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Detailed Solutions

SSC-JE 2018 Mains Test Series (PAPER-II)

Civil Engineering Test No : 6

Q.1 (a) Solution:

Given: $Q = 3.5$ MLD; $L = 30$ m; $B = 9$ m; $H = 4$ m; MLSS = 1800 mg/l; $S_0 = 130$ mg/l; $X_0 = 150$ mg/l

BOD loading i.e.Organic Loading Rate = $\frac{Q_0 S_0}{V} = \frac{3.5\times10^6\times(130\times10^{-6})}{(30\times9\times4\times10^{-4})}$ kg/ha-m/day

$$= 4212.963 \text{ kg/ha-m/day} \simeq 4213 \text{ kg/ha-m/day}$$

$$\frac{F}{M} = \frac{Q_0 S_0}{VX} = \frac{3.5\times10^6\times130\times10^{-6}}{(30\times9\times4)\times(1800\times10^{-6})\times10^3} = 0.234$$

Aeration period, $t = \frac{V}{Q} = \frac{30\times9\times4}{3.5\times10^6\times10^{-3}}$ day $= 7.41$ hours

Sludge Age: Volume, $V = 30 \times 9 \times 4 \text{ m}^3 = 1080 \text{ m}^3$

$$X = 1800 \text{ mg}/l$$

$$Q = 3.5 \times 10^6\ l/\text{d}$$

$$X_0 = 150 \text{ mg}/l$$

$$\theta_c = \frac{V.X}{QX_0} = \frac{(1080\times10^3)\times1800}{150\times3.5\times10^6} \text{ days}$$

$\therefore$ $\theta_c = 3.70$ days

Q.1 (b) Solution:

(i) at $z = 10$, $r = 0$

$$\sigma_Z = k_B \cdot \frac{Q}{z^2} = \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2} \cdot \frac{Q}{z^2} = \frac{3}{2\pi}\left(\frac{1}{1+0}\right)^{5/2} \cdot \frac{200}{10^2}$$

$$= 0.9549 \text{ kN/m}^2$$

(ii) at $z = 10$ m, $r = 5$ m

$$\sigma_Z = k_B \cdot \frac{Q}{z^2} = \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{r}{z}\right)^2}\right)^{5/2} \cdot \frac{Q}{z^2} = \frac{3}{2\pi} \times \left(\frac{1}{1+\left(\frac{5}{10}\right)^2}\right)^{5/2} \cdot \frac{200}{10^2}$$

$$= 0.5466 \text{ kN/m}^2$$

Q.1 (c) Solution:

Kennedy's theory: According to Kennedy, the critical velocity in the channel may be defined as the mean velocity of flow which will just keep the channel free from silting or scouring on basis of observations made on certain stable reaches of upper Bari Doab canal system, Kennedy gave the following formula for critical velocity

$$V_0 = 0.55\, mD^{0.64}$$

Where D = depth of the flow in the channel in metres.

This equation is applicable to channels flowing in sandy silt of the same grade as that which exists an upper Bari Doab canal system on which observations were made.

Kennedy later reorganized that grade played a significant role in determining the critical velocity and therefore introduced another factor in the above equation, Critical velocity ratio (CVR), m to make equation applicable to other regions also. The equations was then given by;

$$V = 0.55\, mD^{0.64} \quad \text{...(i)}$$

For the bed slope of the channel Kennedy considered Kutter's formula as noted below;

$$V = \frac{23 + \frac{1}{n} + \frac{0.00155}{s}}{1+\left(23 + \frac{0.00155}{s} + \frac{n}{\sqrt{R}}\right)}\sqrt{RS}$$

Using these equations the following procedure may be adopted:

The design discharge Q, rugosity coefficient n, bed slopes and CVR(m) are known.

(i) Assume trial value of D metres.

(ii) Compute value of V using equation (i)

(iii) Compute area of flow section using continuity equation, $A = \frac{Q}{V}$.

(iv) Assuming side slope of the channel as $\frac{1}{2}:1$, compute the bed width of the channel B using the relation;

$$A = (B + 0.5D)D$$

(v) Compute the wetted perimeter P using the relation

$$P = \left(B + \sqrt{5}D\right)$$

and also compute the hydraulic radius R since $R = \frac{A}{P}$

(vi) Compute mean velocity of flow from Kutter's equation. If this velocity is same as that found in step (ii), assumed depth is correct.

Q.1 (d) Solution:

Extra widening of pavement, $W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$

Where, $n = 2$, $l = 6$ m, $R = 300$ m, $V = 80$ km/h

hence, $$W_e = \frac{2 \times 6^2}{2 \times 300} + \frac{80}{9.5 \times \sqrt{300}} = 0.606\,\text{m}$$

Now the length of transition curve is maximum of the following:

(i) By rate of change of centrifugal acceleration:

Allowable rate of change of centrifugal acceleration, C ,is given by

$$C = \frac{80}{75 + V} = \frac{80}{75 + 80} = 0.516\,\text{m/sec}^3$$

$$L_s = \frac{0.0215V^3}{CR} = \frac{0.0215 \times 80^3}{0.516 \times 300} = 71.11\,\text{m}$$

(ii) By rate of introduction of total superelevation:

Superelevation, $$e = \frac{V^2}{225R} = \frac{80^2}{225 \times 300} = 0.0948 > 0.07$$

So adopt, $e = 0.07$

Check for the transverse skid resistance developed,

$$f = \frac{V^2}{127R} - e = \frac{80^2}{127 \times 300} - 0.07 = 0.098 < 0.15 \quad \text{(Safe)}$$

Total pavement width including extra widening,

$$B = W + W_e = 2 \times 3.5 + 0.606 = 7.606 \text{ m}$$

Rate of introduction of super elevation = 1 in 150

$$\therefore \quad L_s = eN\frac{(W+W_e)}{2} = 0.07 \times 150 \times \frac{7.606}{2} = 39.93 \text{ m}$$

(iii) From IRC formula,

$$L_s = 2.7 \times \frac{V^2}{R} = 2.7 \times \frac{80^2}{300} = 57.6\text{m}$$

Adopt maximum of the above three values and therefore the length of transition curve is 71.11 m.

Q.2 (a) Solution:

The latitude and departure of the lines are given by:

$$\text{Latitude } (L) = l\cos\theta$$

$$\text{Departure } (D) = l\sin\theta$$

Line	Length	Bearing	Latitude	Departure
AB	217.50	S 59°45′ E	–109.57	187.88
BC	300.00	N 62°30′ E	138.52	266.10
CD	375.00	N 37°36′ W	297.11	–228.80
DE	280.00	N 24°42′ W	254.38	–117.00

Sum of Latitude,

$$\Sigma L = -109.57 + 138.52 + 297.11 + 254.38$$

$$= 580.44 \text{ m}$$

Sum of departure,

$$\Sigma D = 187.88 + 266.10 - 228.80 - 117.00$$

$$= 108.18 \text{ m}$$

Let L be the latitude of EA and D be the departure of EA, then

$$580.44 + L = 0$$

$$\Rightarrow \quad L = -580.44 \text{ m}$$

Similarly,

$$D = -108.18 \text{ m}$$

$$\text{Length of } EA = \sqrt{L^2 + D^2} = \sqrt{(-580.44)^2 + (-108.18)^2} = 590.43 \text{ m}$$

$$\tan\theta = \frac{\Sigma D}{\Sigma L} = \frac{108.18}{580.44} = 0.1864$$

$$\theta = 10°33'26.72''$$

As both the latitude and departure of EA are negative, the quadrant is S-W.

Bearing of EA = **S10°33′26.72″ W**

Q.2 (b) Solution:

The uses of contour maps in civil engineering projects are as follows:

1. **Drainage area estimation :** The extent of drainage area may be estimated on contour map by locating the ridge line around the watershed.
2. **Capacity of Reservoir :** Reservoirs are made for water supply and for power or irrigation projects. A contour map is very useful to study the possible location of a dam and the volume of water to be confined. All contour lines are closed within the reservoir area.
3. **Site of structures :** The most economical and suitable site for the structure such as buildings, bridges, dams etc. can be found from large scale contour maps.
4. **Earthwork Estimates :** On the contourlines of original surface, the contours of the desired altered surface are drawn. The volume of a cut or a fill is found by multiplying the average by the contour interval.
5. **Route location :** By inspection a contour map, the most suitable site. For road, railway, canal etc. can be selected. By following the contour lines, steep gradients, cutting and filling etc. may be avoided.

Q.2 (c) Solution:

Ballast: Ballast is a layer of broken stone gravel, moorum or any other gritty material which is placed and packed around and below sleepers to distribute the load from sleeper to formation and for providing drainage to track as well as giving lateral and longitudinal stability to track.

Uses/ Function of Ballast in Railway track:

(a) to transfer and distribute the vertical load from sleepers to larger area of formation.
(b) to provide necessary resistance to sleepers and track for lateral longitudinal stability.
(c) to provide effective drainage to track.
(d) to provide elasticity and resilience to track for getting proper riding comfort.
(e) to provide flexibility and means to maintain geometric alignment of track.

Different type of ballast used:

1. Stone aggregate
2. Brick aggregate
3. Sand ballast
4. Moorum ballast
5. Coal ash ballast

Necessity to replace stone ballast from time to time: Stone ballast needs to be cleaned time to time to maintain its drainage property, however cleaning can only be done a certain number of time before the ballast is damaged to the point that it cannot be reused, further more, track ballast that is completely fouled cannot be corrected by shoulder cleaning, in such case it is necessary to replace the ballast altogether.

Q.2 (d) Solution:

Given: Length, $l = 16 \text{ m} = 16 \times 10^3 \text{ mm}$

Cross-sectional area, $A = 4 \text{ mm}^2$

Weight of the wire ABC, $W = 20 \text{ N}$

Modulus of elasticity, $E = 200 \text{ GPa} = 200 \times 10^3 \text{ N/mm}^2$

Deflection at C

What know that deflection of wire at C due to self weight of the wire AC.

$$d\,l_C = \frac{Wl}{2AE} = \frac{20 \times (16 \times 10^3)}{2 \times 4 \times (200 \times 10^3)} = 0.2 \text{ mm}$$

Deflection at B

We know that the deflection at B consists of deflection of wire AB due to self weight plus deflection due to weight of the wire BC. We also know that deflection of the wire at B due to self weight of wire AB

$$\delta\,l_1 = \frac{\left(\frac{W}{2}\right) \times \left(\frac{l}{2}\right)}{2AE} = \frac{10 \times \left(8 \times 10^3\right)}{2 \times 4 \times \left(200 \times 10^3\right)} = 0.05 \text{ mm} \qquad \text{...(i)}$$

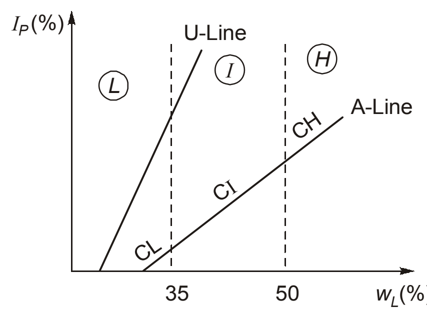
and deflection of the wire at B due to weight of the wire BC

$$\delta\,l_2 = \frac{\left(\frac{W}{2}\right) \times \left(\frac{l}{2}\right)}{AE} = \frac{10 \times \left(8 \times 10^3\right)}{4 \times \left(200 \times 10^3\right)} = 0.1 \text{ mm} \qquad \text{...(ii)}$$

$\therefore$ Total deflection of the wire at B

$$\delta\,l_B = \delta l_1 + \delta l_2 = 0.05 + 0.1 = 0.15 \text{ mm}$$

Q.3 (a) Solution:



Equations of *A*-line and *U*-line for plasticity chart:

***A* - line** $\rightarrow I_P = 0.73\,(w_L - 20)$

***U* - line** $\rightarrow I_P = 0.9\,(w_L - 8)$

- Plasticity chart is used to classify fine grained soils.

  $L \rightarrow$ Low compressible/Low plastic $\rightarrow w_L < 35\%$

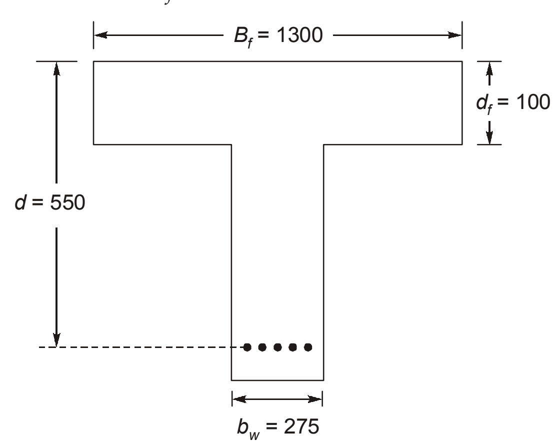
  $I \rightarrow$ Medium compressible/Medium plastic $\rightarrow w_L = 35$ to 50%

  $H \rightarrow$ High compressible/High plastic $\rightarrow w_L > 50\%$

  $C \rightarrow$ clay $\quad M \rightarrow$ silt $\quad O \rightarrow$ Organic soil.
- Plasticity index of soil is calculated and let it be $(I_P)_{soil}$.
- $(I_P)_{soil} > (I_P)_{A\text{-line}}$, then soil is clay else it is silt or organic soil.
- *U*-line represents upper boundary above which no result should lie. If results are found to be above *U*-line then test should be repeated.

Q.3 (b) Solution:

Given effective flange width, $B_f = 1300$ mm



Area of steel, $A_{st} = 5 \times \frac{\pi}{4} \times 25^2 = 2454.3693\ \text{mm}^2$

Limiting depth of neutral axis, $(x_u)_{\lim}$ = 0.48d

$$(x_u)_{\lim} = 0.48 \times 550 = 264 \text{ mm}$$

Assuming actual depth of neutral axis $x_u < d_f$

$$\Rightarrow \quad 0.36 f_{ck}.\, B_f.\, x_u = 0.87 f_y.\, A_{st}$$

$$0.36 \times 15 \times 1300 \times x_u = 0.87 \times 415 \times 5 \times \frac{\pi}{4} \times 25^2$$

$$\Rightarrow \quad x_u = 126.23 > d_f$$

Hence our assumption is wrong.

Assuming $\quad x_u > d_f$

and $\quad \frac{3}{7}x_u < d_f$

$$\therefore \quad y_f = 0.15\, x_u + 0.65\, d_f = 0.15\, x_u + 0.65 \times 100$$

$$= 0.15\, x_u + 65$$

Hence, $C_u = 0.36 f_{ck}\,.\, b_w.\, x_u + 0.45 f_{ck}\,(B_f - b_w).y_f = 0.87 f_y.\, A_{st}$

$$0.36 \times 15 \times 275 \times x_u + 0.45 \times 15\,(1300 - 275)(0.15\, x_u + 65) = 0.87 \times 415 \times 2454.3693$$

$$\Rightarrow \quad x_u = 172.99 \text{ mm} > d_f$$

$$\frac{3}{7}x_u = 74.14 \text{ mm} < d_f$$

Hence our assumption is correct.

$$\therefore \quad \text{Depth of neutral axis, } x_u = 172.99 \text{ mm}$$

$$y_f = 0.15 \times 172.99 + 65 = 90.95 \text{ mm}$$

$$\text{Ultimate Moment of Resistance} = \left[0.36 f_{ck}.b_w.x_u(d - 0.42\, x_u) + 0.45 f_{ck}(b_f - b_w).y_f.\left(d - \frac{y_f}{2}\right)\right]$$

$$(\text{M.R})_u = [0.36 \times 15 \times 275 \times 172.99 \times (550 - 0.42 \times 172.99)]$$

$$+ \left[0.45 \times 15 \times (1300 - 275) \times 90.95 \times \left(550 - \frac{90.95}{2}\right)\right]$$

$$(\text{M.R})_u = 440102582.3 \text{ N-mm}$$

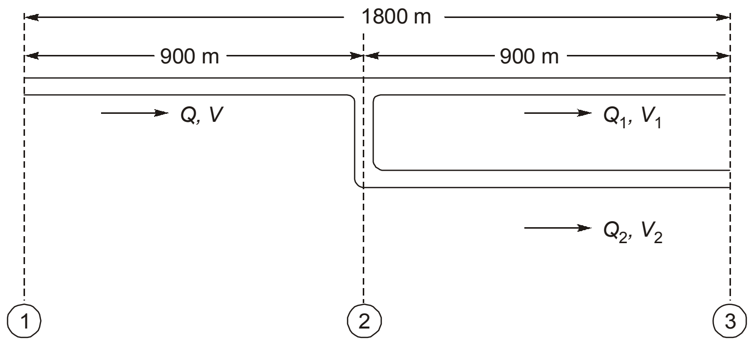
$$= 440.1026 \text{ kN-m}$$

Q.3 (c) Solution:

Neglecting minor losses, the difference in water levels i.e., 16 m, of the two reservoirs represents the head loss in friction, hence

$$16 = f\frac{L}{d}\frac{V^2}{2g} = 0.018 \times \frac{1800}{0.3} \times \frac{\left(\frac{4Q}{\pi(0.3)^2}\right)^2}{2 \times 9.81}$$

from which, $Q = 0.12055 \text{ m}^3/\text{s}$



To augment discharge by 25%, let a pipe of the same diameter be connected midway as shown. The augmented discharge

$$Q = 1.25 \times 0.12055 = 0.1507 \text{ m}^3/\text{s}$$

This discharge flows in the first half of the pipeline, thereafter it gets divided into Q_1 and Q_2 which flow in the two pipes as shown. From the continuity principle,

$$Q = Q_1 + Q_2$$

The head loss in the first half of the pipeline

$$(h_f)_{1\text{-}2} = f\frac{L_{12}}{d}\frac{V^2}{2g}; \quad V = \frac{Q}{A} = \frac{0.1507}{\left(\frac{\pi}{4}\right)(0.3)^2} = 2.1311 \text{ m/s}$$

$$= 0.018 \times \frac{900}{0.3} \times \frac{(2.1311)^2}{2 \times 9.81} = 12.50 \text{ m}$$

The remaining head (16 – 12.50 = 3.50 m) will be available for driving the flow Q_1 and Q_2 into the respectively pipelines.

Thus, $(h_f)_{2\text{-}3} = 3.50$ m, given $L_{23} = 900$ m, $d = 0.3$ m

$$\Rightarrow \quad f \times \frac{L_{23}}{d}\cdot\frac{V_1^2}{2g} = 0.018 \times \frac{900}{0.3} \times \frac{\left(\frac{4Q}{\pi d^2}\right)^2}{2 \times 9.81} = 550.4\ Q_1^2$$

From which $Q_1 = 0.07974 \text{ m}^3/\text{s}$

From the continuity relationship,

$$Q_2 = Q - Q_1 = 0.1507 - 0.07974$$
$$= 0.07095 \text{ m}^3/\text{s}$$

For this discharge to flow in the new pipeline connected in parallel to the original one, the length $L_{2\text{-}3}$ can be obtained as follows:

$$(h_f)_{2\text{-}3} = f\frac{L_{2-3}}{d}\frac{V_2^2}{2g} = f\frac{L_{2-3}}{d}\frac{4\left(4Q_2/\pi d\right)^2}{2g}$$

Substituting the known quantities

$$3.50 = 0.018\times\frac{L_{2-3}}{0.3}\times\frac{\left[\frac{4\times 0.07095}{\pi\times(0.3)^2}\right]^2}{2\times 9.81}$$

From which, $L_{2\text{-}3} = 1135.67$ m

$\therefore$ The length of the new pipe = 1135.67 m

Q.3 (d) Solution:

Limit state are the states beyond which the structure no longer satisfies the specified performance requirement. **As per IS 800: 2007, the limit states are generally grouped under.**

1. **Limit State of Strength:** Limit state of strength are associated with failure of structure under the worst combination of loading including appropriate partial factor of safety. The limit state of strength include
   (a) Loss of stability/equilibrium of structure (including the effect of sway) or any of parts including supports and foundation.
   (b) Strength limit (general yielding, formation of mechanism, rupture of structure or any of its parts of components)
   (c) Fatigue and brittle failure.

2. **Limit state of serviceability:** There are limit states beyond which specified service criteria are no longer met. These include
   (a) Deformation and deflections
   (b) Vibrations in the structure or any of its component causing discomfort to people or damages to the structure
   (c) Ponding of structures
   (d) Corrosion and durability
   (e) repairable damage due to fatigue

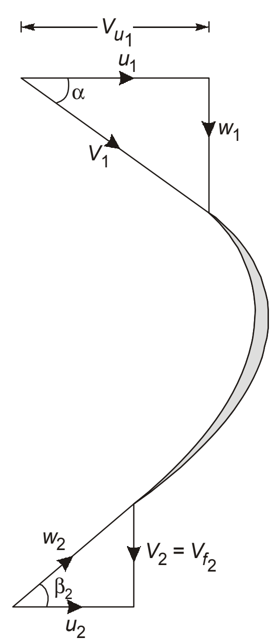
Q.4 (a) Solution:

Fibre saturation point (FSP) indicates that decreased moisture content level achieved during the drying process of timber at which only water bound in cell walls that is 'bound water' remains and all other water that is free water is removed from the cell cavities. Drying timber below FSP results in shrinkage. FSP average value generally lies between 25 to 30%.

Difference between natural seasoning and kiln seasoning:

S.No.	Natural Seasoning	Kiln Seasoning
1.	Difficult to reduce moisture level below 18%	Moisture level can be reduced to any level
2.	Seasoned timber is prone to attacks of insects and fungi.	Less proned
3.	More space required for stacking	Less space required Comparatively
4.	Slow process	Comparatively fast
5.	Result into stronger timber	Comparatively weaker product.
6.	Simple process and economical	Basically it is quite technical process and expensive

Q.4 (b) Solution:



Since the vanes are radial at inlet,

$$\beta_1 = 90°, w_1 = V_{f_1} \text{ and } V_{u_1} = u_1$$

The guide vane angle at inlet is given by,

$$\tan\alpha_1 = \frac{V_{f_1}}{u_1}$$

$$\text{The discharge } Q = \pi d_1 b_1 V_{f_1} = \pi d_2 b_2 V_{f_2}$$

$$\text{Data given: } d_1 = 2.5 \text{ m}, d_2 = 1.5 \text{ m}, b_1 = b_2 = 0.20 \text{ m}$$

$$n = 300 \text{ rpm}, Q = 10.00 \text{ m}^3/\text{s}$$

Making substitutions,

$$V_{f_1} = \frac{10.00}{\pi \times 2.5 \times 0.20} = 6.365 \text{ m/s}$$

$$V_{f_2} = \frac{10.00}{\pi \times 1.5 \times 0.20} = 10.61 \text{ m/s}$$

The peripheral velocities of blade at inlet and outlet are given by

$$u_1 = \frac{\pi d_1 n}{60} \text{ and } u_2 = \frac{\pi d_2 n}{60}$$

$$= \frac{\pi \times 2.5 \times 300}{60} = 39.27 \text{ m/s}$$

and $$u_2 = \frac{\pi \times 1.5 \times 30}{60} = 23.56 \text{ m/s}$$

Since discharge is radially outwards, $\alpha_2 = 90°$, and $V_{u_2} = 0$

Hence, $$\tan\beta_2 = \frac{V_{f_2}}{u_2}$$

$\therefore$ $$\tan\alpha_1 = \frac{6.365}{39.27} = 0.1621 \qquad \therefore \quad \alpha_1 = 9.21°$$

$\therefore$ $$\tan\beta_2 = \frac{10.61}{23.56} = 0.4503 \qquad \therefore \quad \beta_2 = 24.24°$$

The guide vane angle to be set at the inlet, $\alpha_1 = 9.21°$, and the blade angle at the outlet = 24.24°.

Q.4 (c) Solution:

Size of foundation

Column load, $P = 1500$ kN

Let weight of the foundation, $P_f = 10\%$ of column load $(P) = 0.1 \times 1500 \text{ kN} = 150 \text{ kN}$

$\therefore$ Total load, $P_t = 1500 + 150 \text{ kN} = 1650 \text{ kN}$

Area of footing required, $$A = \frac{1650}{160} = 10.31\, m^2$$

Note: Do not use factored load of 1.5 × 1650 kN here, since safe bearing capacity of soil itself takes into account the factor of safety.

Assume $B = 2.5$ m

$\therefore$ $$L = \frac{A}{B} = \frac{10.31}{2.5} = 4.12 \text{ m} = 4.2 \text{ m (say)}$$

$\therefore$ Area provided, $A = 4.2 \times 2.5 \text{ m}^2 = 10.5 \text{ m}^2 > 10.31 \text{ m}^2$ (OK)

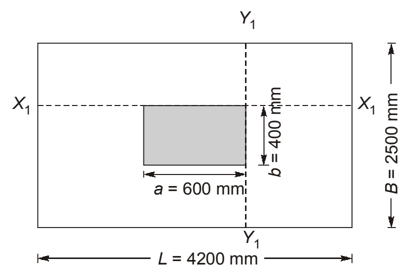
Net soil pressure

$$w_0 = \frac{P}{A} = \frac{1500}{10.5} = 142.86 \text{ kN/m}^2$$

$$< 160 \text{ kN/m}^2 \quad \text{(OK)}$$

Factored soil pressure, $w_{u0} = 1.5 \times 142.86 = 214.29 \text{ kN/m}^2$

Depth of foundation from bending moment criterion



The critical section for BM will be at the face of the column.

Moment at section X_1-X_1, $M_{ux} = w_{u0}\left(\frac{B-b}{2}\right)\left(\frac{B-b}{4}\right)$

$$= 214.29 \times \frac{(2.5-0.40)^2}{8} = 118.13 \text{kNm}$$

Moment at section Y_1-Y_1, $M_{uy} = w_{u0} \times \left(\frac{L-a}{2}\right)\left(\frac{L-a}{4}\right)$

$$= 214.29 \times \frac{(4.2-0.6)^2}{8} = 347.15 \text{ kNm}$$

$\therefore$ Maximum moment = 347.15 kNm

$$Q = 0.36 f_{ck} \times 0.46 \times (1 - 0.42 \times 0.46)$$

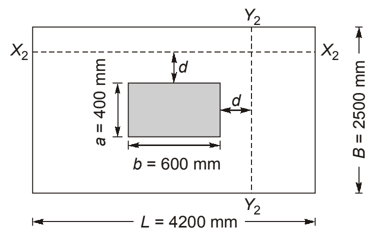
$$= 0.36 \times 25 \times 0.46\,(1 - 0.42 \times 0.46)$$

$$= 3.34$$

Depth of foundation, $d = \sqrt{\frac{M_{uy}}{QB}} = \sqrt{\frac{347.15 \times 10^6}{3.34 \times 1000}} = 322.39 \text{ mm}$

Take d = 330 mm

Check for one way shear



Critical section for one way shear will be at a distance of 'd' from the face of the column

Maximum shear force at section Y_2-Y_2

$$V_{uy} = w_{u0} \times 1\,\text{m} \times \left[\frac{L-b}{2} - d\right]$$

$$= 214.29 \times 1\left[\frac{4.2-0.6}{2} - 0.33\right]$$

$$= 315 \text{ kN}$$

Nominal shear stress, $\tau_v = \dfrac{V_{uy}}{Bd} = \dfrac{315 \times 10^3}{1000 \times 330} = 0.955 \text{ N/mm}^2$

$$k = 1.0 \text{ for } d > 300 \text{ mm}$$

$$k\tau_c = 1 \times 0.29 \text{ N/mm}^2 < \tau_v\ (= 0.955 \text{ N/mm}^2) \text{ Fails}$$

Revised depth of the footing,

$$k\tau_c = 0.29 = \frac{V_{uy}}{Bd}$$

$\therefore$ $$d = \frac{V_{uy}}{B \times 0.29} = \frac{315 \times 10^3}{1000 \times 0.29} = 1086.20 \text{ mm}$$

Average of 330 mm and 1086.20 mm $=$

$\dfrac{330 + 1086.20}{2} = 708.10 \text{ mm} \approx 700 \text{ mm}$ (say)

Check for $d = 700$ mm

$$V_{uy} = 214.29 \times \left[\frac{4.2-0.6}{2} - 0.7\right] = 235.72 \text{ kN}$$

$$\tau_v = \frac{V_{uy}}{Bd} = \frac{235.72 \times 10^3}{1000 \times 700} = 0.34 \text{ N/mm}^2 > \tau_c\left(= 0.29 \text{ N/mm}^2\right) \text{ Fails}$$

Try $d = 800$ mm

$$V_{uy} = 214.29 \times \left[\frac{4.2-0.6}{2} - 0.80\right] = 214.29 \text{ kN}$$

$$\tau_v = \frac{V_{uy}}{Bd} = \frac{214.29 \times 10^3}{1000 \times 800} = 0.268 \text{ N/mm}^2 < 0.29 \text{ N/mm}^2 \quad \text{(OK)}$$

$\therefore$ Adopt effective depth, d = 800 mm

Check for two-way shear (Punching shear)

Net punching shear stress developed,

$$\tau_{vp\text{ (developed)}} = \frac{P_u - w_{u_0}(a+d)(b+d)}{2[(a+d)+(b+d)]d}$$

$$= \frac{(1.5 \times 1500) - 214.29(0.4+0.8)(0.6+0.8)}{2[(0.4+0.8)+(0.6+0.8)] \times 0.8}$$

$$= \frac{225-360}{4.16}$$

$$= 454.33 \text{ kN/m}^2 = 0.454 \text{ N/mm}^2$$

$$\tau_{vp\text{ (permissible)}} = 0.25\sqrt{f_{ck}} \times k_\beta$$

$$\left[k_\beta = 0.5 + \frac{b}{a} = 0.5 + \frac{400}{600} = 1.167 \ngtr 1.0\right]$$

$$= 0.25\sqrt{25} \times 1.0$$

$$= 1.25 \text{ N/mm}^2 \qquad \text{(Hence SAFE)}$$

Thus, taking 60 mm effective cover,

Total depth = 800 + 60

= 860 mm

Q.5 (a) Solution:

Let minimum thickness of plate $t_{min.}$ is = 8 mm

Dead load, P_{DL} = 20 kN

Live load, P_{LL} = 60 kN

Total service load, P = $P_{DL} + P_{LL}$

= 20 + 60 = 80 kN

Design factored load, P_1 = $\gamma_f \times P$

= 80 × 1.5 = 120 kN

Strength of bolt will be minimum of two:

(a) Shearing strength of a bolt = $(1 \times A_{net}) \times \dfrac{f_{ub}}{\sqrt{3} \times \gamma_{mb}}$

$$= \left[0.78 \times \frac{\pi}{4} \times (16)^2\right] \times \frac{400}{\sqrt{3} \times 1.25} = \mathbf{28.974\ kN}$$

(b) Bearing strength of Bolt $= 2.5\, k_b \times d \times t_{min} \times \dfrac{f_{up}}{\gamma_{m1}}$

$$K_b \text{ is minimum of } \begin{bmatrix} \dfrac{e}{3d_0} & = \dfrac{1.5d_0}{3d_0} = 0.5 \\ \left(\dfrac{p}{3d_0} - 0.25\right) & = \left(\dfrac{2.5 \times 16}{3 \times 18} - 0.25\right) = 0.49 \\ \left(\dfrac{f_{ub}}{f_{up}}\right) & = \dfrac{400}{410} = 0.98 \\ 1 & \end{bmatrix}$$

$\therefore$ $k_B = 0.49$

$\therefore$ Bearing strength of bolt $= 2.5 \times 0.49 \times 16 \times 8 \times \dfrac{410}{1.25} = 51.43$ kN

$\therefore$ Strength of bolt is minimum of two = 28.974 kN

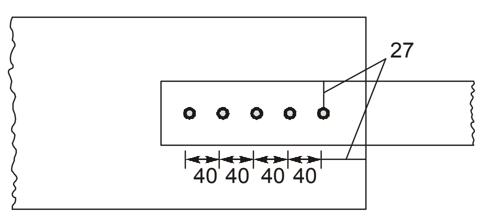
$\therefore$ no. of bolts required, $n = \dfrac{120}{28.974} = 4.14 \simeq 5$ bolts

Pitch distance provided, $p = 2.5\, d$

$= 2.5 \times 16 = 40$ mm

end distance $= 1.5\, d_0$

$= 1.5 \times 18 = 27$ mm



(All dimensions are in mm)

edge distance $= 1.5\, d_0$

$= 1.5 \times 18 = 27$ mm

$$\begin{bmatrix} \text{Width of plate provided, } B = 27 \times 2 = 54 \simeq 55 \text{ mm} \\ \text{thickness of plate, } t = 8 \text{ mm} \\ \text{no. of bolts provided, } n = 5 \text{ bolts} \end{bmatrix}$$

Q.5 (b) Solution:

When the bending moment required to be resisted is more than the moment of resistance of a balanced section of singly reinforced beam of given size, there are two alternatives:

(i) To use an **over-reinforced** section.

(ii) To use doubly reinforced section.

An over reinforced section is always uneconomical and also undesirable because of sudden failure probability. Also the increase in the moment of resistance is not in proportion to the increase in the area of tensile reinforcement. The reason behind this is that the concrete, having reached maximum allowable stress, cannot take more additional load without adding compression steel. The other alternative is to provide reinforcement in the compression side of the beam and thus to increase the moment of resistance of the beam beyond that of a balanced section.

Doubly reinforced sections are also useful in following situations:

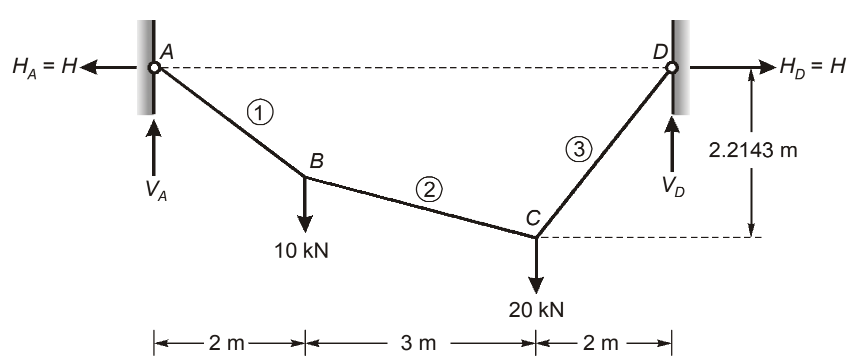
(i) Where the members are subjected to probable reversal of external loads and thereby the bending moment in the section reverses, such as in concrete piles etc.

(ii) When the members are subjected to loading, eccentric to either side of the axis, such as in columns subjected to wind loads.

(iii) When the members are subjected to accidental lateral loads, shock or impact.

The steel reinforcement provided in the compression zone is subjected to compressive stress. However, concrete undergoes creep strains due to continued compressive stress, with the result that the strain in concrete goes on increasing with time. This increases compressive strain in steel in addition to creep strain in compressive steel. Thus the total compressive strain in compressive steel will be much greater than the strain in surrounding concrete due to flexure alone. Thus, compressive steel takes up all the additional compressive stresses beyond the permissible compressive stress for concrete making the section safe against failure in flexure.

Q.5 (c) Solution:



Support reactions:

Considering the overall free-body.

$$\Sigma M_A = 0$$

$$\Rightarrow \quad -(V_D)(7) + (20)(5) + (10)(2) = 0$$

$$\Rightarrow \quad V_D = 17.143 \text{ kN} \uparrow$$

$$\Sigma F_y = 0$$

$$\Rightarrow \quad V_A + V_D - 10 - 20 = 0$$

$$\Rightarrow \quad V_A = 12.857 \text{ kN} \uparrow$$

Considering the free-body of the segment DC

$$M_C = 0$$

$$\Rightarrow \quad (V_D)(2) - H(2.2143) = 0$$

$$\Rightarrow \quad (17.143)\,(2) - H(2.2143) = 0$$

$$\Rightarrow \quad H = 15.484 \text{ kN}$$

Cable Tensions: (N_1, N_2 and N_3 in the three segments)

Let V_i and H be the vertical shear and the horizontal thrust in the i^{th} segment of the cable. The values of V_i are easily obtainable by considering vertical force equilibrium in free bodies of the three segments.

Accordingly, applying $\Sigma F_y = 0,$

$$V_1 = V_A = 12.857 \text{ kN}$$

$$V_2 = V_A - 10 = 2.857 \text{ kN}$$

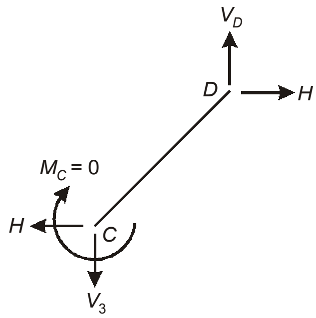
$$V_3 = V_D = 17.143 \text{ kN}$$

The axial tension in the i^{th} segment is given by

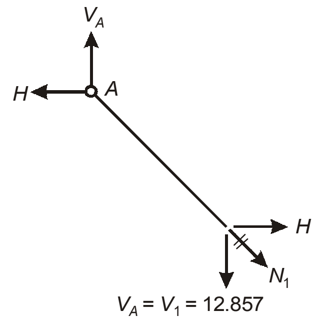
$$N_i = \sqrt{V_i^2 + H^2}$$

$$\Rightarrow \quad N_1 = \sqrt{(12.857)^2 + (15.484)^2} = 20.126 \text{ kN}$$

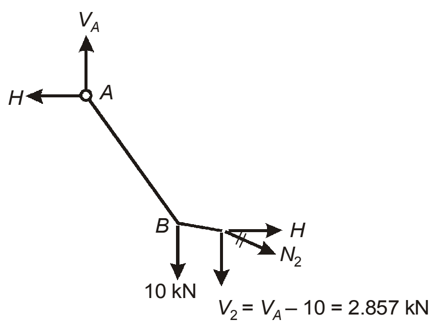
$$\Rightarrow \quad N_2 = \sqrt{(2.857)^2 + (15.484)^2} = 15.745 \text{ kN}$$



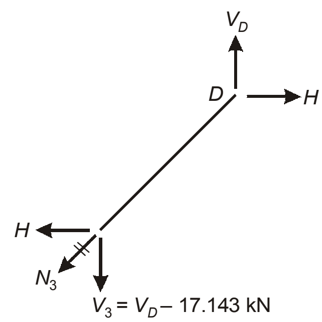
(a) Free body DC[+]



(b) Section through segment 1



(c) Section through segment 2



(d) Section through segment 3

$$\Rightarrow \qquad N_3 = \sqrt{(17.143)^2 + (15.484)^2} = 23.101\text{ kN}$$

Sag at B

Considering the free-body of the segment AB,

$$M_B = 0$$

$$\Rightarrow (V_A = 12.857)(2) - (H = 15.484)(y_B) = 0$$

$$\Rightarrow \qquad y_B = 1.6607 \text{ m}$$

Q.5 (d) Solution:

$$\text{Total filtration, } F_p = \int_0^t f_p dt$$

where, $f_p = 0.5 + 1.2e^{-0.5t}$

In 4 hours cumulative infiltration

$$F_p = \int_0^t \left(0.5 + 1.2e^{-0.5t}\right) dt$$

$$= \left[0.5t - \frac{1.2e^{-0.5t}}{(0.5)}\right]_0^4$$

$$= (0.5 \times 4) - 0 - 2.4\left\{e^{-(0.5\times4)} - \left(e^0\right)\right\}$$

$$= 2 - 2.4\ (0.13533 - 1) = 4.075 \text{ cm}$$

Average infiltration capacity rate, $f_{avg} = \dfrac{4.075}{4} = 1.02 \text{ cm/hr}$

Q.6 (a) Solution:

When loading is applied over a certain period of time, then all time calculations are done by assuming the middle of the construction period as datum. Thus, in this problem, the building was constructed between July 1987 and July 1989. Hence, the datum would be July 1988. All further timings would be calculated from this datum.

Hence, 113 mm settlement observed in July 1992 would be after a period of 4 years from this datum (July 1988).

Settlement S_1 after t_1 (4 years) = 113 mm = 0.113 m

Total settlement (S i.e. ΔH) = 360 mm = 0.360 m

Settlement in July 1997, i.e. after 9 years from datum = S_2 at t_2 (9 years) = ?

Now, U_1 = degree of settlement after t_1 (4 years) time = $\dfrac{0.113 \text{ m}}{0.360 \text{ m}} = 0.314$

U_2 = degree of settlement after t_2 (9 years) time = $\dfrac{S_2}{0.360 \text{ m}} = 2.778\, S_2$

Now, from equation

$$T_v = \frac{\pi}{4}U^2 \quad (\text{for } U \le 0.60)$$

$$\therefore \quad T_{v_1} = \frac{\pi}{4}(0.314)^2 \quad \text{...(i)}$$

And

$$T_{v_2} = \frac{\pi}{4}(2.778\, S_2)^2 \quad \text{...(ii)}$$

(assuming U_2 to be ≤ 0.6)

Also,

$$T_v = \frac{C_v}{d^2}\cdot t$$

$$\therefore \quad T_{v_1} = \frac{C_v}{d^2}\cdot t_1,\ T_{v_2} = \frac{C_v}{d^2}\cdot t_2$$

$$\therefore \quad \frac{T_{v_1}}{T_{v_2}} = \frac{t_1}{t_2}$$

or $$\frac{\frac{\pi}{4}(0.314)^2}{\frac{\pi}{4}(2.778\,S_2)^2} = \frac{4 \text{ years}}{9 \text{ years}}$$

or $$\frac{(0.314)}{2.778\,S_2} = \sqrt{\frac{4}{9}} = \frac{2}{3}$$

∴ $$S_2 = \frac{3}{2}\times\frac{0.314}{2.778} = 0.1698 \text{ m}$$

Check for U_2 :

So, $$U_2 = 2.778\ S_2 = 2.778 \times 0.1696$$
$$= 0.471$$

(which is less than 0.6, and hence our assumption is OK)

Hence, the expected settlement after 9 years, i.e. in July 1997

$$= 0.1696 \text{ m} = 169.6 \text{ mm}$$

Q.6 (b) Solution:

$$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w = \frac{2.7+0.8}{1+0.8}9.81 = 19.08 \text{ kN/m}^3$$

$$\gamma' = 19.08 - 9.81 = 9.27 \text{ kN/m}^3$$

$$i = 45°; \quad \phi = 15°$$

(i) Submerged case: For $i = 45°; \quad \phi = 15°; \quad S_n = 0.083$

$$F_c = \frac{c}{S_n\gamma' H} = \frac{14}{0.089\times 9.27\times 5} = 3.64$$

(ii) Drawdown case: Taking $F_\phi = 1$ and $\phi_m = \phi$

$$\phi_w = \frac{\gamma'}{\gamma_{sat}}\phi_m = \frac{\gamma'}{\gamma_{sat}}\phi = \frac{9.27}{19.08}\times 15 \simeq 7.3°$$

For $i = 45°$ and $\phi = 7.3$, $S_n = 0.122$

∴ $$F_c = \frac{c}{\gamma_{sat}HS_n} = \frac{14}{19.08\times 5\times 0.122} = 1.2$$

Q.6 (c) Solution:

Constituents of a good brick earth :

1. Alumina : 20% to 30%
2. Silica : 50% to 60%

3. Lime : not exceeding 5%
4. Oxides of iron : 5% to 6%
5. Magnesia : Small quantity.

Properties of a first class brick :

- Burnt uniformly in kilns
- Uniform color throughout the brick.
- The surfaces and edges of the bricks are sharp, square, smooth and straight.
- These are used for superior work of permanent nature.
- They comply with all the qualities of good bricks.

Test for bricks :

(1) Absorption :
  - A brick is taken and it is weighted dry. It is then immersed in water for a period of 16 hours. It is weighed again and the difference in weight indicates the amount of water absorbed by the brick.
  - It should not exceed 20% of weight of dry brick.

(2) Crushing strength :
  - It is found out by placing it in compression testing machine. It is pressed till it breaks.
  - As per IS : 1077 - 1970, the minimum crushing strength of bricks is 3.50 N/mm^2.

(3) Hardness :
  - A scratch is made on brick surface with finger nail. if no impression is left on the surface, the brick is treated to be sufficiently hard.

(4) Presence of soluble salts:
  - Brick is immersed in water for 24 hours. It is then taken out and allowed to dry in shade. The absence of grey or white deposits on its surface indicates absence of soluble salts.

(5) Shape and Size :
  - Brick is closely inspected. It should be of standard size and its shape should be truly rectangular with sharp edges.

(6) Soundness :
  - Two bricks are struck with each other. The bricks should not break and clear ringing sound should be produced.

(7) Structure :

- A brick is broken and its structure is examined. It should be homogeneous, compact and free from any defects such as holes, lumps, etc.

Q.6 (d) Solution:

Thermal power plants, generally use coal as the fuel. Fuel oil is also sometimes used as fuel. Use of fuel gas is the most modern advancement in this field.

If coal is used, pollutants like fly ash, sulphur dioxide, and nitrogen oxides are produced on a large scale. In case of oil, however, only sulphur dioxide and nitrogen oxides are produced, as the major pollutants. Coal, thus proves to be a worse fuel, and is an important source responsible for particulate air pollution. The amount of fly ash and sulphur dioxide produced by the coal depends upon the quality, and sulphur and ash content in it.

The environmental impacts can be enumerated in the following way:

1. Air pollution: Almost all thermal power plants heavily pollute the air of surrounding region. Around the coal based plants the ambient concentrations of sulphur dioxide, oxides of nitrogen and SPM are high. Two other gases, carbon dioxide and ozone are emitted. The high amount of carbon dioxide emission from thermal power plants contribute to global warming leading to climate change. Mercury vapour is also emitted with these gases and its toxicity has far reaching consequences on all life forms.
2. Noise Pollution: The exposure of employees to high noise levels is very high. Increased transportation activities due to the operation of the power plants lead to increase in noise levels in the adjacent localities.
3. Land and Soil Pollution : Large amount of land is used to dispose flyash from the coal based plants. Due to this there is change in natural soil properties. It becomes more alkaline due to the alkaline nature of fly ash.
4. Water Pollution: Water slurry is used to take the ash from the power plant to the ash pond for disposal. Water slowly seeps into the ground while carrying with it the ash leachate (Lye). This water contains harmful heavy metals like Boron.

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