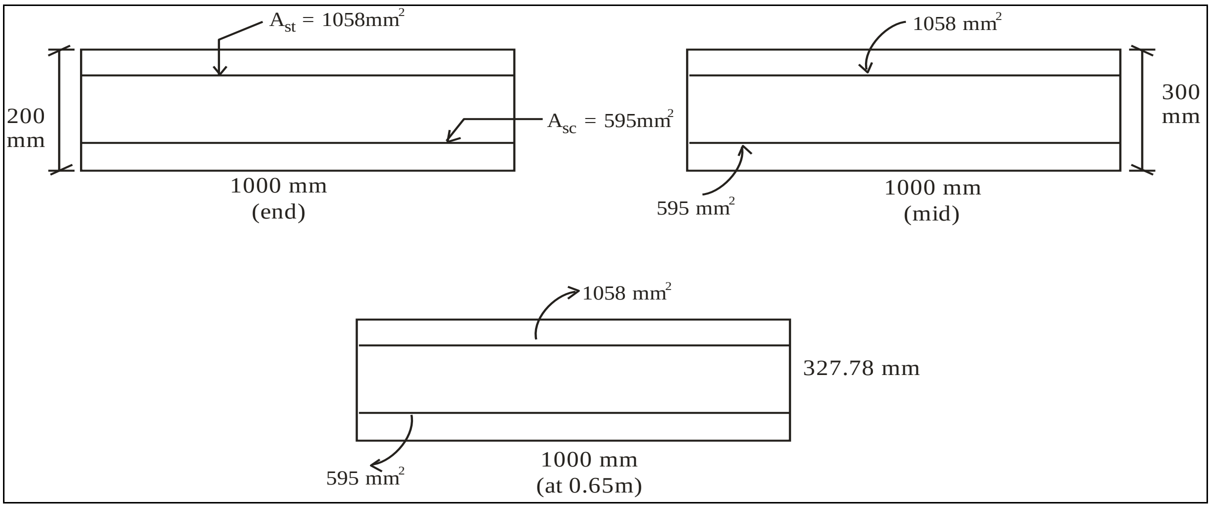

Fig.8. *Area of Reinforcement in Different Section of Cantilever Slab (SP-34).*

$(I_r)_{end} = \frac{1}{3} \times 1000 \times (44.47)^3 + 3427.93 \times (44.47 - 46)^2 + 7152.08 \times (152 - 44.47)^2$

$= 1.117 \times 10^8 mm^4$

$(I_r)_{mid} = 3.1264 \times 10^8 mm^4$

$(I_r)_{at\ 0.65m} = 5.58 \times 10^8 mm^4$

z= lever arm i_{end}=0.9d=136mm, z_{mid}=226mm, $z_{at\ 0.65m}$=251mm

$(M)_{end}$=kerb + port & railing + lode to pedestrian=7.48+3.45+0.97+4.35+3.24=19.49kN-m

(M_{mid})=w.c+R.C slab=13.12kN-m

$M_{0.65m}$=wheel load=46.70kN-m

$(I_{eff})_{end} = \frac{(i_r)_{end}}{1.2 - \frac{(mr)_{end}}{(m)_{end}} \times \frac{z_{end}}{d_{end}} \times \left(1 - \frac{x_{end}}{d_{end}}\right) \times (\frac{b_w}{b})} = \frac{1.117 \times 10^8}{1.2 - \frac{27.6}{19.49} \times \frac{136}{152}(1 - \frac{44.47}{152})} = 3.67 \times 10^8 mm^4$

$(I_r)_{end} < (I_{eff})_{end} < (I_{gr})_{end}$(Safe)

$(\mathrm{I_{eff}})_{\mathrm{mid}} = (-)1.455 \times 10^8 > (\mathrm{I_r})_{\mathrm{mid}}$
So $(\mathrm{I_{eff}})_{\mathrm{mid}} = (\mathrm{I_r})_{\mathrm{mid}} = 3.1264 \times 10^8 \mathrm{mm}^4$
$(\mathrm{I_{eff}})_{\text{at 0.5 m}} = 88.84 \times 10^8 \mathrm{mm}^4 > (\mathrm{I_{gr}})_{\text{at 0.65m}}$
So $(\mathrm{I_{eff}})_{\text{at 0.65 m}} = 29.35 \times 10^8 \mathrm{mm}^4$
$(\delta_1)_{\mathrm{end}} = \frac{wl^3}{3E_cI_r} = \frac{10212\times(1800)^3}{3\times(2.958\times10^4)\times(3.67\times10^8)} = 1.83\mathrm{mm}$
$(\delta_1)_{\mathrm{mid}} = \frac{15350\times(1800)^3}{8\times(2.958\times10^4)\times(3.1264\times10^8)} = 1.21\mathrm{mm}$
$(\delta_1)_{\text{At 0.65m}} = \frac{1}{E_cI_r}\left[\frac{1.15\times w\times l^2}{2} + \frac{wl^3}{3}\right] = 0.64\mathrm{mm}$
Total deflection due to short tern loading =3.68mm

Deflection Due to Shrinkage
$a_{cs} = k_3\Psi_{cs}l^2$
$k_3 = 0.5$ (for cantilever)
$p_t = 0.7\%$, $p_c = 0.4\%$
$k_4 = 0.72\frac{p_t - p_c}{\sqrt{p_t}} = 0.26 < 1.0$
$0.25 \leq p_t - p_c = 0.3 < 1.0$ (Safe)
$(\Psi_{cs})_{\mathrm{end}} = 3.9 \times 10^{-7}$
$a_{cs} = 0.63$mm
At other points 'Ψ_{cs}' will give much lesser value relating least shrinkage deflection.

Deflection Due to Creep
$E_{ce} = \frac{E_c}{1+\theta}$
$\theta = 1.6$ (for 28-days strength)
$E_{ce} = 1.1377 \times 10^4 \mathrm{N/mm^2}$
$m = E_s/E_c = 17.58$
Transformed area for compression steel $(m-1)A_{sc} = 9864.7\ \mathrm{mm}^2$
Transformed area for tension steel $mA_{st} = 18598.93\ \mathrm{mm}^2$
Let x-be the depth of neutral axis.

At End:
$1000\times x\times\frac{x}{2} + 9864.7\times(x-46) = 18598.93\times(152-x)$
Or $500x^2 + 28463.63x - 3280813.56 = 0$
Or x = 57.40 mm

At Mid:
$500x^2 + 28463.63x - 5140706.56 = 0$
Or x = 76.853mm

At 0.65 m:
$500x^2 + 28463.63x - 5657384.835 = 0$
Or x = 81.65 mm
Same lever arm as used before will be used.
So $I_{r.end} = \frac{1}{3}\times1000\times(57.4)^3 + 9864.70(57.4-46)^2 + 18598.93(152-57.4)^2 = 2.30\times10^8\ \mathrm{mm}^4$
Similarly $I_{r.mid} = 7.3\times10^8\ \mathrm{mm}^4$ and $I_{r\text{ at }0.65\mathrm{m}} = 9.22\times10^8\ \mathrm{mm}^4$

$$(I_{r.eff})_{end} = \frac{I_{r.end}}{1.2-(\frac{M_r}{M})_{end}\times(\frac{z}{d})_{end}\times(1-\frac{x}{d})_{end}\times\frac{b_w}{b}} = 5.59\times10^8 mm^4$$

So

$I_{r.end} < I_{eff.end} < I_{gr.end}$ (Safe)

Similarly $(I_{eff})_{mid} = -4.17\times10^8 mm^4 < (I_r)_{mid}$

$(I_{eff})_{mid} = 7.3\times10^8 mm^4$

$(I_{eff})_{at\ 0.65m} = 29.35\times10^8 mm^4$

But in this case, for calculating permanent creep using above equations & E_{ce}, only permanent load is taken care

So $(W)_{end}$=kerb +R.C pol & railing=5.25+2.562=7.812 kN/m

$(W)_{mid}$ =W.C+R.C slab=1.85+13.5=15.35 kN/m

$(a_{i.cc})_{perm} = \frac{(W_{end})\times l^3}{3E_{ce}\times I_{eff}} + \frac{(W_{mid})\times l^3}{8E_{ce}\times I_{eff}}$

=(2.39+1.35)mm=3.74mm

Short term deflection due to permanent load

$a_{i.perm} = \frac{\mathbf{7.812\times10^3\times1800^3}}{\mathbf{3\times2.958\times10^4\times3.67\times10^8}} + \frac{\mathbf{15.35\times10^3\times1800^3}}{\mathbf{8\times2.958\times10^4\times3.1264\times10^8}}$=1.4+1.21=2.61mm

So deflection due to creep is given by

3.74-2.61=1.13mm

Total deflection is =3.68+0.63+1.13=5.44mm

However this deflection will be lesser in practical as more accurate calculations will reveal the result

As per clause 12.4.1 of IRC:112-2011, the deflection should be limited to = $\frac{\text{cantilever span}}{375}$

$=\frac{1800}{375} = 4.8\text{mm} < 5.44\text{mm}$

Verifying as per Cl-23.2 of IS: 456-2000

f_s=220.27, p_t=0.7%

Modification factor=1.2

p_c=0.4%

Modification factor=1.12

Basic $\frac{\text{span}}{\text{depth}} = 7$(for cantilever), Modified $\frac{\text{span}}{\text{depth}}$=7×1.12×1.2=9.408

Our $\frac{\text{span}}{\text{depth}}$= 6<9.408 (Safe)

But revising the section as providing 16 ϕ bars 225mm c/c & 12mm ϕ bars@225mm alternatively and rechecking, the defection criteria is satisfied. Hence total reinforcement provided is given by 1396 mm^2.

Second Trail- Rechecking of deflection for Cantilever Slab (Annex C of IS: 456-2000)

Short Term Deflection

$(I_{gr})_{end} = \frac{b\times d^3}{12} = \frac{1000\times200^3}{12} = 6.67\times10^8 mm^4$

$(I_{gr})_{mid} = 22.5\times10^8 mm^4, (Igr).65m = 29.35\times10^8 mm^4$

$F_{cr} = 0.7\sqrt{f_{ck}} = 4.141 n/mm^2$

$(M_r)_{end} = \frac{4.141\times6.67\times10^8}{100} = 27.6 kN-m$

$(M_r)_{mid} = \frac{4.141\times22.5\times10^8}{150} = 62.115 kN-m$

$(M_r)_{0.65m}$=74.158 kN-m

$E_c = 5000\sqrt{f_{ck}} = 2.958\times10^4 N/mm^2$

$E_s = 2\times10^5 \frac{n}{mm^2}, m = \frac{Es}{Ec} = 6.76$

Transformed area of compression steel = (m-1)×A_{sc} =3720.96mm^2
Transformed area of tension steel= m×A_{st} =9436.96mm^2
Let “x” be the depth of neutral axis

At End:
$=1000 \times x \times \frac{x}{2} + 3720.96 \times (x - 46) = 9436.96 \times (152 - x)$
$=500x^2 + 13157.92x - 1605582.08 = 0$
=x=45.02mm

At Mid:
$=500x^2 + 13157.92x - 2549278.08 = 0$
X=59.45mm

At 0.65m:
$=500x^2 + 13157.92x - 2811436.83 = 0$
X=62.97mm
$(I_r)_{end}=\frac{1}{3} \times 1000 \times (45.02)^3 + 3720.96 \times (45.02 - 46)^2 + 9436.96 \times (152 - 45.02)^2$
$=1.38 \times 10^8 mm^4$
$(I_r)_{mid} = 4.2 \times 10^8 mm^4$
$(I_r)_{at\ 0.65m} = 5.27 \times 10^8 mm^4$
z= lever arm. z_{end}=0.9d=136mm,z_{mid}=226mm,$z_{at\ 0.65m}$=251mm
$(M)_{end}$=kerb + port & railing +load due to pedestrian=7.48+3.45+0.97+4.35+3.24=19.49kN-m
(M_{mid})=Wearing Coat + R.C slab=13.12kN-m
$M_{0.65m}$=wheel load=46.70kN-m
$(I_{eff})_{end}=\frac{(i_r)_{end}}{1.2-\frac{(m_r)_{end}}{(m)_{end}}\times\frac{z_{end}}{d_{end}}\times\left(1-\frac{x_{end}}{d_{end}}\right)}=4.5 \times 10^8 mm^4$
$(I_r)_{end} < (I_{eff})_{end} < (I_{gr})_{end}$(ok)
$(I_{eff})_{mid} = (-)2.05 \times 10^8 mm^4$; should be greater than $(I_r)_{mid} = 4.2 \times 10^8 mm^4$
$(I_{eff})_{at\ 0.65\ m} = 54.44 \times 10^8 mm^4 < (I_{gr})_{at\ 0.65m} = 29.35 \times 10^8 mm^4$
$(\delta_1)_{end}=\frac{wl^3}{3E_cI_r} = 1.5mm$, $(\delta_1)_{mid}$ =0.9mm, $(\delta_1)_{At\ 0.65m} = \frac{1}{E_cI_r}\left[\frac{1.15 \times w \times l^2}{2} + \frac{wl^3}{3}\right] = 0.64mm$
Total deflection due to short tern loading =3.04mm

Deflection Due to Shrinkage
$a_{cs}=k_3\Psi_{cs}l^2$
k_3=0.5(for cantilever)
p_t=0.92% ,p_c=0.425%
k_4= 0.37 < 1.0, $0.25 \leq p_t - p_c = 0.495 < 1.0$ (Safe)
$(\Psi_{cs})_{end} = 5.55 \times 10^{-7}$, a_{cs}= 0.9mm

Deflection Due to Creep
$E_{ce} = \frac{E_c}{1+\theta}$
θ=1.6(for 28-days strength), $E_{ce} = 1.1377 \times 10^4 \frac{N}{mm^2}$, $m=\frac{E_s}{E_{ce}} = 17.58$
Transformed area of compression steel = (m-1)×A_{sc} =10710.68mm^2
Transformed area of tension steel=m×A_{st} =24541.68 mm^2

Depth of Neutral Axis at Different Section

At End

x=63.2mm

At Mid

x=85.566mm

At 0.65m

x=91.08mm

$(I_r)_{end} = 2.80 \times 10^8 mm^4$, $(I_r)_{mid} = 9.03 \times 10^8 mm^4$

$(I_r)_{0.65m} = 2.80 \times 10^8 mm^4$

$(I_{eff})_{mid} = -5.63 \times 10^8 mm^4 < (I_r)_{mid}$

$(I_{eff})_{end} = 6.08 \times 10^8 mm^4$

$(I_{eff})_{at\ 0.65m} = 47.80 \times 10^8 mm^4$; which should not be greater than I_{gr} at 0.65m

$(W)_{end} = 7.812$ kN/m, $(W)_{mid} = 15.35$ kN/m

$(a_{i.cc})_{perm} = \frac{(W_{end}) \times l^3}{3E_{ce} \times I_{eff}} + \frac{(W_{mid}) \times l^3}{8E_{ce} \times I_{eff}}$

= (2.2+1.1) mm = 3.3mm

Short term deflection due to permanent load

$a_{i.perm} = \frac{7.812 \times 10^3 \times 1800^3}{3 \times 2.958 \times 10^4 \times 3.67 \times 10^8} + \frac{15.35 \times 10^3 \times 1800^3}{8 \times 2.958 \times 10^4 \times 3.1264 \times 10^8} = 1.4 + 1.21 = 2.61$mm

Deflection due to creep = 0.69mm ≈ 0.70mm

Hence total deflection = 3.04 + 0.9+0.7 = 4.64 mm < 4.8 mm (Safe)

CONCLUSION

The proposed bridge has two piers against four piers of the existing bridge in order to allow more waterways. Also due to reduction in number of piers, piles and pile caps, better economy is achieved in terms of approximate 30% material reduction when the number of piers reduces by 50%. Under-reamed pile foundation is provided keeping the soil strata in mind as well as from reference of National Highway Division. The analysis made by Pigeaud’s method is user friendly and has maximum simplicity in adopting for the design purpose. The entire structure is found to be stable against sliding and overturning, besides, it has been observed that provision of long span decreases the obstruction by increasing the water way. The uplifting action of the deck slab due to wind action has been counter balanced by the gravity load as assigned.

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