1) Design a RC slab culvert for NH crossing to suit the following details.

* Carriage way 2 lane → 7.5m wide
* foot path → 1m on either side
* Clear span → 6m
* wearing coat → 80mm
* width of bearing = 400mm
* M_{25} & Fe 415 Steel
* loading = 70R tracked vehicle.

Solution:-

Step 1) Data:-

Carriage way = 7.5m

Foot path = 1m on either side

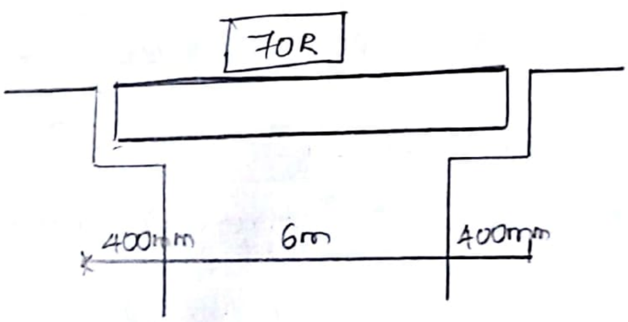
width of bearing = 400mm

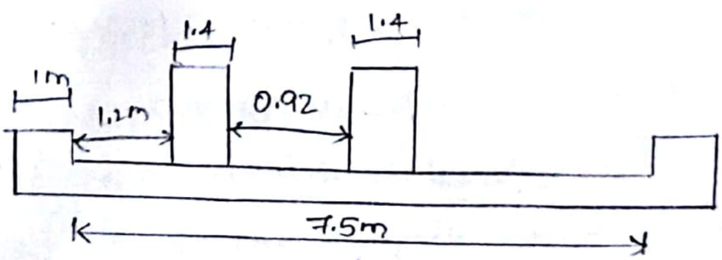
clear span = 6m

wearing coat = 80mm

70R loading tracked = 700 kN

M_{20} & Fe 415





2) Permissible stress:

With reference IRC 21, table 9 & 10

$\sigma_{cb} = 8.33\ N/mm^2$

$\sigma_{st} = 200\ N/mm^2$, $m = 10$

$$n = \frac{1}{1+\frac{\sigma_{st}}{m\sigma_{cb}}} = \frac{1}{1+\frac{200}{10\times 8.33}} = 0.29$$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.29}{3} = 0.9$$

$$Q = 0.5\times\sigma_{cb}\times nj = 0.5\times 8.33\times 0.29\times 0.9 = 1.1$$

3) Depth of slab & effective span.

Overall thickness of slab $= 80\times 6 = 480mm$

adopt overall thickness as 500mm

use 16mmϕ bar & clear cover as 30mm

effective depth $d = 500 - 30 - \frac{16}{2} = 462\ mm$

Effective span:-

1) Clear span $+ 2\times\frac{\text{bearing}}{2} = 6 + \frac{2\times 0.4}{2} = 6.4m$

2) Clear span $+ d = 6 + 0.462 = 6.462m$

$\therefore$ effective span $= 6.4m$

4) Moment calculations:-

Dead load:-

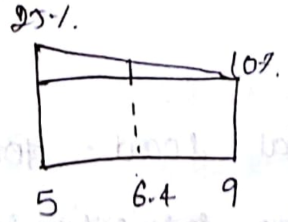
i) self wt of slab $= 0.5\times 1\times 24 = 12\ KN/m$

ii) self wt of wc : $0.08\times 1\times 22 = 1.76\ KN/m$

$13.76 \simeq 14 KN/m$

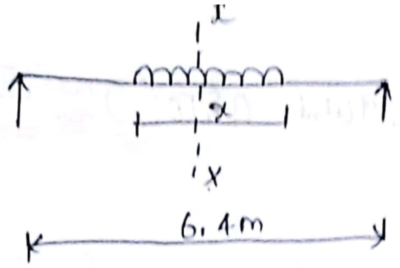
$$M_d = \frac{wl^2}{8} = \frac{14\times 6.4^2}{8} = 71.68 \simeq 72\ KNm.$$

Live load:

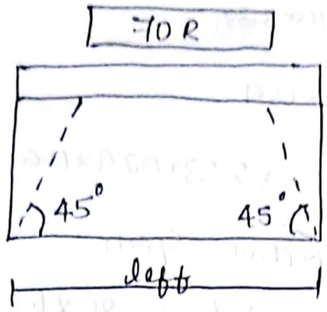


Impact factor for $6.4 = 19.75\%$

loading the slab symmetrically to span of load to span of slab



Effective length of load l_{eff}: $4.57+2(0.5+0.08) = 5.73$



Effective width of the slab $\perp$ to span (b_{eff})

$b_{eff} = \alpha a\left[1-\frac{a}{L_0}\right]+b_2$ [from Pg 52, IRC21]

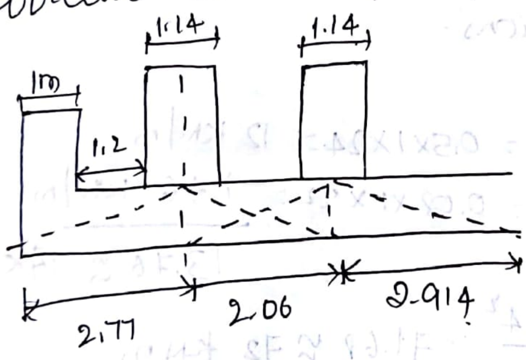
$\frac{b}{L_0} = \frac{9.5}{6.4} = 1.46$

from IRC21, Pg 53, CLS.305 $\alpha = 2.83$

$b_t = 1.44 + 2\times 0.08 = 1.8m$ $a = 3.2m$ $L_0 = 6.4$

$b_{eff} = 2.83\times 3.2\left[1-\frac{3.2}{6.4}\right]+1.3 = 5.828\,m$

Net effective width of dispersion:



Net eff width of load dispersion $= 2.77+2.06+2.914$

$= 7.744m$

Total load $= 700\times 1.975 = 838\,KN$

Avg intensity load $= \frac{838}{5.73\times 7.744} = 18.88\ KN/m^2$

load $= 700\times 1.975 = 838\,KN$

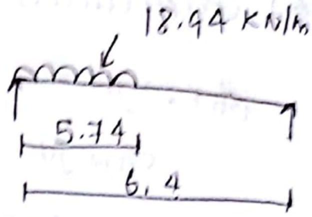
Scanned by CamScanner

Avg intensity of load $= \frac{838}{5.73 \times 7.72}$

$= 18.94 \ KN/m^2$

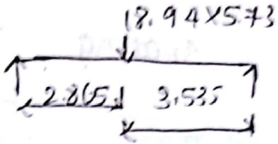
Max SF $= \frac{Wb}{L}$

$= \frac{18.94 \times 5.73 \times 3.535}{6.4}$



$V = 59.94 \approx 60 \ KN$

$V = 60 + 44.8 = 104.8 \approx 105 \ KN$



5) Shear force calculation:-

i) Shear force due to dead load: $\frac{wl}{2} = \frac{14.64}{2}$

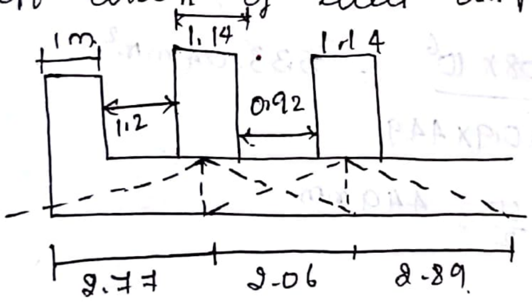
ii) Shear force due to live load $= 44.8 \ KN$

$b_{eff} = \alpha a \left[1 - \frac{a}{L_0}\right] + b$

$a = \frac{5.73}{2} = 2.865 m$

$b_{eff} = 2.83 \times 2.865 \left[1 - \frac{2.865}{6.4}\right] + 1.3 = 5.78 m$

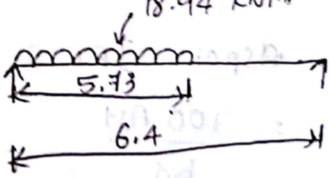
Net eff width of load dispersion



$= 2.77 + 2.06 + 2.89 = 7.72 m$

load $= 700 \times 1.1925 = 838 \ KN$

avg intensity of load $= \frac{838}{5.73 \times 7.772} = 18.94 \ KN/m^2$

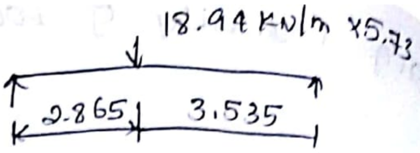


Max SF $= \frac{Wb}{L}$

$= \frac{18.94 \times 5.73 \times 3.535}{6.4}$

$= 59.94 \simeq 60\ KN$

$V = 60 + 44.8 = 104.8 \simeq 105\ KN$



6) Calculation for main reinforcement.

$$A_{st} = \frac{M}{\sigma_{st}\ jd} = \frac{167.61 \times 10^6}{200 \times 0.9 \times 462} = 2015.51\ mm$$

use 16 mm ϕ

$$Spacing = \frac{\frac{\pi}{4} \times 16^2}{2015.51} \times 1000 = 99.57 \simeq 90\ mm$$

Provide 16mm ϕ @ 90mm c/c

8) Check for depth

$$= \sqrt{\frac{M}{Qb}} = \sqrt{\frac{167.61 \times 10^6}{1.1 \times 1000}} = 390.34 < 462\ mm$$

Hence safe

Calculation of distribution Steel

$M = 0.3 ML + 0.2 MD$

$= 0.3 \times 95.61 + 0.2 \times 72$

$= 43.08\ KN$

$$A_{st} = \frac{M}{\sigma_{st} \times j \times d} = \frac{43.08 \times 10^6}{200 \times 0.9 \times 449} = 533.04\ mm^2$$

$d = 500 - 30 - 16 - \frac{10}{2} = 449\ mm$

use 10mm ϕ bars

$$Spacing = \frac{\frac{\pi}{4} \times 10^2}{533.04} \times 1000 = 147.34 \simeq 140\ mm$$

Permissible Shear Stress

As per IRC 21, Table 126, Tc for corresponding

$$= \frac{100 \cdot A_{st}}{bd}$$

Ast provided = $\frac{\frac{\pi}{4} \times 16^2}{90} \times 1000$

$= 2234.021 \; mm^2$

$= \frac{1000 \times 2234.02}{1000 \times 462}$

$= 0.5 \; N/mm^2$

★ Permissible shear stress in solid slab.

$= K \times \tau_c$ from IRC 21, Table 12C for depth of 500mm K value = 1

$= 1 \times 0.31$

$= 0.31 \; N/mm^2$

Hence $\tau < \tau_c$ Shear stress within the permissible limit Hence safe.

Verify the stability of the abutment shown in Fig. 12.6. The other salient details are given below:

Material of the abutment: Concrete
Density of the soil: 18 kN/m^3
Coefficient of friction: 0.6
Angle of repose of the soil: ϕ = 30°
Live load on the bridge: IRC Class AA (Tracked)

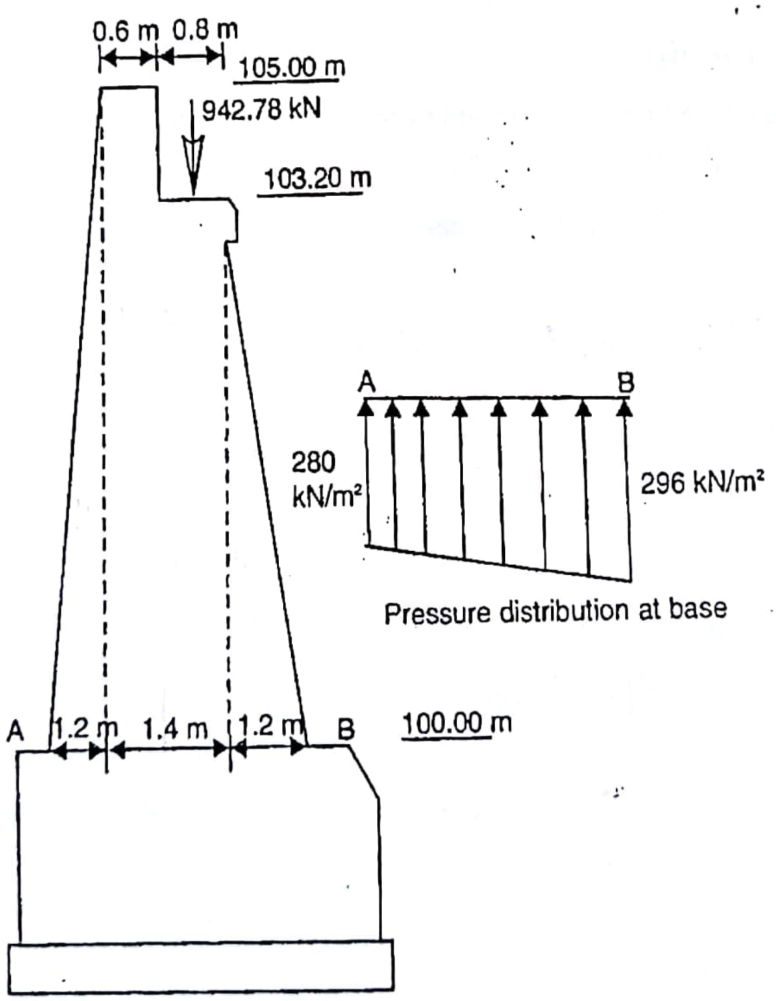


Fig. 12.6 Section of the abutment (Example 12.1).

Span of the bridge: 15 m

Angle of friction between the soil and concrete: $\delta = 18°$

The bridge deck consists of three longitudinal girders of 1.4 m depth with a deck slab of 200 mm depth.

Analysis

The stability of the abutment is verified at bed level.

Self-weight of the abutment

$$= 24\,[(0.6 \times 5) + (0.8 \times 3.2) + (0.5 \times 1.2 \times 3.2) + (0.5 \times 1.2 \times 5)]$$
$$= 251.52 \text{ kN}$$

Dead load from superstructure

$$= (3 \times 1.4 \times 0.3 \times 15 \times 24) + [(24 \times 0{,}2) + (22 \times 0.08)] \times 8.7 \times 15$$
$$= 453.60 + 856.08 = 1309.68 \text{ kN}$$

Dead load per abutment = 1309.68/2 = 654.84 kN

3.4×2

0.75×2

Dead load per metre run of abutment = 654.84/8.7 = 75.26 kN {where 8.7 m is the width of the deck}

Reaction due to live load

Live load reaction is maximum when the wheel is nearer to the support such that the tip of the wheel touches the support, letting the full portion of the wheel within the span.

Live load reaction = 700[15 – (3.6/2)]/15 = 616 kN

350 * 350

Total load

Total load = 251.52 + 75.26 + 616 = 942.78 kN

Earth pressure

Earth pressure is calculated using Coulomb's formula

$\phi = 30°$, $\tan\theta = 1.2/5$, therefore, $\theta = 13.50°$

Total earth pressure = $0.5 \times 18 \times 5^2 \times \cos 13.50°\, k_a$

where k_a is given by [active earth pressure coefficient]

$$k_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta\cos(\partial + \theta) + \sqrt{\dfrac{\sin(\partial + \phi)\sin\phi}{\cos(\partial + \phi)\cos\phi}}}$$

Upon substitution, $k_a = 0.853$

assume.

Therefore,

$$\text{Earth pressure} = 0.5 \times 18 \times 5^2 \times \cos 13.50° \times 0.853 \doteq 181.46 \text{ kN}$$

$$\text{Horizontal component of earth pressure} = 181.46 \cos(\delta + \theta) = 181.46 \cos(18 + 13.50)$$
$$= 154.72 \text{ kN}$$

$$\text{Vertical component of earth pressure} = 181.46 \sin 31.50 = 94.81 \text{ kN}$$

$$\Sigma V = 94.81 + 942.78 = 1037.59 \text{ kN}$$

$$\Sigma H = 154.72 \text{ kN}.$$

$$\text{Resultant} = \sqrt{(1037.59)^2 + (154.72)^2} = 1049.06 \text{ kN}$$

Check against overturning

The earth pressure is assumed to act at a height of $0.42h' = 0.42 \times 5 = 2.1$ m

Moments having overturning effect $= 2.1 \times 154.72 = 324.91$ kN.m

Restoring moments $\frac{0.6}{2} + 0.8 + 1.2$ $\quad 1.2 \times \frac{2}{3}$ $\quad \frac{0.8}{2} + 1.2$

$$= (0.6 \times 5 \times 2.3 \times 24) + (0.5 \times 1.2 \times 3.2 \times 24 \times 0.8)$$
$$+ (0.8 \times 3.2 \times 24 \times 1.6) + (0.5 \times 1.2 \times 5 \times 24 \times 3) + (942.78 \times 1.6)$$
$$= 2025.20 \text{ kN·m}$$

$\frac{0.8}{2} + 1.2$ $\quad (\frac{1}{3} \times 1.2) + 1.4 + 1.2$

Factor of safety against overturning $= 2025.20/324.91 = 6.23 > 2$. Therefore, the abutment is safe against overturning.

Check against sliding

Factor of safety $= (0.6 \times 942.78)/154.72 = 3.65 > 2$. Therefore, the abutment is safe against sliding.

Maximum and minimum base pressures

$$\text{Distance of the resultant from the toe} = \frac{2025.20 - 324.91}{1049.06} = 1.62 \text{ m}$$

Eccentricity of the resultant from the centre of the base $\quad 3.8$

$$e = \frac{3.27}{2} - 1.62 = 0.015 < \frac{b}{6}$$

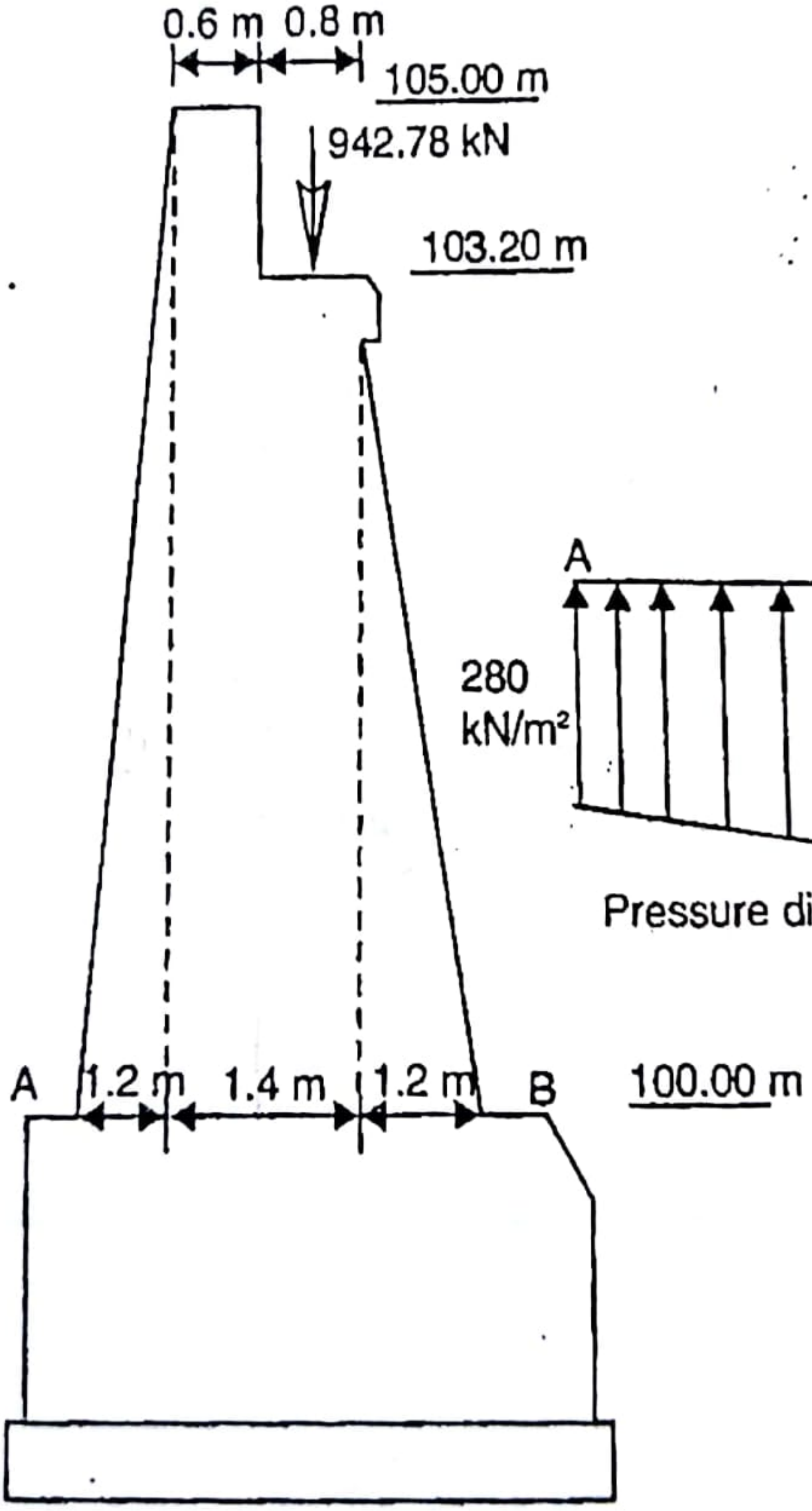
Therefore,

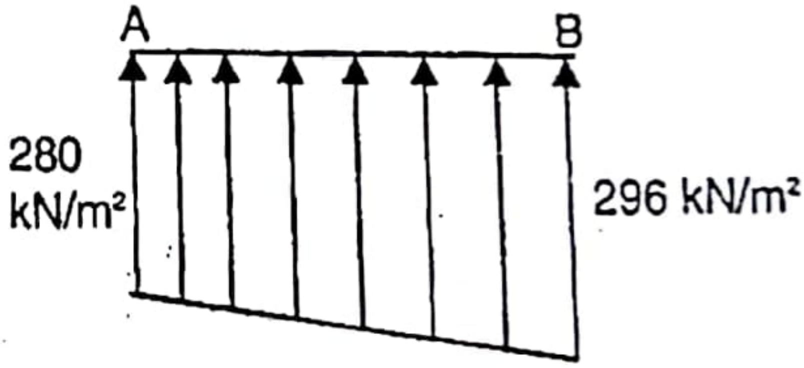
$$\text{Maximum pressure } p_{max} = \frac{942.78}{3.27}\left(1 + \frac{6 \times 0.015}{3.27}\right) = 296.24 \text{ kN/m}^2$$

$$\text{Minimum pressure } p_{min} = \frac{942.78}{3.27}\left(1 - \frac{6 \times 0.015}{3.27}\right) = 280.37 \text{ kN/m}^2$$

Stresses are within limits as the compressive stress for concrete is 2000 kN/m^2.

The abutment cross-section and the pressure distribution at the base are shown in Fig. 12.6.





Pressure distribution at base

Verify the adequacy of the dimensions for the pier shown in Fig. 12.7. The following details are available:

Top width of the pier: 1.6 m

Height of the pier up to springing level: 10 m

c/c of bearings on either side: 1.00 m

Side batter: 1 in 12

High flood level: 1 m below the bearing level

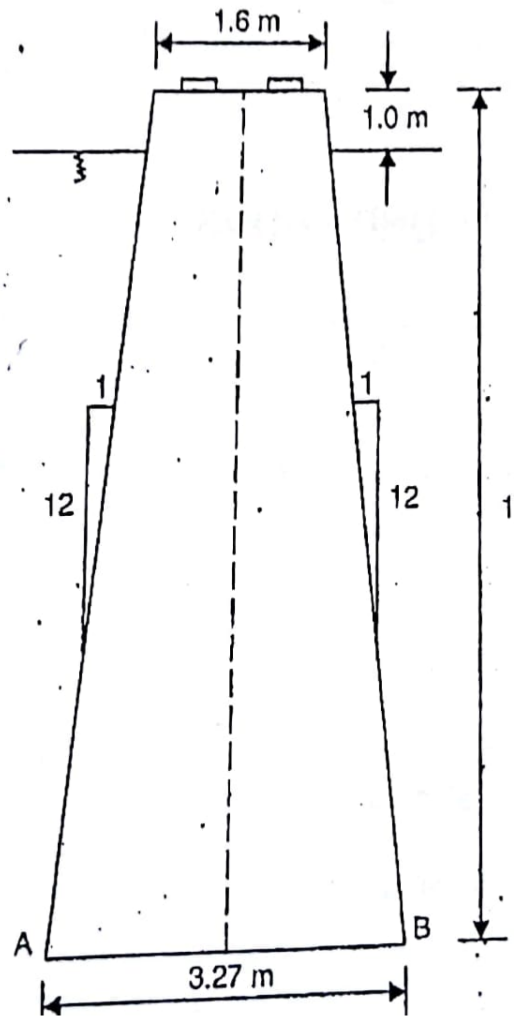
Span of the bridge: 16 m

Loading on span: IRC Class AA

Road: Two-lane road with 1 m wide footpath on either side.

Superstructure: Consists of three longitudinal girders of 1.4 m depth with a deck slab of 200 mm depth. Rib width of girders = 300 mm

Material of the pier: Concrete M15



Analysis

Base width at bed level = 1.6 + (1/12) × (10 × 2) = 3.27 m

Pier length required = 7.5 + (2 × 1) = 9.50 m

3.75 × 2 .

Self-weight of the pier

Area at top = (9.5 × 1.6) + (2 × π × 0.8^2/2) = 17.21 m^2

Area at bottom = (3.27 × 9.5) + (2 × π × 1.6^2/2) = 39.11 m^2

Self-weight = (1/2) × (17.21 + 39.11) × 10 × 24 = 6578.40 kN

Moment of inertia with respect to *X-X* axis

$$= \frac{9.5 \times 3.27^3}{12} + \frac{2\pi \times 3.27^4}{128} = 33.29 \text{ m}^4$$

Dead load from the superstructure

This is due to longitudinal girders and deck slab of the superstructure.
Roughly, it is given by

= (3 × 1.4 × 16 × 0.3) 24 + (24 × 0.2 + 0.08 × 22) × (9.5 × 16)

= 1480.96 kN

Therefore,
Load per metre length of pier = 1480.96/9.5 = 155.89 kN

Design dead load = 6758.40 + 155.89 = 6914.29 kN

Stresses at bottom owing to dead load = 6914.29/39.11 = 176.79 kN/m^2

39.11

Stresses owing to buoyancy

Owing to buoyancy, the pier gets lifted, i.e. there is a relief in stress value. Therefore, stresses due to buoyancy are always negative.

Width of the pier at HFL = 1.6 + (2 × 0.1) = 1.8 m

Area of the pier at HFL = (1.8 × 9.5) + (π × 0.9^2/2) = 18.37 m^2

Submerged volume of the pier = [(18.37 + 39.11)/2] × 9 = 258.66 m^3

Reduction in weight of the pier owing to buoyancy = Weight of the displaced water

= 258.66 × 10 = 2586.6 kN

Stress at base = 2586.6/39.11 = – 66.13 N/mm^2

Stress owing to live load

Reaction owing to live load (Class AA) including impact = 1.1 × 700 = 770 kN

Maximum bending moment at base = 770 × 0.5 = 385 kN/m

Maximum stress at base = (770/39.11) + (385 × 3.27)/(33.29 × 2) = 38.59 kN/m^2

Minimum stress at base = (770/39.11) – (385 × 3.27)/(33.29 × 2) = 0.779 ≈ 0.8 kN/m^2

Stresses owing to longitudinal force

Longitudinal force may be taken as 20% of IRC Class AA loading that is $0.2 \times 700 = 140$ kN

Moment owing to this force at base = $140 \times 10 = 1400$ kN·m

Stresses at base = $\pm (1400 \times 3.27)/(33.29 \times 2) = \pm 68.75$ kN/m^2

Stresses owing to water current

Velocity of water may be taken as 3 m/s

Water pressure = $5.2\ kv^2$ (k is a constant = 0.66 for semicircular cut and ease water).

$= 5.2 \times 0.66 \times 3^2 = 30.88$ kN/m^2

Area of the wetted surface of pier = $9(1.8 + 3.27)/2 = 22.81$ m^2

Force owing to water current = $30.88 \times 22.81 = 704.37$ kN

For the worst effect, the current direction is taken as 20°

Force perpendicular to pier = 704.37 cos 20°

= 661.89 kN

Moment at the base owing to this force = $661.89 \times (2/3) \times 9 = 3971.34$ kN·m

Stresses at the base owing to this force = $\pm [3971.34/33.29] \times (3.27/2) = \pm 195.04$ kN/m^2

Summation of all the stresses

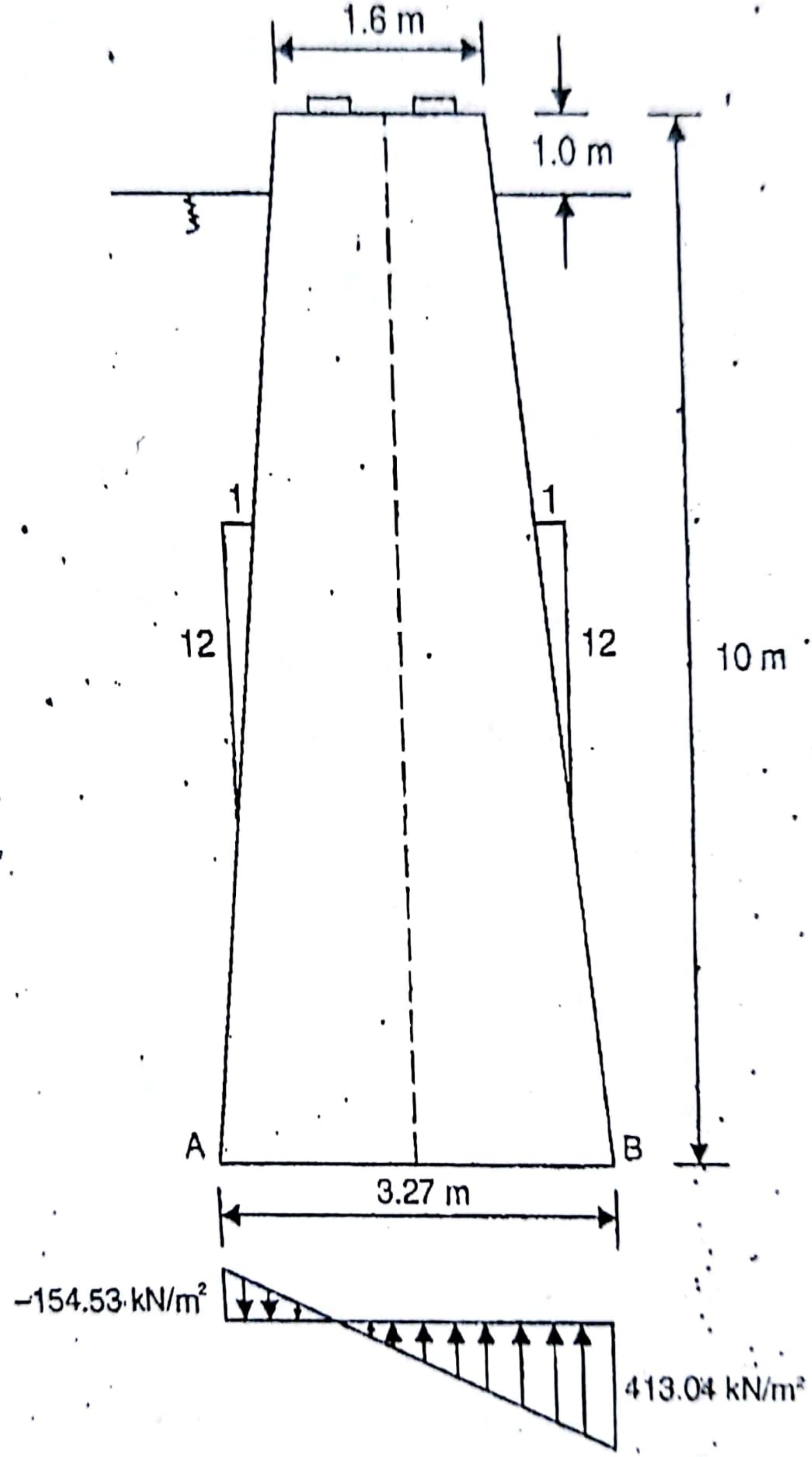
Maximum stress = $176.79 + 38.59 - 66.13 + 68.75 + 195.04 = 413.04$ kN/m^2

Minimum stress = $176.19 - 0.8 - 66.13 - 68.75 - 195.04 = -153.93$ kN/m^2

The stresses developed at the base are within limits.

Negative pressure indicates development of tension at the bottom. This is undesirable. To abate this, the bottom and top widths of the pier may be slightly altered.

The pier cross-section and pressure distribution at the base are shown in Fig. 12.7


1.6 m
1.0 m
10 m
1
12
1
12
A
B
3.27 m
–154.53 kN/m²
413.04 kN/m²

Design a pipe culvert through a road embankment of height 6 m. The width of the road is 7.5 m and the formation width is 10 m. The side slope of the embankment is 1.5:1. The maximum discharge is 5 m^3/s. The safe velocity is 3 m/s. Class AA tracked vehicle is to be considered as live load. Assume bell-mouthed entry. Given $C_e = 1.5$, $C_s = 0.010$ and the unit weight of the soil = 20 kN/m^3.

Solution

Hydraulic design

Discharge through the pipe

$$Q = KAV$$

where

$$K = \frac{1}{\sqrt{1 + K_e + K_f}}$$

Now,

$$K_f = 0.0033\frac{L}{(R)^{1.3}}$$

where L is the length of the pipe, which is equal to the base width of the embankment. Therefore

$$L = 10 + (2 \times 1.5 \times 6) = 28 \text{ m}$$

Assuming 1 m diameter pipe, we have

$$R = \frac{A}{P} = \frac{\frac{\pi}{4}D^2}{\pi D} = \frac{D}{4} = \frac{1}{4} = 0.25$$

Therefore,

$$K_f = \frac{0.0033 \times 28}{(0.25)^{1.3}} = 0.56$$

and

$$K_e = 0.08 \text{ for bell-mouthed entry}$$

Therefore, we have

$$\text{Conveyance factor} = \frac{1}{\sqrt{1 + 0.08 + 0.56}} = 0.78$$

Hence,

$$5 = A \times 0.78 \times 3$$

or

$$A = 2.13 \text{ m}^2$$

Area provided by each pipe

$$= \frac{\pi D^2}{4} = \frac{\pi \times 1^2}{4} = 0.785 \text{ m}^2$$

Therefore the no. of pipes required

$$= \frac{2.13}{0.785} = 2.71 \approx 3$$

Bedding for the pipes

From Table 6.3, for a pipe of internal diameter 1 m, the external diameter is 1.23 m. Therefore, Height of the embankment over the pipe = (6 – 1.23) = 4:8 m

As $C_e = 1.5$, therefore, the load on the pipe owing to earth fill

$$C_e w D^2 = 1.5 \times 20 \times 1.23^2$$
$$= 45.4 \text{ kN/m}$$

and load on the pipe owing to wheel load

$$4C_s IP = 4 \times 0.010 \times 1.5 \times 700 \quad \text{(see Fig. 4.1)}$$
$$= 42 \text{ kN/m}$$

Bedding is chosen based on the strength factor. Referring to IS 458–1988, three edge bearing strength for a NP3 pipe of 1000 mm internal diameter is 72 kN/m. Hence the equation to be satisfied is

$$\frac{\text{Three edge bearing strength (kN/m)}}{\text{Factor of safety}} = \frac{\text{Load owing to earth fill (kN/m)}}{\text{Strength factor (SF)}} + \frac{\text{Load owing to wheel load}}{\text{Factor of safety}}$$

or

$$\frac{72}{1.5} = \frac{45.4}{\text{SF}} + \frac{42}{1.5}$$

Therefore,

$$\text{SF} = 2.30$$

Hence concrete cradle bedding may be provided (see Table 6.2).

Reinforcements

The minimum reinforcements to be provided in the pipe according to IS 458–1988 (Table 6.5) are:

Spiral reinforcement = 21.52 kg/m
Longitudinal reinforcement = 2.66 kg/m
Weight of the 12 mm spiral (diameter = 1.1 m)

$$= \frac{\pi \times 0.012^2 \times 7850}{4} \ (\pi \times 1.1) = 3.068 \text{ kg/m}$$

Providing 30 kg/m of spiral, no. of spirals = $\frac{30}{3.068} = 9.77 \approx 10$

c/c distance = 1000/10 = 100 mm

Providing 6 mm dia mild steel bars as longitudinal steel and providing 4 kg/m of run,

Weight of a single bar = $\frac{\pi \times 0.006^2 \times 1 \times 7850}{4} = 0.22$ kg/m

Providing at 4 kg/m, no. of bars = $\frac{4}{0.22} = 18.18$

Spacing = $\frac{\pi \times 1100}{18.18} = 190.08$ mm $\approx$ 150 mm c/c

The details of reinforcements are shown in Fig. 6.5.

The drawing of the pipe culvert is presented in Plate 3.