Aug. - Dec. 2016 | Design and drawing of Bridges. (10CV766) |
Sem VII - A & B |
Asst. Prof. K Shijina

First Internal Assesment test solutions

Q.1

Data:-

Carraige way width = 7.5m
foot path = 1m
width of vent = 7m.
wearing course t_w = 80mm
width of bearing = 400mm
M25 & Fe415 grade steel
IRC class AA wheeled vehicle.

Preliminary dimensions

Eff. Depth of slab:-

Assume thickness of slab at 80mm per metre of span for highway bridge decks.

Overall thickness of slab = $(80 \times 7) = 560mm$
$\simeq 600mm$.

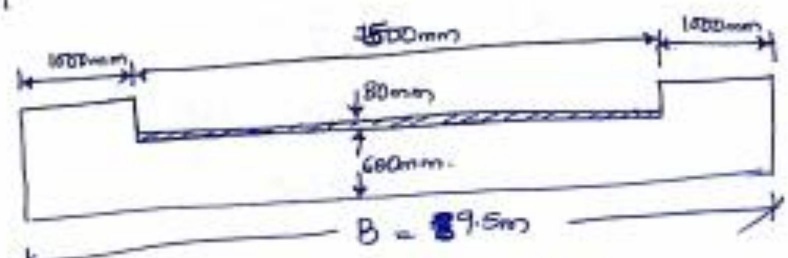
Using 20mm diameter of HYSD with clear cover of 50mm.

Effective thickness = $600 - \left(50 + \frac{20}{2}\right) = 540mm$

Effective span is the least of.

- $7 + 0.4 = 7.4$
- $7 + 0.54 = 7.54$

∴ Effective span = 7.4 m



(ii) Dead load Bending moment per m width of slab.

(a) Weight per m^2 of slab = $0.6 \times 24 = 14.4$ kN

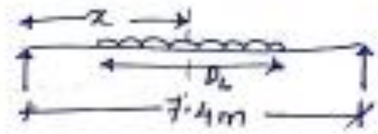
(b) Wearing Course per m^2 of slab = $(0.08 \times 22) = 1.76$ kN

16.16 kN

$\therefore$ Dead load B.M = $\frac{16.16 \times (7.4^2)}{8} = 110.615$ kN-m

(iii) Live load B.M - IRC class AA wheeled vehicles.

Impact factor = 25%.



Effective dispersion length.

$\frac{0.15}{2} + t_w + t_s = \frac{0.15}{2} + 0.08 + 0.54 = 0.695m > 0.6$

$\therefore D_L = 0.15 + 2(t_w + t_s) + 1.2$

$= 0.15 + 2(0.08 + 0.54) + 1.2 = 2.59m.$

Effective dispersion width.

$x = \frac{7.4}{2} = 3.7m$ $\quad B/L = \frac{9.5}{7.4} = 1.284 \quad K = 2.72$

$b_w = 0.3 + 2(t_w) = 0.3 + 2(0.08) = 0.46m.$

$b_{eff} = Kx\left(1 - \frac{x}{L}\right) + b_w = 2.72 \times 3.7\left(1 - \frac{3.7}{7.4}\right) + 0.46$

$= 5.492m.$

$\frac{b_{eff}}{2} = 2.746m \quad \frac{0.3}{2} + c + f_p = \frac{0.3}{2} + 1.2 + 1 = 2.35$

$\frac{b_{eff}}{2} > \left(\frac{0.3}{2} + c + f_p\right)$ & $\frac{b_{eff}}{2} > 0.5$

$D_p = \left(\frac{0.3}{2} + c + f_p\right) + 2.2 + \frac{b_{eff}}{2}$

$= 2.35 + 2.2 + 2.746$

$= 7.296m$

Total load of vehicle including IF = 400 × 1.25 = 500 kN

Average intensity of load = $\frac{500}{2.59 \times 7.296}$ = 26.46 kN/m

Maximum BM due to live load = $\left[\frac{26.46 \times (2.59)}{2} \times \frac{7.4}{2}\right] - \left[\frac{26.46 \times 2.59^2}{8}\right]$

= 104.596 kN-m.

Design B.M = BM due to DL + BM due to LL
= 110.615 + 104.596 = 215.211 kN-m

Structural design of deck slab

σ_{cb} = 8.33 MPa m = 10 n = 0.294

σ_{st} = 200 MPa j = 0.902 Q = $0.5\sigma_{cb} n j$ = 1.1045

Effective depth = $\sqrt{\frac{215.211 \times 10^6}{1.1045 \times 1000}}$ = 441.42 mm < 540 mm provided

Hence OK.

Area of main reinf. req. Ast = $\frac{215.211 \times 10^6}{200 \times 0.902 \times 540}$

= 2209.196 mm²

Provide 20 mm bars spacing = $\frac{314 \times 1000}{2209.196}$ = 142.13

≈ 140 mm c/c.

BM for distribution per m width = 0.3 × 104.596 + 0.2 × 110.615 = 53.50 kN-m

Using 12 mm bars
Eff depth $d_{eff\,dist}$ = 540 − $\left(\frac{20}{2} + \frac{12}{2}\right)$ = 524 mm.

Area of dist. reqr = $\frac{53.5 \times 10^6}{200 \times 0.902 \times 524}$ = 565.96 mm²

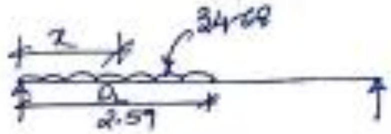
Spacing for dist bars = $\frac{113}{565.96} \times 1000$ = 199.66 ≈ 180 mm c/c

Check for shear

Dead load shear $= \text{Total dead load} \times \frac{\text{Eff span}}{2}$

$= 16.16 \times \frac{7.4}{2}$

$= 59.792\ kN$



$x = \frac{2.59}{2} = 1.295 m$

$b_{eff} = 2.72 \times 1.295 \left(1 - \frac{1.295}{7.4}\right) + 0.46$

$= 3.366\ m$

$\frac{b_{eff}}{2} = 1.683 m \qquad \frac{0.3}{2} + c + f_p = \frac{0.3}{2}\ 2.35$

$\frac{b_{eff}}{2} < \left(\frac{0.3}{2} + c + f_p\right)$ & $\frac{b_{eff}}{2} > 0.5$

$D_p = 2.2 + \frac{b_{eff}}{2} + \frac{b_{eff}}{2} = \underline{5.566 m}$

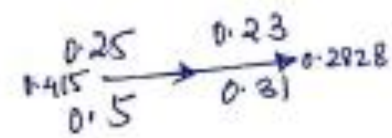
Total live load $= \frac{500}{2.59 \times 5.566} = 34.68\ kN/m$

Live load shear $= \frac{34.68 \times 2.59 \times \left(7.4 - \frac{2.59}{2}\right)}{7.4} = 74.102\ kN$

$\therefore$ Total shear $= 133.894\ kN$

Shear stress $= \tau = \frac{V}{bd} = \frac{133.894 \times 10^3}{1000 \times 540} = 0.248\ N/mm^2$

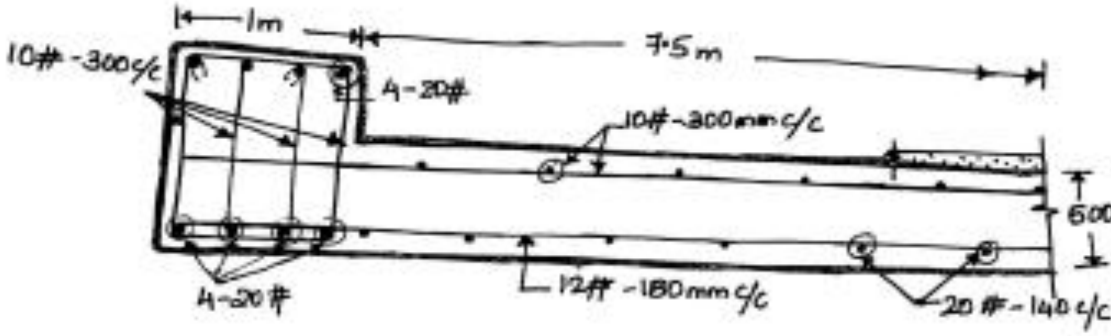
A_{st} provide $= \frac{\pi/4 \times (20^2) \times 1000}{140} = 2242.86\ mm^2$



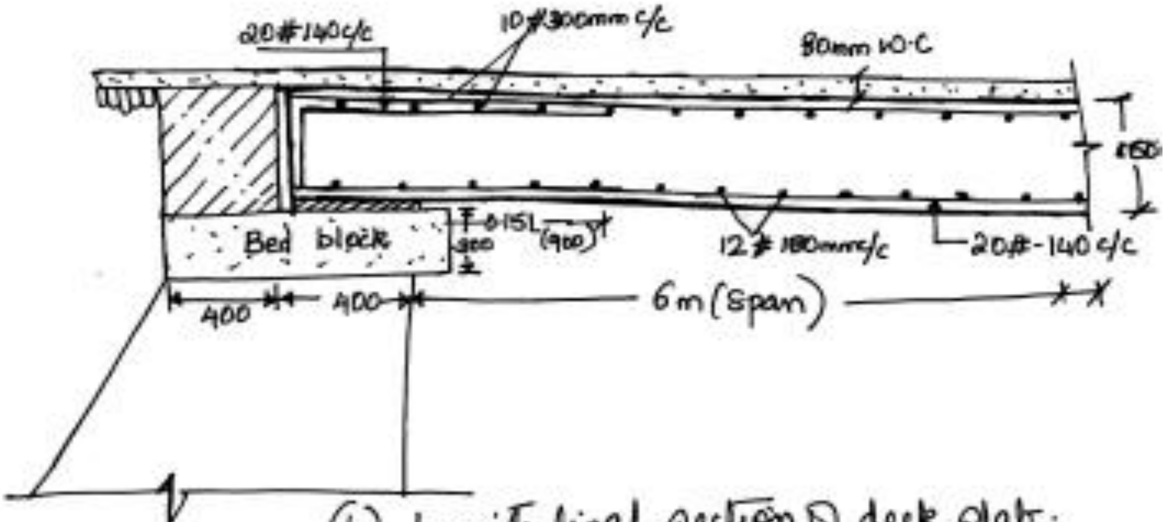
$p_t = \frac{2242.86}{1000 \times 540} \times 100 = 0.415\%$

$\tau_c = 0.2828 \quad \therefore \quad k \cdot \tau_c = 0.2828$

Since $\tau < k\tau_c$ Hence no shear reinf req.



(a) Cross section of deck slab.



(b) longitudinal section of deck slab.

Q 2

a) Diameter of pipe culvert:

Discharge $Q = A \cdot V$ 　　 $Q = \frac{\pi d^2}{4} \times V$

$A = \frac{1.4}{2} = 0.7 \, m^2$ 　　 $d = \sqrt{\frac{4Q}{\pi V}} = 0.944 \, m$

$\frac{\pi d^2}{4} = 0.7 \quad \therefore d = 0.944 \, m$

Adopt NP3 RCC heavy duty non pressure pipe for carrying heavy road traffic. From the Indian standard code

Use NP3 900/100 pipes

i) Load due to Earthfill $= 28.3 \, kN/m$.

ii) load due to IRC class load of 62.5 kN

loading on pipe $= 4 \times C_s \times I.P$

$= 4 \times 0.089 \times 1.5 \times 62.5$

$= 33.375 \, kN/m$.

iii) check for strength factor

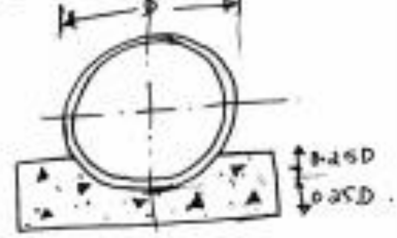
$$\left[\frac{\text{Three Edge bearing Strength}}{F.O.S}\right] = \left[\frac{\text{Load on pipe due to earth fill}}{\text{corresponding SF}} + \frac{\text{Load on pipe due to wheel load.}}{SF}\right]$$

$$\frac{101.40}{1.5} = \frac{28.3}{SF} + \frac{33.375}{1.5}$$

$SF = 0.624$.

Strength factor for First class bedding is 2.8 & for concrete cradle bedding is 3.7. Hence any of these two bedding can be provided for the pipe culvert.

Provide Concrete Cradle bedding



5) Reinforcement in pipe.

The minimum reinforcement for longitudinal & spiral steel reinforcement is 4.35 kg/m & 32.75 kg/m resp.

Using 12mm diameter bars at 60mm centres as spiral reinf

Weight of one spiral of 12mm diameter = $\pi \times 1 \times \frac{\pi}{4} \times \left(\frac{12}{1000}\right)^2 \times 7850$

$= 2.789$ kg

Number of spiral in 1m = $\frac{1000}{60} = 16.67$

Weight of spiral reinf /m length of pipe = 2.789×16.67
$= 46.5$ kg/m

Minimum of spiral reinf = 32.75 kg/m.

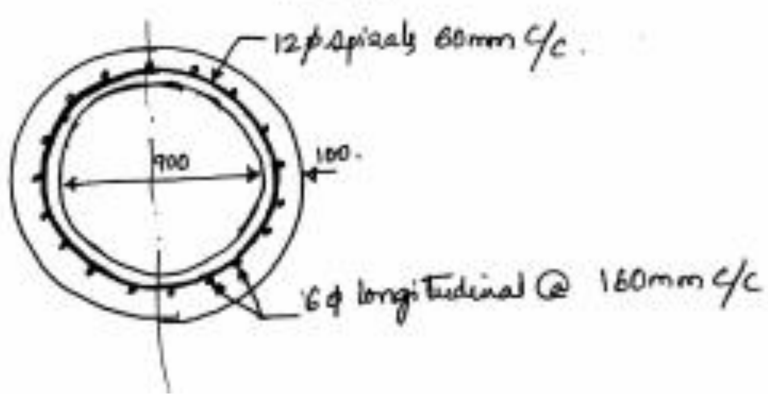
Hence OK.

Providing 6mm diameter mild steel bars as longitudinal reinf

Weight of each bar = $\frac{\pi \times}{4} \left(\frac{6}{1000}\right)^2 \times 7850 = 0.222$ kg/m

Number of bars reqr = $\frac{4.35}{0.222} = 19.6$

Spacing = $\frac{\pi \times 1000}{19.6} = 160.28$ mm
≈ 160 mm



Q2. b

AFFLUX

Afflux is the heading up of water over the flood level caused by construction of waterway at a bridge site.

It is measured by difference in levels of the water surfaces upstream and downstream of the bridge.

$$x = \frac{v^2}{2g}\left[\frac{L^2}{c^2 {L_1}^2} - 1\right]$$

Where, x= afflux

v=velocity of normal flow in the stream

g=acceleration due to gravity

L=width of stream at HFL

L_1=linear waterway under the bridge

c= c

oefficient of discharge through the bridge – (0.7 – for sharp entry) (0.9 - for bell mouthed entry)

Afflux should be kept minimum and limited to 300mm. It causes increase in velocity on the downstream side, leading to greated scour & requiring deeper foundations

Q3. a.

Sr. No.	Main classification	Sub classification
1	Function	Foot; Road; Railway; Road-cum-rail; Pipe line; water conveying (aqueduct); Jetty (Port)
2	Material	Stone; Brick; Stone; Timber; Steel; Concrete; Composite; Aluminium; Fibre
3	Form	Slab; Beam; Arch; Truss; Suspension; Cable supported
4	Type of support	Simply supported; Continuous; Cantilever
5	Position of floor/deck	Deck; Through; Semi through
6	Usage	Temporary; Permanent; Service (Army)
7	With respect to water level	Causeway; Submersible; High level (normal case)
8	Grade separators	Road-over; Road under (sub way); Fly over (Road over road)
9	With respect to connections (Type of jointing)	Pin jointed; riveted/bolted; welded
10	Movable bridges (over	Bascule, Lifting, swing

	navigation channels)	
11	Temporary bridges	Pontoon, Bailey, Callender-Hamilton, Light alloy portable bridges developed by the army.

Q3. b.

FORCES TO BE CONSIDERED FOR THE DESIGN OF BRIDGES

1. Dead load and live loads
2. Dynamic effects
3. Centrifugal force due to curvature of track
4. Temperature and frictional effects
5. Racking force
6. Wind and earthquake forces
7. Earth pressure, including live load surcharge
8. Horizontal forces due to water currents
9. Deformation effects
10. Erection stresses
11. Secondary effects
12. Wave pressure.

Dead load – it includes the self weight of bridge components and the portion of the weight of the superstructure and any fixed loads supported by the member. The dead load can be accurately estimated during design and can be controlled during construction and services.

Live load – bridge design standards specify the design loads, which are meant to reflect the worst loading that can be caused on the bridge by traffic, permitted and expected to pass over it.

Highway bridges are designed in accordance with IRC bridge code. IRC:6 – 2014 – section II gives the specifications for the various loads and stresses to be considered in bridge design. There are three types of standard loadings for which the bridges are designed namely,

IRC class AA loading, IRC class A loading and IRC class B loading.

IRC class AA loading consists of either a tracked vehicle of 700kN or a wheeled vehicle of 400kN with the dimensions as shown below. Normally, bridges on national highways and state highways are designed for these loadings. Bridges designed for class AA should be checked for IRC class A loading also, since under certain conditions, larger stresses may be obtained under class A loading.

Class A loading consists of a wheel load train composed of a driving vehicle and two trailers of specified axle spacings. This loading is normally adopted on all roads on which permanent bridges are constructed. Class B loading is adopted for temporary structures and for bridges in specified areas.

Foot bridges and footpath on bridges – The live load due to pedestrian traffic should be treated as uniformly distributed over the pathway. For the design of footbridges or footpaths on railway bridges, the live load including dynamic effects should be taken as $5.0kN/m^2$.

The live load on footpath for the purpose of designing the main girders has to be taken as follows according to bridge rules:

- For effective spans of 7.5m or less – $4.25kN/m^2$
  The intensity of load is reduced linearly from $4.25kN/m^2$ for a span of 7.5m to $3.0kN/m^2$ for a span of 30m

Q3. C.

ECONOMICAL SPAN

For a given linear waterway, the total cost of the superstructure increases and the total cost of substructure decreases with increase in the span length. The most economical span length is that for which the cost of superstructure equals the cost of substructure.

Let A = cost of approaches

B = cost of two abutments, including foundation.

L = total linear waterway

s = length of one span

n = number of spans

P = cost of one pier, including foundation

C = total cost of bridge

Assuming that the cost of superstructure of one span is proportional to the square of the span length, total cost of superstructure equals n.ks^2, where k is a constant. The cost of railings, flooring etc,

The cost of railings, flooring etc,. is proportional to the total length of the bridge and can be taken as k'L

$$C = A + B + (n-1)P + n.ks^2 + k'L$$

For minimum cost $\frac{dC}{ds}$ should be zero.

Substituting $n = \frac{L}{s}$ and differentiating & equating the result of differentiation to zero, we get

$$P = ks^2$$

Therefore, for an economical span (s_e) can then be computed from

$$s_e = \sqrt{\frac{P}{k}}$$

P and k are to be evaluated as average over a range of possible span lengths.

Q. 3

d) By Empirical method.

$Q = CA^{2/3}$

$Q = 6.5 \times 160^{2/3} = 191.57\ m^3/sec$

= [illegible]

By Rational method.

$Q = AI_0\lambda$

$t_c = \left(0.87 \times \frac{L^3}{H}\right)^{0.385} = \left(0.87 \times \frac{16^3}{96}\right)^{0.385} = 4.02$

$\lambda = \frac{0.56Pf}{t_c+1} = \frac{0.56 \times 0.3 \times 0.67}{4.02+1} = 0.0224$

$Q = 160 \times 55 \times 0.0224 = 197.12\ m^3/sec$

c) By Area velocity method.

$Q = A \cdot V$

$V = \frac{1}{n} R^{0.67} S^{0.5}$

$R = \frac{115}{81} = 1.419$ $S = \frac{1}{500} = 0.002$

$V = \frac{1}{0.3} \times 1.419^{0.67} \times 0.002^{0.5} = 0.188\ m/sec$

$Q = 115 \times 0.188 = 21.62\ m^3/sec$

Design discharge = 197.12 m^3/sec