MADE EASY

India's Best Institute for IES, GATE & PSUs

Detailed Solutions

BPSC Main Exam 2019

ASSISTANT ENGINEER

CIVIL ENGINEERING

Subjective Paper-I

Test 5

Detailed Solutions

Q.1 (a) Solution:

Given: Length, L = 4 m = 4 × 10^3 mm; Force, P_1 = 50 kN = 50 × 10^3 N; Force, P_2 = 70 kN = 70 × 10^3 N and modulus of elasticity, E = 120 GPa = 120 × 10^3 N/mm^2

From the geometry of the figure, we find that diameter of the bar at B.

$$d_B = 20 + (40 - 20) \times \frac{1}{4} = 25 \text{ mm}$$

Similarly, diameter of the bar at C.

$$d_C = 25 + (40 - 20) \times \frac{2}{4} = 35 \text{ mm}$$

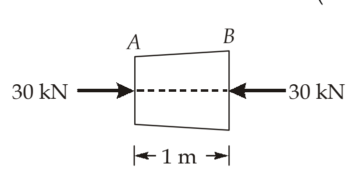
For the sake of simplification, the forces of 50 kN acting at B may be split up into two forces of 30 kN and 20 kN respectively. Similarly the force of 70 kN acting at C may also be split up into two forces of 20 kN and 50 kN respectively.

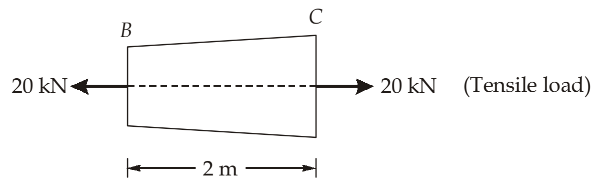
Now it will be seen that bar AB subjected to a compressive load of 30 kN and part BC is subjected to a tensile load of 20 kN and part CD is subjected to a compressive load of 50 kN as shown in figure.

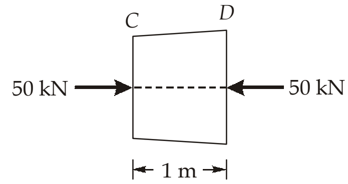
We know, that shortening of the bar AB due to a compressive force of 30 kN.

Axial compression for tapered bar AB,

$$\delta l_1 = \frac{4P_A \times l_{AB}}{\pi E d_A \cdot d_B} = \frac{4 \times (30 \times 10^3) \times (1 \times 10^3)}{\pi \times (120 \times 10^3) \times 20 \times 25} = 0.64 \text{ mm}$$







Similarly elongation of the bar BC due to a tensile load of 20 kN.

$$\delta l_2 = \frac{4P_B \times l_{BC}}{\pi E d_B \cdot d_C} = \frac{4\times(20\times10^3)\times(2\times10^3)}{\pi\times(120\times10^3)\times25\times35} = 0.48 \text{ mm}$$

and shortening of the bar CD due to a compressive load of 50 kN

$$\delta l_3 = \frac{4P_C \times l_{CD}}{\pi E d_C \cdot d_D} = \frac{4\times(50\times10^3)\times(1\times10^3)}{\pi(120\times10^3)\times35\times40} = 0.38 \text{ mm}$$

$\therefore$ Change in length, $\delta l = \delta l_1 - \delta l_2 + \delta l_3$ (Assuming compression of bar as positive)

$= 0.64 - 0.48 + 0.38 = 0.54$ mm (decrease)

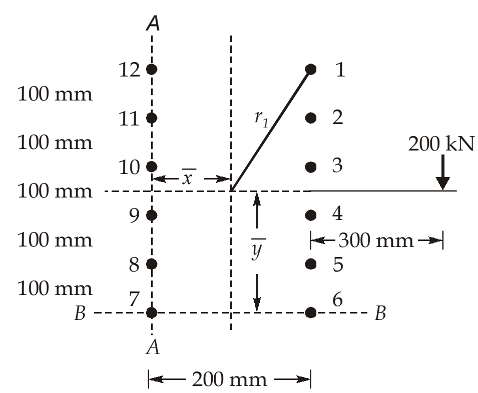
Q.1 (b) Solution:

Assuming the diameter of all the rivets is same.

Now locating the centre of gravity of the rivet group.

Taking 100 kgf = 1 kN

Taking moments about line AA, we get



$$\bar{x} = \frac{6\times a\times 200}{12a}$$

[where a = area of any rivet]

$\Rightarrow$ $\bar{x} = 100$ mm

Taking moments about line BB, we get

$$\bar{y} = \frac{2a\times100+2a\times200+2a\times300+2a\times400+2a\times500}{12a}$$

$\bar{y} = 250$ mm

Force in each rivet due to direct shear,

$$F_D = \frac{200}{12} = 16.67 \text{ kN}$$

If the distance of rivet 1 from the CG of rivet group is r_1, then

$$r_1 = \sqrt{(100)^2+(250)^2} = 269.26 \text{ mm}$$

similarly $r_2 = \sqrt{(100)^2+(150)^2} = 180.28$ mm and $$r_3 = \sqrt{(100)^2 + (50)^2} = 111.80 \text{ mm}$$

As the rivet group is symmetrical about the *CG*, we have

$$r_1 = r_6 = r_7 = r_{12} = 269.26 \text{ mm}$$
$$r_2 = r_5 = r_8 = r_{11} = 180.28 \text{ mm}$$
$$r_3 = r_4 = r_9 = r_{10} = 111.80 \text{ mm}$$

$$\therefore \quad \sum_{i=1}^{n} r_i^2 = 4\left[r_1^2 + r_2^2 + r_3^2\right]$$
$$= 4[(269.26)^2 + (180.28)^2 + (111.80)^2]$$
$$= 470004.26 \text{ mm}^2$$

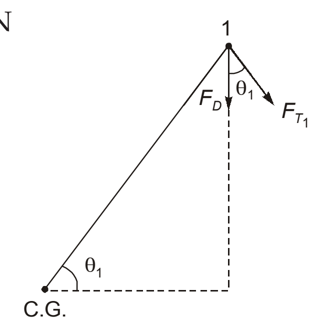
Now, force in a rivet due to torsional moment,

$$F_T = \frac{(Pe)r_i}{\sum_{i=1}^{n} r_i^2}$$

Here $$P = 200 \text{ kN}; e = 300 + 100 = 400 \text{ mm}$$

$$r_1 = 269.26 \text{ mm}; \sum_{i=1}^{n} r_i^2 = 470004.26 \text{ mm}^2$$

$$\therefore \quad F_{T_1} = \frac{(Pe)r_1}{\sum_{i=1}^{n} r_i^2} = \frac{200 \times 400 \times 269.26}{470004.26} = 45.83 \text{ kN}$$



If θ_1 is the angle between F_{D_1} and F_{T_1} then $\cos\theta_1 = \frac{100}{269.26} = 0.3714$

$\therefore$ Resultant force in rivet 1

$$= \sqrt{(F_{D_1})^2 + (F_{T_1})^2 + 2 \times F_{D_1} \times F_{T_1} \times \cos\theta_1}$$

$$= \sqrt{(16.67)^2 + (45.83)^2 + 2 \times 16.67 \times 45.83 \times 0.3714}$$

$$= 54.27 \text{ kN}$$

It may be noted that rivet 1 is the most critical rivet in the connection. It carries the maximum force among all the rivets.

Q.1 (c) Solution:

(i) Unconsolidated undrained test (U-U Test): It is a quick test and may completed in 5-10 minutes. In this test water is neither allowed to leave the soil during consolidation state (confining stage) nor shear stage (deviator stage).

In this test no significant change in volume is expected. Such test are suitable for low permeability soil such as clay with fast loading rate.

Example: Construction of building over saturated clay.

(ii) Consolidated Undrained Test (C-U Test): During the first stage of confining pressure, flow of water from soil is permitted i.e. drainage is permitted, then consolidation will take place. Whereas during vertical shear loading (shear stage) drainage is not permitted.

Example: Stability analysis of earthen dam during sudden draw down.

(iii) Consolidation Drained Test (C-D Test): In C-D test, the drainage is permitted during confining pressure stage and shear stage both. This test is most time taking and for some soil may take several weeks. Since pore water is allowed to flow out, Hence significant volume change in soil may occur.

This test is suitable for soil with high permeability and slow loading rate.

Example: Stability of retaining walls having sandy fills.

(iv) Unconfined Compression Test (U-C test): It is an special case of triaxial test in which confining pressure is zero, No rubber membranes is provided and only compressive stress is applied.

This test is suitable only for moist (saturated) clay and silt.

Section - A

Q.2 Solution:

Let V_A and V_D be the vertical reactions

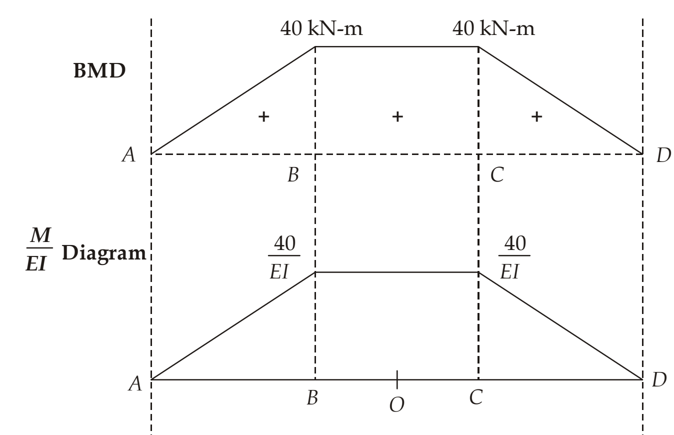
$$\Sigma F_y = 0$$

$\Rightarrow$ $$V_A + V_D = 40 \text{ kN}$$

From symmetry, $$V_A = V_D$$

$\Rightarrow$ $$V_A = V_D = 20 \text{ kN}$$

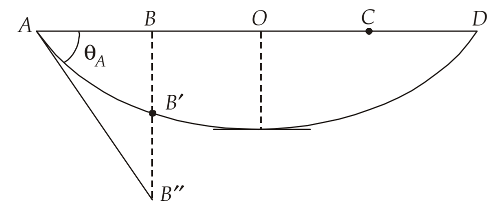
BM Diagram: Sign Convention: Sagging B.M. = Positive B.M. $\boxed{+}$



Using Mohr's I Theorem,

$$\theta_{AO} = \text{Area of } \frac{M}{EI} \text{ diagram between } A \text{ and } O.$$

$$\theta_O - \theta_A = \frac{1}{2}\times\frac{40}{EI}\times 2 + 1\times\frac{40}{EI}$$



or, $$\theta_O - \theta_A = \frac{80}{EI}$$

or, $$\theta_A = -\frac{80}{EI}$$

'–' sign signifies Anticlockwise direction of rotation

$B''B' = \delta_{B/A}$ = deflection of B w.r.t. tangent at A.

Using Mohr's II Theorem,

$\delta_{B/A}$ = Moment of area of $\frac{M}{EI}$ diagram between A and B about B.

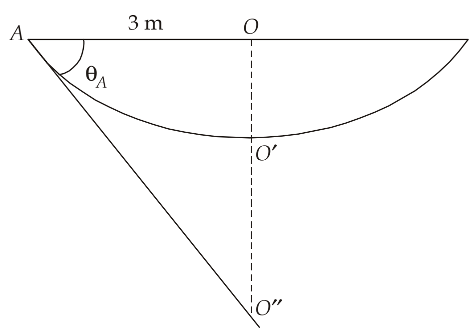
$$= \frac{1}{2} \times \frac{40}{EI} \times 2 \times \frac{2}{3} = \frac{80}{3EI}$$

$$BB'' = \theta_A \times L_{AB} = \frac{80}{EI} \times 2 = \frac{160}{EI}$$

Deflection of B, $$\delta_B = BB'' - B''B'$$

$$= \frac{160}{EI} - \frac{80}{3EI} = \frac{400}{3EI}$$ **{downward deflection}**

Similarly,



Using Mohr's I Theorem,

$O'O'' = \delta_{O/A}$ = Moment of Area of $\frac{M}{EI}$ diagram between A and O about O

$$= \frac{1}{2} \times \frac{40}{EI} \times 2 \times \left(1 + \frac{2}{3}\right) + \frac{40}{EI} \times 1 \times 0.5 = \frac{200}{3EI} + \frac{20}{EI} = \frac{260}{3EI}$$

$$OO'' = \theta_A \times L_{OA} = \frac{80}{EI} \times 3 = \frac{240}{EI}$$

$\therefore$ **Deflection of Mid point,** $$\delta_0 = OO'' - O'O'' = \frac{240}{EI} - \frac{260}{3EI} = \frac{460}{3EI}$$

3. Solution:

(i) Fixed end moments,

$$M_{FBC} = M_{FCD} = -\frac{20 \times 4}{8} = -10 \text{ kN-m}$$

$$M_{FCB} = M_{FDC} = +\frac{20 \times 4}{8} = +10 \text{ kN-m}$$

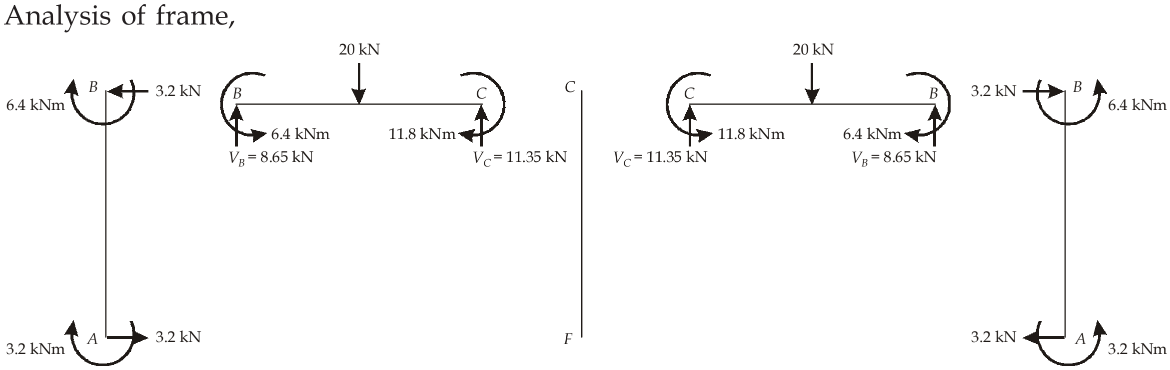
(ii) Distribution factors

Joint	Member	Relative Stiffness	Sum	D.F.
B (or D)	BA (or DE)	$\frac{2I}{3}=\frac{8I}{12}$	$\frac{12.5I}{12}$	0.64
	BC (or DC)	$\frac{1.5I}{4}=\frac{4.5I}{12}$		0.36
C	CB	$\frac{1.5I}{4}=\frac{3I}{8}$	$\frac{9}{8}I$	0.333
	CF	$\frac{3}{4}\times\frac{I}{2}=\frac{3I}{8}$		0.333
	CD	$\frac{1.5I}{4}=\frac{3I}{8}$		0.333

(iii) Moment distribution

A	B		C			D		E	
	BA	BC	CB	CF	CD	DC	DE		
	0.64	0.36	0.333	–	0.333	0.36	0.64		
–	–	–10.0	+10.0	–	–10.0	+10.0	–	–	F.E.M.
–	+6.40	+3.60	–	–	–	–3.60	–6.40	–	B.
+3.20	–	–	+1.80	–	–1.80	–	–	–3.20	C.O.
–	–	–	–	–	–	–	–	–	B.
+3.20	+6.40	–6.40	+11.8	0	–11.8	+6.40	–6.40	–3.20	Final moments

Analysis of frame,



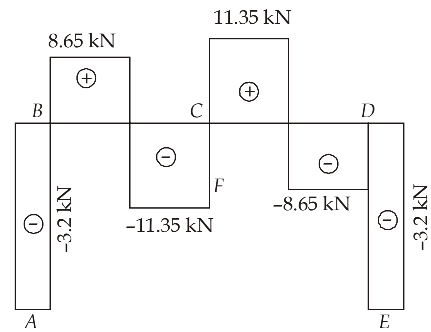
(i) $$H_A = H_B = \frac{6.4+3.2}{3}(\text{due to moment}) = 3.2 \text{ kN}$$

(ii) $$V_B = \frac{20}{2} = 10 \text{ kN (due to load)} - \frac{(11.8-6.4)}{4}(\text{due to moment}) = 8.65 \text{ kN}$$

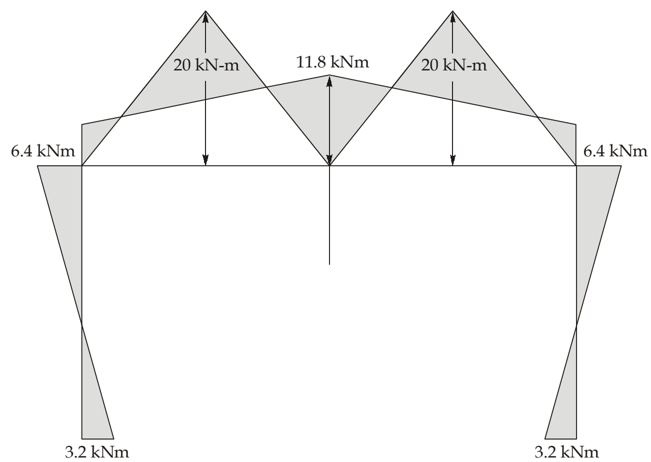
(iii) $$V_C = \frac{20}{2} = 10 \text{ kN (due to load)} + \frac{(11.8-6.4)}{4}(\text{due to moment}) = 11.35 \text{ kN}$$

Similarly, V_D, H_D and H_E can be calculated.

SFD

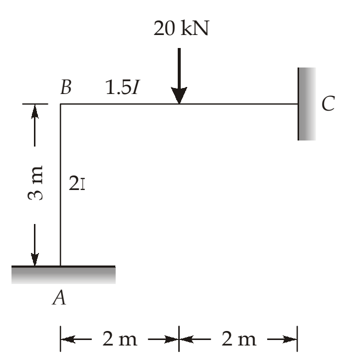


The bending moment diagram and the deflected shape of the structure have been shown in figure.



Alternatively,

Since structure is symmetric about column *CF* so we can assume structure as



$$FEM_{BC} = -10 \text{ kNm}, FEM_{CB} = 10 \text{ kNm (as shown earlier)}$$

Moment distribution factor's

Joint	Member	Relative Stiffness	Sum	DF
B	BA	$\frac{2I}{3}=\frac{8I}{12}$	$\frac{12.5I}{12}$	0.64
	BC	$\frac{1.5I}{4}=\frac{4.5I}{12}$		0.36

Moment distribution

A	B		C
AB	BA	BC	CB
∞	0.64	0.36	∞
–	–	–10	10
	6.4	3.6	
3.2			1.8
3.2	6.4	–6.4	11.8

Similar to previous distribution only.

Section - B

Q.4 Solution:

Given that, P = 1200 kN, Area of section = 2 × 5366 = 10732 mm^2

$$\therefore \quad \sigma_{ac} \text{ provided} = \frac{1200\times10^3}{10732} = 111.82 \text{ N/mm}^2$$

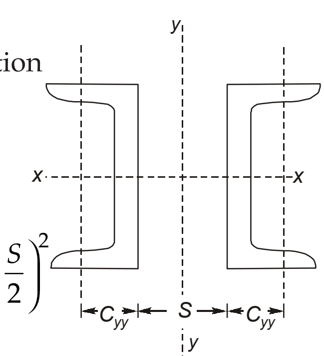
$$\lambda \text{ (for } \sigma_{ac} = 111.82 \text{ N/mm}^2) = 60 + \left(\frac{100-60}{126-82}\right)\times(126-111.82) = 72.9$$

$$\text{But} \quad \lambda \le \frac{l_{eff}}{r_{\min}}$$

$$\Rightarrow \quad r_{\min} \ge \frac{l_{eff}}{\lambda} \ge \frac{5.8\times10^3}{72.9} \ge 79.56 \text{ mm}$$

$$\therefore \quad I_{\min} \ge \text{Area of section} \times (r_{\min})^2 = 2\times5366\times(79.56)^2 = 67.93\times10^6 \text{ mm}^4$$

Let the spacing between the two channel section be S.



For most efficient use of material I_{xx} of combination = I_{yy} of combination

In this case the compression member will carry maximum force

$$I_{xx} \text{ of combination} = 2\times I_{xx} \text{ of single section}$$
$$= 2\times100.08\times10^6 = 200.16\times10^6 \text{ mm}^4$$

$$I_{yy} \text{ of combination} = 2\times I_{yy} \text{ of one section} + 2\times A\times\left(24.4+\frac{S}{2}\right)^2$$

But for the loading of 1200 kN, the design will be economical when

$$I_{\min} = I_{yy} \text{ of combination}$$

$$\therefore \quad 2\times4.306\times10^6 + 2\times5366\times\left(24.4+\frac{S}{2}\right)^2 = 67.93\times10^6$$

$$\Rightarrow \quad 24.4+\frac{S}{2} = 74.35$$

Q.5 Solution:

Step-1: Loads

Let overall depth of beam, D = 650 mm

and width of beam, B = 300 mm

Self weight of beam, = 25(0.3) (0.65) = 4.875 kN/m

Live load = 30 kN/m (Given)

∴ Total load = 34.875 kN/m

∴ Factored load w = 1.5 × 34.875 = 52.3125 kN/m

Step-2: Calculate design moment and design shear

Design moment $M_u = \frac{wl^2}{8} = 52.3125 \times \frac{(8)^2}{8} = 418.5 \text{ kNm}$

Design shear force $V_u = \frac{wl}{2} = 52.3125 \times \frac{8}{2} = 209.25 \text{ kN}$

Step-3: Calculate limiting MOR for singly reinforced beam section

For Fe 415, limiting moment of resistance

$$M_{u\,lim} = 0.138\, f_{ck} bd^2$$

Let effective cover = 50 mm

Effective depth of beam d = 650 – 50 mm = 600 mm

$$\therefore \quad M_{u\,lim} = 0.138\,(20)\,(300)\,(600)^2$$
$$= 298.08 \text{ kNm} < M_u\ (= 418.5 \text{ kNm})$$

Thus doubly reinforced beam section is required.

Step-4: Calculate amount of compression reinforcement required.

$$\Delta M_u = M_u - M_{u\,lim}$$
$$= 418.5 - 298.08 \text{ kNm} = 120.42 \text{ kNm}$$

$$\therefore \quad f_{sc} = \frac{0.0035\left(x_{u\lim} - d'\right)}{x_{u\lim}} \times E_s$$

$$= \frac{0.0035\left(0.48 \times 600 - 50\right)}{0.48 \times 600} \times 2 \times 10^5 \text{ N/mm}^2$$

$$= 578.47 \text{ N/mm}^2$$

But $f_{sc} \not> 0.87 f_y = 0.87(415) = 361.05 \text{ N/mm}^2$

$$\therefore \quad A_{sc} = \frac{M_u - M_{u\lim}}{f_{sc}\left(d - d'\right)} = \frac{120.42 \times 10^6}{361.05\left(600 - 50\right)} = 606.413 \text{ mm}^2$$

Provide 2-20ϕ bars on compression side so that

$$A_{sc\ provided} = 628.3 \text{ mm}^2 > 606.413 \text{ mm}^2 \qquad \text{(OK)}$$

Step-5: Calculate amount of tension reinforcement required

$$A_{st2} = \text{Area of tension steel to balance } A_{sc}$$

$$= \frac{f_{sc}A_{sc}}{0.87 f_y} = 606.413 \text{ mm}^2$$

$$A_{st1} = A_{st\,lim} = \frac{0.362 f_{ck} b x_{ulim}}{0.87 f_y}$$

$$= \frac{0.362(20)(300)(0.48\times600)}{0.87(415)} = 1732.5 \text{ mm}^2$$

Alternatively

$$p_{t\,lim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\frac{x_{u\,lim}}{d} = 41.61\left(\frac{20}{415}\right)(0.48) = 0.9625\%$$

$\therefore$ $$A_{st\,lim} = \frac{0.9625}{100}\times300\times600 = 1732.5 \text{ mm}^2 \quad \text{(same as above)}$$

$\therefore$ $$A_{st} = A_{st1} + A_{st2}$$

$$= 1732.5 + 606.413 \text{ mm}^2$$

$$= 2338.913 \text{ mm}^2 \approx 2339 \text{ mm}^2$$

Provide 5-25ϕ bars so that $A_{\text{st provided}} = 5\times\frac{\pi}{4}\times25^2 = 2454.4 \text{ mm}^2 > 2339 \text{ mm}^2$

$p_{\text{t provided}} = 1.3636\%$ (OK)

Step-6: Design of shear reinforcement

Nominal shear stress $$\tau_v = \frac{V_u}{bd} = \frac{209.25\times1000}{300\times600} = 1.1625 \text{ N/mm}^2$$

For M 20 concrete and 1.3636% p_t, design shear strength of concrete as per table 19 of IS: 456-2000

$(\tau_c) = 0.7 \text{ N/mm}^2 < \tau_v \ (= 1.1625 \text{ N/mm}^2)$

$\therefore$ Shear reinforcement is required.

$$V_{us} = (\tau_v - \tau_c)\, bd$$

$$= (1.1625 - 0.7)\ 300 \times 600 \text{ N} = 83.25 \text{ kN}$$

Using 2-legged 8 mm dia. stirrups,

$$A_{sv} = 2\times\frac{\pi}{4}\times8^2 = 100.53 \text{ mm}^2$$

$\therefore$ $$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87(415)100.53(600)}{83.25\times1000} = 261.6 \text{ mm c/c}$$

Maximum spacing of stirrups $$\not> \begin{cases} 0.75d = 0.75(600)\text{mm} = 450 \text{ mm c/c} \\ 300 \text{ mm} \end{cases}$$

(whichever is less)

∴ Provided 2-legged 8 mm diameter stirrups @ 250 c/c near the supports and spacing can be increased gradually towards the mid span of beam.

Step-7: Deflection control

$$\left(\frac{l}{d}\right)_{actual} = \frac{8000}{600} = 13.33$$

$$\left(\frac{l}{d}\right)_{maximum} = \left(\frac{l}{d}\right)_{basic} k_t k_c$$

$$A_{sc} = 628.3 \text{ mm}^2$$

$$\therefore \quad p_c = \frac{628.3}{300 \times 600} \times 100 = 0.35\%$$

$$\therefore \quad k_t = 0.9 \quad \text{(fig. 4 of IS : 456-2000)}$$

$$k_c = 1.1 \quad \text{(fig. 5 of IS : 456-2000)}$$

$$\therefore \quad \left(\frac{l}{d}\right)_{maximum} = 20 \times 0.9 \times 1.1$$

$$= 19.8 > 13.33 \quad \text{(OK)}$$

Section - C

Q.6 (a) Solution:

1. **Standard penetration test:** The standard penetration test is specially designed for granular soil which cannot be easily sampled. This test is extremely usefull for determining following:
   - (i) Angle of shearing resistance
   - (ii) Relative density
   - (iii) Allowable bearing capacity on the basic of settlement criteria
   - (iv) Point resistance of pile
   - (v) Unconfined compressive strength of cohesive soil.

The standard penetration test is conducted in a bore hole using a standard split-spoon sampler. The sampler is driven into the soil by a drop hammer of 65 kg mass falling through the height of 75 cm at the rate of 30 blows per minute. The number of hammer blows required to drive 150 mm of the sample is counted. The number of blows recorded for the first 150 mm is disregarded which is called seating drive. The number of blows required for 300 mm penetration beyond seating drive is called SPT number.

The standard penetration number is corrected for dilatancy correction and over burden correction.

(1) Over burden correction:

$$N_1 = N_0 \times \frac{350}{\bar{\sigma} + 70}$$

Where N_0 is observed SPT No. and $\bar{\sigma}$ is effective over burden pressure at the test level.

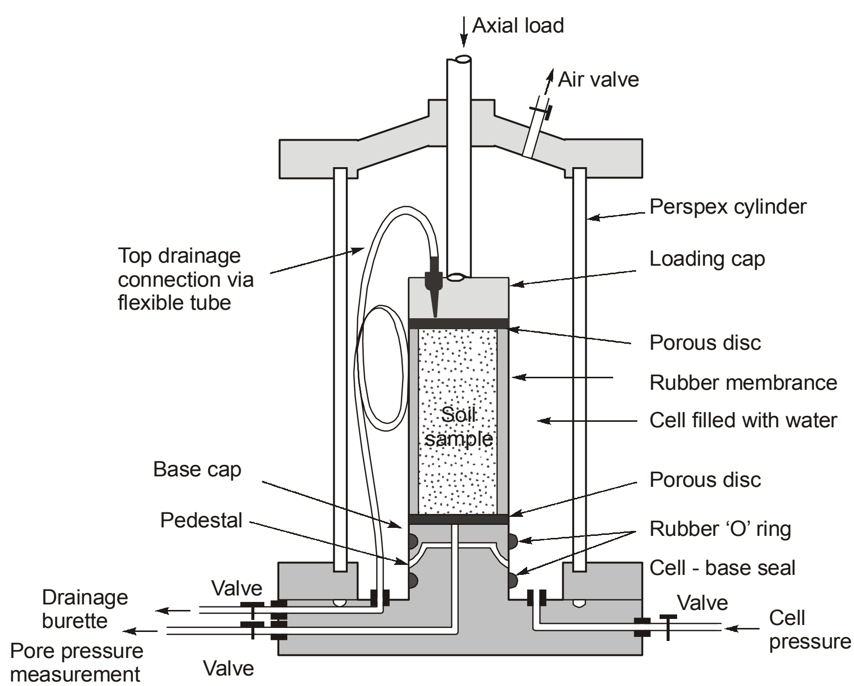
(2) Dilatancy correction:

$$N_2 = 15 + \frac{1}{2}(N_1 - 15)$$

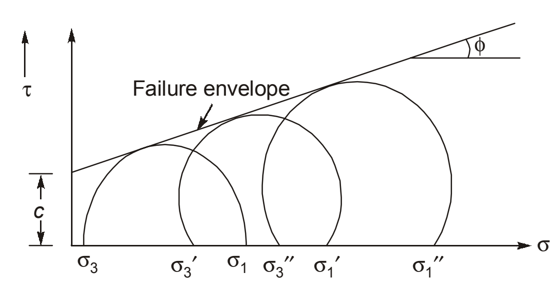
Where, N_1 is SPT number after over burden correction.

2. **Slope protection of embankment:** The upstream slope of embankment should be protected from destructive wave action. The different type of surface protection of upstream slope included stone, stone rip-rap (either dry dumped or hand placed), concrete pavement and steel facing. Some times sacked concrete or willow mattresses are used for relatively small and unimportant embankments. The rock for rip-rap should consist of hard, dense, durable boulders or rock fragment from the quarries.
For downward slope, the simplest and most cost effective means of stabilizing bare soil surface is through the use of vegetation or mulches. This can also be achieved by providing a heavy layer of coarse gravelly material. If sufficient rock or cobble is available, it is preferable to provide a downstream rock or cobble fill. This, addition to its primary function of providing a stabilizing weight, also furnished a protective covering for the underlying earth slope. On high embankments, the effects of surface runoff may be minimized by the use of berms or shoulder at a intervals on the slope to collect and dispose off the runoff water.

3. **Triaxial Test:** The triaxial test is the most versatile among all the shear testing methods. It is suitable for all types of soils and test conditions. In this test, there is a complete control over drainage conditions. In the triaxial test, failure occurs at weakest plane which is not predetermined. The general arrangement is shown in figure.



Triaxial Cell

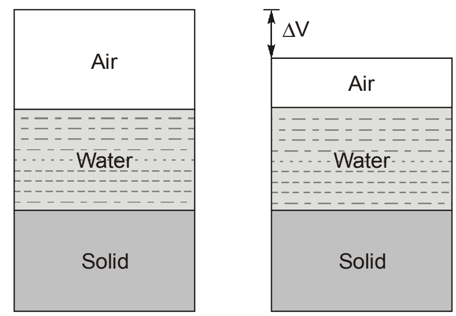


The soil specimen is covered by a thin membranes which extends over a top cap and a bottom pedestal. The appex cell is filled with water, and the required cell or confining pressure

(σ_3) is applied. This confining pressure acts horizontally on the cylindrical surface of the sample and vertically on the top of the specimen. An additional vertical stress called deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) is then applied and steadily increased until failure of specimen and steadily increased until failure of specimen occurs. A mohr circle is drawn at failure condition and test is repeated with changing value of confining pressure σ_3 and the corresponding deviator stress, which gives σ_1, at failure for each case Mohr' circle at failure is drawn and drawn a common tangent to all failure circle called Mohr's circle envelope. Using Mohr's circle envelope c and ϕ can be obtained.

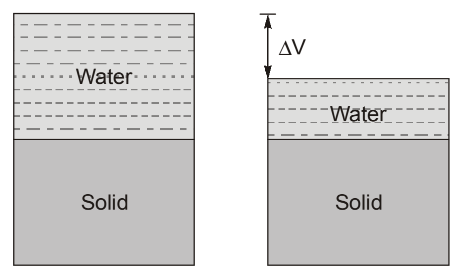
4. **Soil compaction:** Soil compaction is the process by which soil particles are forced to pack more closely together by reducing air voids. This densification is attained by applying some mechanical force on the soil.

   The state of compaction of a soil is measured by the dry density and associated water content.



(i) Before compaction (ii) After compaction

   Consolidation of soil: When soil is fully saturated, then on change in effective stress, soil undergoes change in volume due to expulsion of pore water and molecular rearrangement. This compressibility in volume of soil is known as consolidation.



(i) Before consolidation (ii) After consolidation

Q.6 (b) Solution:

Given Data:

Column load, Q = 150 kN

Depth of footing, $D_f = 1.5$ m

Submerged unit weight, $\gamma' = 11$ kN/m^3

Angle of shearing resistance $\phi = 30°$

Factor of safety, FOS = 3

$$N_q = 10;\ N_\gamma = 6.0$$

We know that ultimate bearing capacity for square footing is given by

$$q_u = 1.3\,cN_c + \gamma' D_f N_q + 0.4\gamma' B N_\gamma$$

Since the value of c is not given, hence assuming the soil to be sandy.

Thus $\quad c = 0$

$\therefore \quad q_u = \gamma' D_f N_q + 0.4\gamma' B N_\gamma$

$\Rightarrow \quad q_u = 11 \times 1.5 \times 10 + 0.4 \times 11 \times B \times 6$

$\Rightarrow \quad q_u = 165 + 26.4B$

$\therefore \quad q_{nu} = q_u - \gamma' D_f = 165 + 26.4B - 11 \times 1.5$

$$\Rightarrow \quad q_{nu} = 148.5 + 26.4B$$

and $$q_{ns} = \frac{q_{nu}}{FOS} = \frac{148.5 + 26.4B}{3}$$

Thus $$q_s = q_{ns} + \gamma D_f$$

$$q_s = \frac{148.5 + 26.4B}{3} + 11 \times 1.5$$

$$q_s = 66 + 8.8\,B$$

$$\because \quad q_s \geq \frac{Q}{B^2}$$

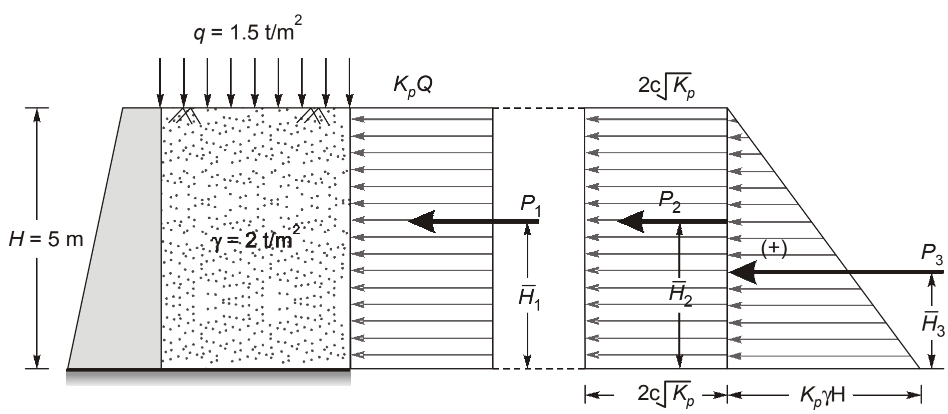
$$\therefore \quad 66 + 8.8B \geq \frac{150}{B^2}$$

$$\Rightarrow 8.8B^3 + 66B^2 - 150 \geq 0 \quad \Rightarrow B \simeq 1.3828125 \text{ m}$$

$$\Rightarrow \quad B = 1.38 \text{ m}$$

Here $D_f > B$ which is against the assumption of Terzaghi's theory. In strict sense, Terzaghi's theory is not applicable here.

Q.7 (a) Solution:



Given, $q = 1.5 \text{ t/m}^2$ $\quad c = 2 \text{ t/m}^2$

$\gamma = 2 \text{ t/m}^3$ $\quad \phi = 30°$

$H = 5 \text{ m}$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1 + \sin 30°}{1 - \sin 30°} = 3$$

$$P_1 = K_p \times q \times H \times 1 = 3 \times 1.5 \times 5 \times 1 = 22.5 \text{ t/m}^2$$

$$\bar{H}_1 = \frac{H}{2} = \frac{5.0}{2} = 2.5 \text{ m from the base}$$

$$P_2 = 2c\sqrt{K_P} \times H \times 1 = 2 \times 2 \times \sqrt{3} \times 5 \times 1 = 34.64 \text{ t/m}$$

$$\overline{H}_2 = \frac{H}{2} = 2.5 \text{ m from base}$$

$$P_3 = \frac{1}{2}\times K_p \times \gamma \times H^2 = \frac{1}{2}\times 3\times 2\times 5^2 = 75 \text{ t/m}$$

$$\overline{H}_3 = \frac{H}{3} = \frac{5}{3} \text{ m from the base}$$

Total passive thrust, $P_p = P_1 + P_2 + P_3 = 22.5 + 34.64 + 75 = 132.14$ t/m

Point of application of total thrust,

$$\overline{H} = \frac{P_1\overline{H}_1 + P_2\overline{H}_2 + P_3\overline{H}_3}{P_1 + P_2 + P_3}$$

$$= \frac{22.5\times 2.5 + 34.64\times 2.5 + 75\times\left(\frac{5}{3}\right)}{132.14} = 2.027 \text{ m from base}$$

Q.7 (b) Solution:

(i) Plasticity Index: It is the difference between liquid limit (w_L) and plastic limit (w_P) i.e

$$I_P = w_L - w_P$$

It is denoted by I_P. It defines the range of consistency in which soil behave in plastic stage. The greater the plasticity index of the soil, the higher will be the attraction between the particles of the soil and the greater the plasticity of soil. On the basis of plasticity index, the soils are classified by Atterberg as follows:

Plasticity index (%)	Plasticity
0	Non-Plastic
< 7	Low Plastic
7 - 17	Medium Plastic
> 17	High Plastic

(ii) Liquidity Index: The liquidity Index (I_L) is the ratio expressed as a percentage of the difference between natural water conent and plastic limit to its plasticity index i.e.,

$$I_L = \frac{w - w_P}{I_P}$$

or $$I_L = 1 - I_C$$

When, $I_L < 0$; Soil is in semi solid state (brittle)

$I_L = 0$; Soil is in stiff state

$0 < I_L < 1$; Soil is in plastic state

$I_L > 1$; Soil is in Liquid state

(iii) Consistency Index: The consistency index (I_C) of a soil is the ratio of the difference between liquid limit to natrual water conetnt to its plasticity index i.e.,

$$I_C = \frac{w_L - w}{I_P}$$

When $I_C < 0$; Soil is in Liquid state

$0 < I_C < 1$; Soil is in semisolid state

$I_C > 1$; Soil is in solid state

(iv) Uniformity Coefficient: Coefficient of uniformity is defined as-

$$C_u = \frac{D_{60}}{D_{10}}$$

Where, D_{60} mean that size below which 60% particles are below this size by weight. D_{10} is that size below which 10% particles are finer than this size by weight. It is also called effective size.

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