Original Research Paper

Engineering

ANALYSIS AND DESIGN OF A BRIDGE I-SECTION BEAM

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ABSTRACT Composite bridge is a structure with a combination two different materials reinforced concrete and Steel. In the composite bridge is designed by T beams are 2m I sections. on the I sections lateral girders are placed and on that RCC slab is laid. The shear connectors are connected between I sections and the RCC slab. These shear connectors will act as a mediator and control the shear between them. In this we considered IRC class AA tracked loading. The review of a design is given below. We follow the code book of IRC 6-2000 for Road loading standards.

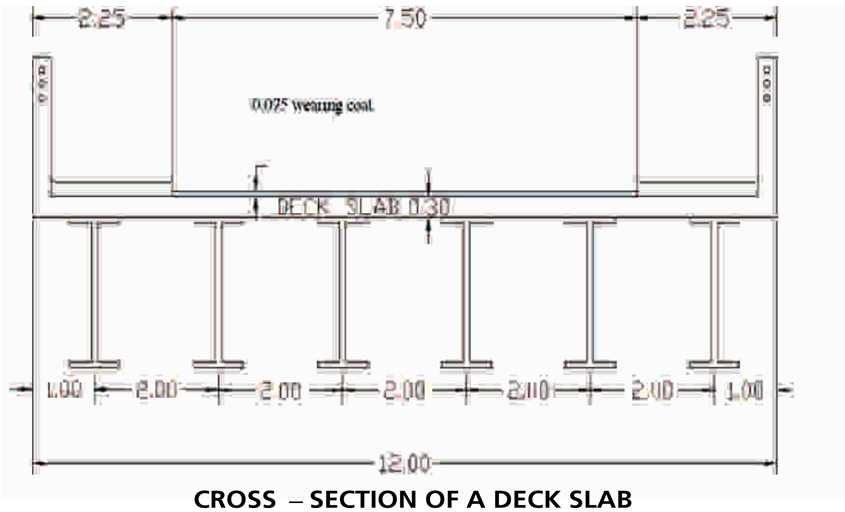
KEYWORDS :

Design of a composite bridge deck with the reinforced concrete slab and steel plate girder to cover a span of 34.5m

Clear width of road way=7.5m   Foot path: 2.25 m on either side

Spacing of main girders = 2 m   Materials: M_{25} grade concrete and Fe415steel

1. Design of deck slab



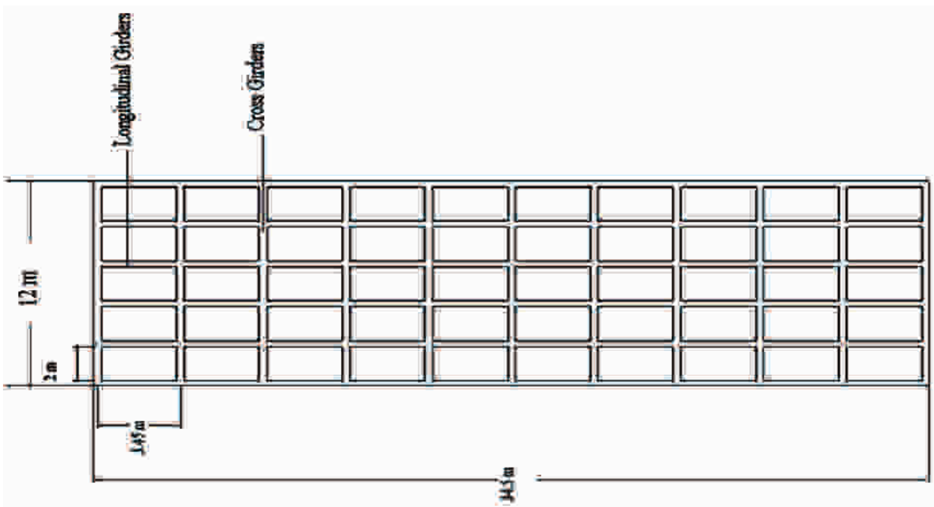
CROSS – SECTION OF A DECK SLAB

PLAN OF STEEL GIRDERS

2: Design of deck slab

Number of main girders (Longitudinal girders) = 6 numbers in longitudinal direction c/c distance is 2 m. as shown in figure below

Number of cross girders (Perpendicular to main girders) = 11 number of @ 34.5 m c/c as shown in figure below.



3: Design of deck slab

Panel dimension =2 m by 3.45 m,Dead weight of slab = 0.3 ×24 = 7.2 KN/m^2
Dead weight of wearing coat =0.075×22 = 1.65 KN/m^2
Total dead load = 8.85 KN/m^2

4: Live load bending moment

Live load is I.R.C class AA tracked vehicle.

u = 0.85 + 2×0.075 = 1 m, v = 3.6 + 2×0.075= 3.75 m

$\frac{u}{B} = \frac{1}{2} = 0.5$, $\frac{v}{l} = \frac{3.75}{3.45} = 1.07$, $K = \frac{B}{L} = \frac{2}{3.45} = 0.571$

From pie guard's curve for (K=0.6)

Read out the values of $m_1 = 0.078 m_2 = 0.024$

7: Design of steel plate girder

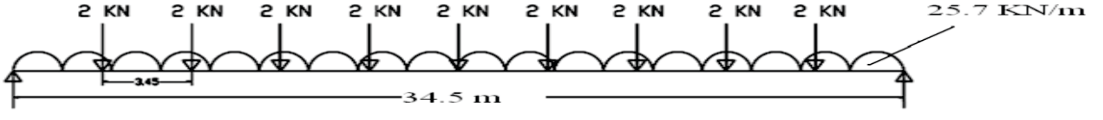
Spacing of main girders = 2 m, Spacing of cross girders = 3.45 m

Dead load on girder =8.85×2 = 17.7 KN/m

Self weight of main girder = (0.2L+ 1) = ((0.2×34.5)+1) = 7.9 KN/m = 8 KN/m

Total load = 17.7 +8 = 25.7 KN/m

Self weight of cross girders assumed as 1 KN/m =2 ×1 = 2 KN/m



Dead loads on plate girder

8: Dead load moments

Sum of vertical loads $v_a + v_b$ = (25.7×34.5) + (9×2) = 904.65 KN

Taking moment about A

v_b×34.5=(25.7×34.5×$\frac{34.5}{2}$) 2(3.45+6.9+10.35+13.8+17.25+20.7+24.15+27.6+31.05)

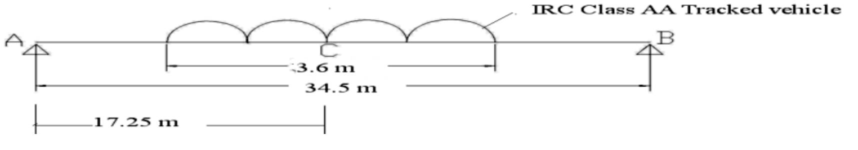
$v_b = \frac{15605.21}{34.5}$ = 452.32 KN, V_a= 904.65 – 452.32 = 452.32 KN

Maximum bending moment at center is

= (452.32 ×$\frac{34.5}{2}$) – (25.7×$\frac{34.5}{2} \times c$) – 2(3.45+6.9+10.35+13.8) = 3909 KN-m

Therefore maximum dead load bending moment is 3909 KN-m

9: Live load moments



Live load onplate girder

$v_a = v_b = \frac{350}{2}$= 350 KN

Maximum bending moment at center= (17.5×$\frac{34.5}{2}$) – (350×0.9×$\frac{1}{2}$) = 2861.05 KN-m

Impact factor = 10%

Live load bending moment = (2861.05×1.1) = 3147.155 KN-m

Dead load bending moment = 3147.155 + 3909 =7056.155 KN-m

10: Shear forces

Dead load shear = 452.32 KN

Live load shear with impact factor = 175 × 1.1 = 192.5 KN

Total design shear V = 452.32 +192.5 = 644.82 KN

11: Proportioning of trial section of web plate

Approximate depth of girder = $\frac{1}{8} = \frac{1}{10}$span = $\frac{34.5}{8}$to $\frac{34.5}{10}$= 4.3125 to 3.45 m

Therefore depth of girder = 3.45 m =3450 mm

Economical depth of girder = $5\sqrt[3]{\frac{M}{\sigma_b}} = 5\sqrt[3]{\frac{7056.155\times10^6}{165}}$= 1748.49 mm

Web depth based on shear considerations assuming 20 mm thick plate is

d = $\frac{V}{\tau\times20} = \frac{644.82\times10^3}{100\times20}$= 322.41 mm, Try web plate of size 20 mm × 2000 mm

12: Flange plates

Approximate flange area required $A_f = \left(\frac{M}{\sigma_b\times d} - \frac{A_W}{6}\right) = \left(\frac{7056.155\times10^6}{165\times2000} - \frac{20\times2000}{6}\right)$ = 14715.62 mm^2

Flange width B = $\frac{L}{45}$to $\frac{L}{40} = \frac{34500}{45}$to $\frac{34500}{40}$= 766.67 mm to 862.5 mm

Therefore flange width = $\frac{766.67+862.5}{2}$ = 814.585 mm

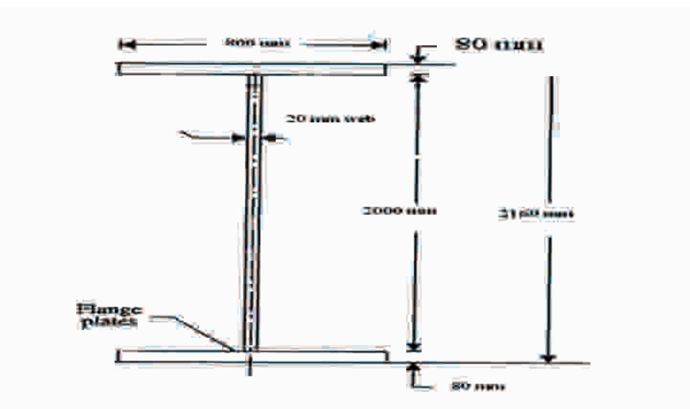
Adopt flange width = 800 mm

Thickness of plate = $\frac{A_f}{b_f} = \frac{14715.62}{800}$= 18.39 mm

Adopt flange thickness = 80 mm

Therefore size of flange plate is 800 mm ×80 mm

13: Checkformaximumstresses



Cross-section of plate girder

Moment of inertia of section I =$\frac{2o\times1000^3}{12}$+ 2(800×80)×540^2= 3.8991 ×10^{10} mm^4

Bending tensile stress = $\sigma_b = \frac{M\times y}{I} = \frac{7056.155\times10^6\times580}{3.8991\times10^{10}}$= 50.23 N/mm^2 < 165 N/mm^2

Average shear stress = $\frac{644.82\times10^3}{500\times20}$= 64.482 N/mm^2

Permissible average shear stress depends upon the ratio of $\left(\frac{d}{t}\right) = \frac{1000}{20}$= 50

Using stiffener spacing c= 1000 mm = d

From code IS: 226and IRC: 24- 1967 Allowable average shear stress is 87 N/ mm^2

64.482 N/ mm^2 < 87 N/ mm^2, Hence average shear stress is within safe permissible limits.

14: Connection between flange and web

Maximum shear at the junction of web and flange is given by $\tau = \frac{Va\bar{y}}{I}$

Where a = Area of flange = 800 × 80

$\tau = \frac{644.82\times10^3\times(800\times80)\times540}{3.8991\times10^{10}}$ = 571.541 N/ mm

Assuming continuous weld on either side, strength of weld of size‘s’ is

= (2×0.7 s×102.5) = 143.5 s

143.5 s = 483

$S = \frac{483}{143.5}$ = 3.36 mm Use 6 mm fillet weld, continuous on either side.

15: Intermediate stiffeners

Since the ratio of $\left(\frac{d}{t}\right) = \frac{1000}{20}$ **=** 50<85

Therefore vertical stiffeners are required.

Spacing of stiffeners = 0.33d to 1.5d = (0.33×1000) to (1.5×1000) = 330 to 1500 mm

Adopt 900 mm spacing, Hence space c = 900 mm

The intermediate stiffeners are designed to have a minimum moment of inertia of

$I = \frac{1.5\times d^3\times t^3}{c^2} = \frac{1.5\times1000^3\times20^3}{900^2}$ = 29.629 ×10^6 mm^4

Using 20 mm thick plate

Maximum width of plate not to exceed 12t for flats.

Use a plate 20 mm×180 mm

h = 180 mm (not greater than 240 mm)

$I = \frac{20\times180^3}{3}$ = 38.88×10^6 mm^4

But minimum moment of inertia = $I = \frac{1.5\times d^3\times t^3}{c^2} = \frac{1.5\times2000^3\times20^3}{1800^2}$ = 14.814814 ×10^6 mm^4

38.88×10^{6}mm^4 > 14.814814 ×10^6 mm^4, Hence selected plate is ok.

16: connection of vertical stiffener to web

Shear on welds connecting stiffener to web= $\frac{125\times t^2}{h} = \frac{125\times20^2}{180}$ = 277.87 KN (where‘t is the thickness of web and ‘h’ is out stand of stiffener in mm)

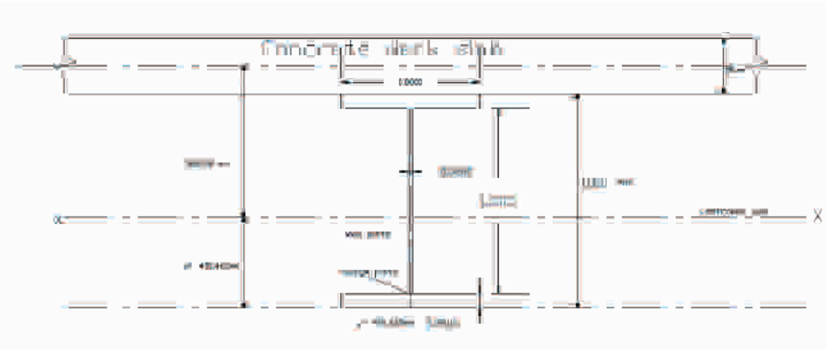
Size of weld = 0.7S×102.5 = 277.78, $S = \frac{277.78}{0.7\times102.5}$ = 3.87mm

Use 6 mm fillet weld, continuous on either side.

Effective length of weld not less than 10t = 10×20 = 200 mm

Use 200 mm long, 6 mm fillet welds alternately on either side.

19: Properties of composite section



Properties of composite section

$A_{ce} = \frac{(1000\times300)}{13}$ = 23076.923 mm^{2}Modular ratio m = 13

The centroid of the composite section is determined by first moment of the areas about the axis XX.

$A\bar{y}$ = [(23076.923×1310) + (800×80×620) + (1000×20×580) + (800×80×40) = 84.0707×10^6 mm^3

Area of composite section A = (23076.923) + (2×800×80) + (1000×20) = 171076.923 mm^2

Therefore $\bar{y} = \frac{A\bar{y}}{A} = \frac{84.0707\times10^6}{171076.923}$ = 491.420 mm

$I_{comp} = (23076.923\times818.58^2) + \frac{(800\times1160^3)}{12} - \frac{(780\times1000^3)}{12} + (168000\times1345.06^2) = 3.58413\times10^{11}$ mm^4

Maximum shear force at junction of reinforced concrete slab and girder is given by $\tau = \frac{Va\bar{y}}{I}$

Where V = 644.82 KN

a = 46154 mm^2

I = 2.139×10^{11} mm^4

$\bar{y}$ = 1345.06 mm

$\tau = \frac{(644.82\times10^3\times46154\times964.94)}{2.139\times10^{11}}$ =134.25 N/mm

Total shear force at junction = 134.25×800 = 107405.706 N

Using 20 mm diameter mild steel studs, capacity of one shear connector is given by

8Where d is the diameter of mild steel studs = 20 mm

$Q = 196\, d^2\sqrt{f_{ck}} = (196)\, 20^2\sqrt{25}$ = 392000 N

Number of studs in one row = $\frac{107405.706}{392000}$= 0.27 < 1

Provide a minimum of 2 mild steel studs in a row.

20: Pitch of shear connectors

Pitch of shear connectors = $p = \frac{NQ}{F\tau}$

Where N = number of shear connectors in a row = 2

Q = capacity of one shear connector = 392000 N

τ = horizontal shear per unit length =134.25 N/mm

F = Factor of safety = 2

$p = \frac{2\times392000}{2\times134.25}$= 2919.92 mm

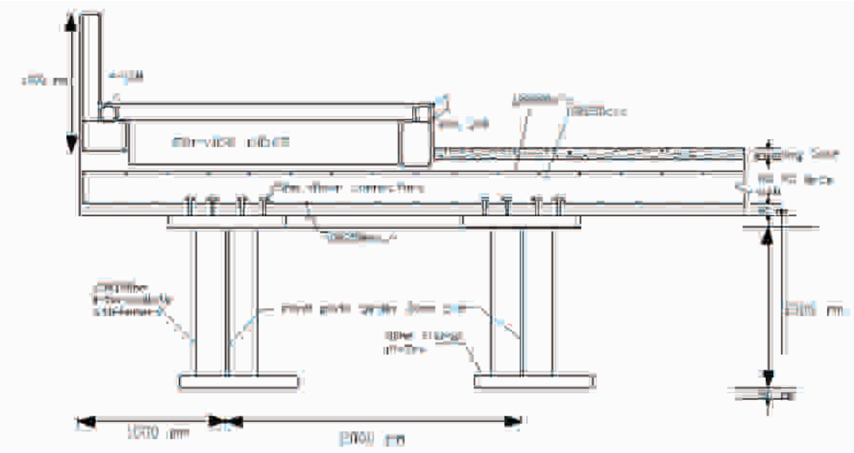
Maximum permissible pitch is the least of

(i) Three times the thickness of slab = (3×300) = 900 mm

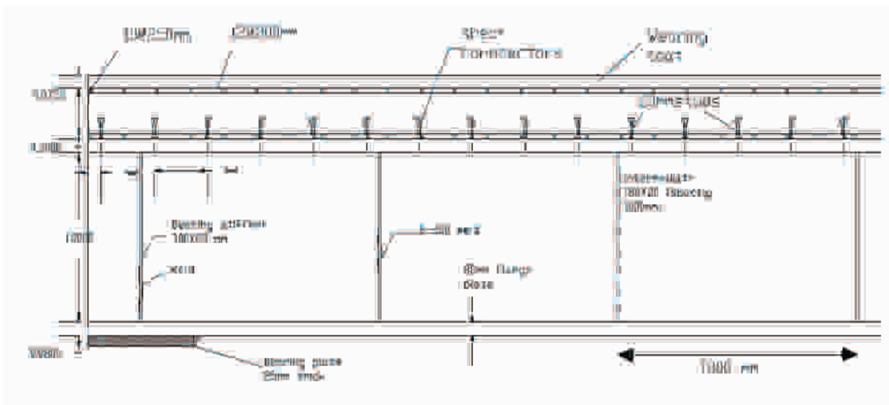
(ii) 4 times the height of the stud = (4×100) = 400 mm

(iii) 600 mm.

Hence adopt a pitch of 400 mm in the longitudinal direction



Cross section of deck slab



Longitudinal Elevation of Composite member