DETAILS EXPLANATIONS

CE : Paper-1 (Paper-6) [Full Syllabus]

[PART : A]

1. The dynamic modulus of concrete is determined in laboratory by determining the lowest natural frequency of longitudinal vibration of prismatical specimens.

2. C_3S : Contributes more to the early strengths, whereas.
   C_2S : Give later and final strength to cement.

3. Maximum permissible specing
   = 5d or 450 mm (leaser)
   ⇒ Maximum spacing
   = 5 × 120 = 600 mm or 450 mm
   ⇒ So, maximum spacing = 450 mm

4. Slenderness ratio

$$\lambda = \frac{l}{\sqrt{(I/A)}} = \frac{l}{\sqrt{\left(\frac{\pi}{64}D\right)/\left(\frac{\pi}{4}D^2\right)}} = \frac{4l}{D}$$

$$\therefore \frac{(l/D)}{\lambda} = \frac{1}{4}$$

5. $$\frac{A_{sv}}{B.d} = \frac{40S_v}{f_y.d}\%$$

6. $\because$ Bulk modulus (k) $= \frac{\Delta p}{(\Delta v / V)}$

$$\therefore \quad \Delta V = \frac{1330 \times 8000}{1.33 \times 10^6} = 8cc$$

7. Radius of mohr's circle :

$$r = \sqrt{\left(\frac{\sigma_1 - \sigma_2}{2}\right)^2 + q^2}$$

8. Factor of safety of finite slope :

$$F = \frac{Cr^2\theta}{we}$$

where, r = Radius of rupture
C = Unit cohesion.

9. Maximum bending moment in purlin :

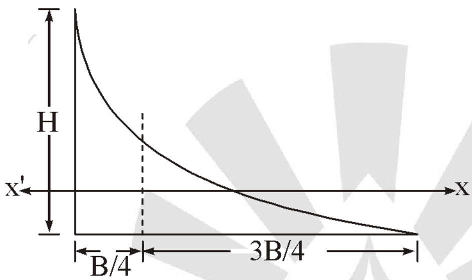
$$M_{max} = \frac{wl^2}{10} = \frac{20 \times 5^2}{10} = 50 \text{ kN-m}$$

10. Beaded pointing is the special type of pointing formed by a steel or ironed with a concave edge. It gives good appearance, but tiable to damage easily.

11. Porosity (n) = $\frac{V_D}{V_n} = \frac{1}{2} = 0.5$

$\therefore$ Void Ratio $\Rightarrow e = \frac{n}{1-n} = \frac{0.5}{1-0.5} = 1$

12.



13. Plastic section modulus for rectangular section :

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{B.d^2}{4}$$

$$Z_p = \frac{25 \times 40^2}{4} = 10000 \text{ cm}^3$$

14. We know, $k \propto D_{10}^2$

So, with half the effective size, permeability will be one forth of initial value

$$k_1 = 0.4 \text{ cm/hr}$$

$$\therefore \quad k_2 = \frac{0.4}{4} = 0.1 \text{ cm/hr}$$

15. Compactive energy ratio :

$$= \frac{(MgH)_h}{(MgH)_l} = \frac{4.9 \times 9.81 \times 450 \times 5 \times 25}{2.6 \times 9.81 \times 310 \times 3 \times 25} = 4.56$$

16.
- Dead loads
- Imposed loads
- Temperature effects
- Hydrostatic and soil pressure
- Errection loads
- Accidental loads
- Wind and Earthquake force etc.

17.
- Material of the column
- Cross sectional configuration
- Length of the column
- Support conditions at the end
- Residual stresses

18. $$\frac{\text{Volume of helical reinforcement}}{\text{Volume of the core}} \nless 0.36\left(\frac{A_g}{A_c}-1\right)\frac{f_{ck}}{f_y}$$

Where, A_g = Gross sectional Area

A_c = Core area

19. For mild steel :

$E = 2 \times 10^5 \text{ N/mm}^2$

$\mu = 0.28$

$\therefore \quad E = 2G(1 + \mu)$

$$G = \frac{2\times10^5}{2(1+0.28)} = 78125 \text{ N/mm}^2$$

20.
- Plain section remains plain before and after bending.
- Hook's law is valid.

[PART : B]

21. The center of mohr's circle

$$C = \frac{\sigma_1+\sigma_2}{2} = \frac{14+2}{2} = 8 \text{ N/mm}^2$$

Radius of mohr's circle :

$$= \sqrt{\left(\frac{\sigma_1-\sigma_2}{2}\right)^2+q^2} = \sqrt{\left(\frac{14-2}{2}\right)^2+8^2}$$

$= 10 \text{ N/mm}^2$

$\therefore$ Maximum shear stress = Radius = 10 N/mm^2

22. $\therefore$ Hoop stress $(\sigma_\theta) = \frac{pr}{t}$

Longitudinal stress $\Rightarrow \sigma_z = \frac{pr}{2t}$

Strain in circumpherential direction :

$$\in_q = \frac{1}{E}(\sigma_\theta-\mu\sigma_z) = \frac{pr}{2t}(2-\mu)$$

Change in radius :

$$\Delta r = \frac{pr^2(2-\mu)}{2Et}$$

23. The muller Breslau principal may be stated as follows :
"If an internal stress component, or a reaction component is considered to act through some small distance and thereby to deflect or displace a structure, the curve of the deflected or displaced structure will be, to some scale, the influence line for the stress or reaction component."

24. Assume neutral Axis in flange :

$$x_{u_{lim}} = 0.53d = 0.53 \times 400 = 212 \text{ mm}$$

for x_u; C = T

$0.36f_{ck}b_fx_u = 0.87f_yA_{st}$

$$\Rightarrow 0.36 \times 20 \times 740x_u = 0.87 \times 250 \times 4 \times \frac{\pi}{4} \times 20^2$$

$\Rightarrow x_4 = 51.29 \text{ mm} < D_F$

Assumption correct, N.A. is in flange.

$$x_u < x_{u_{lim}}$$

$$m_u = 0.87f_y A_{st}(d - 0.42x_u)$$

$$= 0.87\times250 \times 4 \times\frac{\pi}{4}\times 20^2(400 - 0.42 \times 51.29)\times10^{-6}$$

$$m_u = 103 \text{ kN-m}$$

25. For prestressing effect,
Prestressing force = p.A

$$P = 1200\times4\times\frac{\pi}{4}\times5^2 = 94200 \text{ N}$$

$$\sigma_b = \frac{P}{A}+\frac{Pe}{z} = \frac{94.2\times10^3}{150\times300}+\frac{94.2\times10^3\times50}{(150\times300^2/6)}$$

$$\sigma_b = 4.2 \text{ MPa}$$

26. IS code concept :
The serviceability requirement for the deflection should be such that neither the efficiency nor appearance of a structure should be affected by the deflection which will occur during it's life.
As per IS 456 : 2000,

- The final deflection of horizontal members below the level of casting should not exceed $\frac{\text{Span}}{250}$.
- The deflection taking place after the construction of partitions or application of finishes shuld not exceed $\frac{\text{Span}}{350}$ or 20 mm which ever is less.

27. Assumptions :

- Soil mass is homogenous, dry, isotropic, cohensionless, elastic and semi-infinite.
- Base of wall is smooth and vertical.
- Ground surface is plain.
- Failure should be in the form of element.
- For any two elements for which stress relation developes, rupture plain should also be same.

28. For quick sand condition :

$$i = i_c = \text{Critical hydraulic gradient}$$

$$i_c = \frac{G-1}{1+e}$$

$$e = \text{Void ratio} = \frac{n}{1-n}$$

$$= \frac{0.6}{1-0.6} = \frac{0.6}{0.4} = 1.5$$

$$i_c = \frac{H_L}{L} = 0.668$$

Head loss $= H_L = 0.668 \times 4$

$\Rightarrow \quad H_L = 2.672$ m

29. ***IS light compaction test and Heavy compaction test in table :***

Particular	IS-Light Compaction Test	Heavy Compaction Test
Volume of mould	1000 cc	1000 cc
Weight of hammer	2.6 kg	4.9 kg
Height of fall	310	450 mm
Number of layer	3	5
Blows on each layers	25	25

30. ***Portal frames with side sway Caures :***

- Eccentric or unsymmetrical loading on the portal frame.
- Unsymmetrical out-line of portal frame.
- Different end conditions of the columns of the portal frame.
- Non-uniform section of the members.
- Horizontal loading.
- Settlement of the supports.

31. ***Constant Head Permeability Test***

In this test, water is passed through soil sample of L-length and quantity in per unit time passed is measured.

The cofficient of permeability

$$k = \frac{Q.L}{t.H_L.A}$$

Where, Q = Volume of water collected in time 't' in m^3.

Constant head permeability test is useful for coarse grain soil and it is a laboratory method.

$$k \propto \frac{1}{\mu}$$

$$\mu \propto \frac{1}{T}$$

So, $k \propto T$

32. ***Bending moment Capacity :***

$$\Rightarrow \quad M = f_y \cdot Z = 250 \times \frac{BD^2}{6}$$

$$= 250 \times \frac{150 \times 200^2}{6} \times 10^{-6}$$

$$M = \frac{25 \times 15 \times 2 \times 2}{6} = 250 \text{ kN-m}$$

$$M = 250 \text{ kN-m}$$

[PART : C]

33. By inspection, elements DE and FB of the wall will have the maximum stress. Let us work out the stresses at plinth level.

Loads : Parapet $= \left(\frac{19+3}{100}\right) \times 0.9 \times 20000 = 3960$ N/m.

Wall $= \frac{19+1.5}{100} \times 3 \times 20000 = 12300$ N/m

Total = 16260 N/m

(It is common practice not to make any deductions for opening since calculations for the design of masonry are not very precise.)

Roof-Load :

RCC slab = 0.1 × 1 × 1 × 25000 = 2500 N/m^2

Lime concrete terrace 10 cm thick

= 0.1 × 1 × 1 = 20000 = 2000 N/m^2

Live load = 1.5 kN/m^2 = 1500 N/m^2

$\therefore$ Total roof load = 2500 + 2000 + 1500 = 6000 N/m^2

Effective span of slab = 3.0 + 0.1 = 3.1 m

$\therefore$ Roof load on wall = $\frac{6000 \times 3.1}{2}$ = 9300 N/m

Protion FB of wall :

Length of wall = 0.4 m + $\frac{0.9}{2}$ = 0.5 m.

Though this comes under the definition of column, we will treat it as wall because of stiffening by cross -wall. Due to this, no area reduction factor is applicable.

The wall will carry additional load due to window opening.

$\therefore$ Total load = (16260 + 9300)[0.5 + 0.9/2] = 24282 N

Because of raked joints, t = 19 – 1 = 18 cm = 180 mm

$\therefore$ Compressive stress = $\frac{24282}{180 \times 500}$ = 0.27 N/mm^2

Slenderness ratio = $\frac{h}{t} = \frac{(1.5 + 3 + 0.05)}{0.18} \times 0.75$ = 19

Effective length $l = 2l = 2 \times 0.5 = 1$ m

$\therefore$ $S_R = \frac{1.0}{0.18} = 5.6 = 6$

Hence governing = S_R = 6

Stress factor (for S_R = 6) is 1.00

Hence basic stress required = $\frac{0.27}{1}$ = 0.27 N/mm^2

Since we find that bricks of 3.5 N/mm^2 could be used. Shape modifiction factor for this strength is 1.2.

$\therefore$ Requisite basic stress of masonary

$$= \frac{0.27}{1.2} = 0.225 \text{ N/mm}^2$$

Protion DE of wall

Length = 0.6 m

This wall carries load of both the openings to its either side.

$\therefore$ Load on wall = (16260 + 9300)$\left[\frac{0.9}{2} + 0.6 + \frac{0.9}{2}\right]$ = 38340 N

∴ Compressive stress(f) at plinth level

$$= \frac{38340}{180+600} = 0.355 \text{ N/mm}^2$$

This portion of wall comes under the definition of a column.

∴ Effective height = 1.5 × height of taller opening.

$$= 1.5 \times 2 = 3 \text{ m}$$

Effective height of wall

$$= 0.75 \text{ H} = 0.75(1.5 + 3 + 0.05)$$

$$= 3.41 \text{ m}$$

Effective height of wall taken as column

$$\text{H} = 1.5 + 3 + 0.05 = 4.55 \text{ m}$$

Effective height = 3 m

$$S_R = \frac{3}{0.18} = 16.7$$

Stress factor k_s for S_R = 16.7 is

$$k_s = 0.58 - \frac{0.58-0.5}{2} \times 0.7 = 0.55$$

Area of wall in plan = 18 × 60 = 1080 cm²

Area reduction factor $k_a = 0.75 + \frac{1080}{1200} = 0.84$

Hence basic stress of requisite masonry with unity shape modification factor

$$= \frac{f}{k_s \times k_a} = \frac{0.355}{0.55 \times 0.84}$$

$$= 0.768 \text{ N/mm}^2$$

Thus this portion of wall carries maximum stress and will govern the design. We find that bricks of 10.5 N/mm² strength will be required.

The shape reduction factor of modular bricks of this stress is 1.1 from,

∴ f_b required $= \frac{0.768}{1.1} = 0.7 \text{ N/mm}^2$

Hence, we find that masonry should have bricks of 10.5 N/mm² strength, with M2 mortar (1 : 2 : 9 or 1 : 6), giving a basic strength of 0.85 N/mm².

Hence required masonry is 105 – M2.

34. Since no indication is given in the code, the effective length is taken as the actual length.

(i) Connected by two bolts at the ends.

(a) For two bolts at each end, for fixed condition,

$$k_1 = 0.20;\ k_2 = 0.35$$

and $$k_3 = 20\ ;\ \in = \left(\frac{250}{f_y}\right)^{0.5} = 1.0$$

$$\lambda_{vv} = \frac{\left(\frac{3000}{39.3}\right)}{\sqrt{\left(\frac{\pi^2 E}{250}\right)}}$$

$$\lambda_\phi = \frac{(200+200)}{\left[(2\times20)\sqrt{(\pi^2\times2\times10^5/250)}\right]}$$

$$= 0.1125$$

$$\lambda_{et} = \sqrt{[k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2]}$$

$$= \sqrt{[0.20 + 0.35\times0.859^2 + 20\times0.1125^2]}$$

$$= 0.843$$

$$f_{cd} = \frac{(f_y/\gamma_{mo})}{[\phi + (\phi^2 - \lambda_e^2)^{0.5}]}$$

$$\phi = 0.5[1+0.49(0.843 - 0.2) + 0.843^2]$$

$$= 1.013$$

$$f_{cd} = \frac{(240/1.1)}{\left[1.013 + 10.13^2 - 0.843^2\right]^{0.5}}$$

$$= 138.56\ N/mm^2$$

$$p_d = 138.56 \times \frac{7640}{1000} = 1058.6\ kN$$

(b) If we consider the fixity condition as hinged :

$$k_1 = 0.70\ ;\ k_2 = 0.60$$

and $$k_3 = 5$$

Hence $$\lambda_e = \sqrt{(0.70 + 0.60\times0.85g^2 + 5\times0.1125^2)}$$

$$\lambda_e = 1.098$$

$$\phi = 0.5[1 + 0.49(1.098 - 0.2) + 1.098^2]$$

$$\phi = 1.322$$

$$f_{cd} = \frac{(240/1.1)}{[1.322 + (1.322^2 - 1.098^2)^{0.5}]}$$

$$f_{cd} = \frac{218.18}{2.058} = 106 \text{ N/mm}^2$$

$$p_d = \frac{106 \times 7640}{1000} = 809.8 \text{ kN}$$

(ii) Connected by only one bolt at the ends and the fixity condition taken as hinged,

$$k_1 = 1.25;\ k_2 = 0.50 \text{ and } k_3 = 60$$

Hence $\lambda_e = \sqrt{(1.25 + 0.50 \times 0.859^2 + 60 \times 0.1125^2)}$

$$\lambda_e = 1.542$$

$$\phi = 0.5[1 + 0.49(1.542 - 0.2) + 1.542^2]$$

$$\phi = 2.018$$

$$f_{cd} = \frac{(240/1.1)}{[2.018 + (2.018^2 - 1.542^2)^{0.5}]}$$

$$f_{cd} = \frac{218.18}{3.32} = 65.72 \text{ N/mm}^2$$

$$p_d = \frac{65.72 \times 7640}{1000} = 502 \text{ kN}$$

(iii) When the strut is welded at each end, it is similar to case (a) and the strength will be equal to 1058.6 kN.

Note that when two bolts are provided at the ends, depending on the assumed fixity condition we get the capacity as 1058.6 kN or 809.8 kN. The value of 1058.6 kN is very close to the concentric loading case 1058.

Let us cross-check this value with the equation

$$\lambda = \left(\frac{kL}{r\pi}\right)\sqrt{\left(\frac{f_y}{E}\right)}$$

$$\lambda = \frac{3000}{(33.3 \times \pi)}\sqrt{\left(\frac{250}{2 \times 10^5}\right)} = 0.859$$

$$p_d = \frac{(0.990 + 0.150\lambda - 0.360\lambda^2 - 0.020\lambda^3)}{\lambda ml}$$

$$p_d = \frac{250 \times 7640(0.990 + 0.15 \times 0.859 - 0.360 \times 0.859^2 - 0.02 \times 0.859^3)}{1.1 \times 1000}$$

$$p_d = \frac{191.03 \times 7640}{1000} = 1459.5 \text{ kN}$$

This shows that the value adopted by the indian code is very conservative.

35. (i) For f_y = 250 MPa, from table, maximum outstand between for the flange to be compact = 8.4.

Actual between, using figure.

$$8.25 = \frac{(325 - 15/2)}{25} = 12.7 > 8.4$$

Maximum between for the flange to be semi-compact = 13.6

Hence the section is semi-compact and $m_d = z_e f_y$

$$I_z = \frac{BD^3}{12} - \frac{(B - t_w)d^3}{12}$$

$$I_z = \frac{650 \times 1550^3}{12} - \frac{(650 - 15)1500^3}{12}$$

$$I_z = 23116.15 \times 10^6 \text{ mm}^4$$

$$z_{ez} = 23116.15 \times 10^6 \left(\frac{1500}{2 + 25}\right)$$

$$z_{ez} = 29827.3 \times 10^3 \text{ mm}^3$$

$$m_d = \frac{250 \times 29827.3 \times 10^3}{10^6}$$

$$m_d = 7456.8 \text{ kN-m}$$

$$z_p = \frac{2Bt_f(D - t_f)}{2} + \frac{t_w d^2}{4}$$

$$z_p = \frac{2 \times 650 \times 25(1500 - 25)}{2} + \frac{15 \times 1500^2}{4}$$

$$z_p = 32406.25 \times 10^3 \text{ mm}^3$$

$$m_p = \frac{32406.25 \times 10^3 \times 250}{10^6}$$

$$m_p = 8101.5 \text{ kN/m}$$

Hence reduction in capacity from that corresponding to compact behaviour.

$$= \frac{(8101.5 - 7456.8)}{8101.5 \times 100} = 7.96\%$$

(ii) For f_y = 410 MPa, maximum b/t for the flange to be semi-compact

$$= 13.6 \times \left(\frac{250}{410}\right)^{0.5} = 10.6.$$

Therefore the section is slender limit for the effective flange width for semi compact behaviour

$$= 10.6 \times 25 = 265 \text{ mm}$$

Hence effective tap width of the flange

$$= 265 \times 2 + 15 = 454 \text{ mm}$$

Location of neutral axis

Taking moments about the top edge, the distance of the neutral axis from the tap edge,

$$\bar{y} = \frac{\left[\frac{545\times25\times25}{2} + 1500\times15\times\left(\frac{1500}{2+25}\right) + 650\times25\times\left(\frac{1525+25}{2}\right)\right]}{545\times25+1500\times15+650\times25}$$

$$\bar{y} = 813.2 \text{ mm}$$

$$I_z = (545 \times 25)(813.2 - 12.5)^2 + \left(\frac{15\times1500^3}{12}\right) + (15 \times 1500)$$

$$\times (813.2 - 775)^2 + 650 \times 25(736.8 - 12.5)^2$$

$$I_z = 2.1511 \times 10^{10} \text{ mm}^4$$

$$z_z(\text{top flange}) = \frac{2.1511\times10^{10}}{813.2} = 26453 \times 10^3 \text{ mm}^3$$

$$m_d = \frac{410\times26453\times10^3}{10^6} = 10845.8 \text{ kN-m}$$

Capacity, if the whole section is effective as the semi-compact section behaviour.

$$= \frac{12229.2-10845.8}{12229.2}\times100 = 11.3\%$$

Capacity, if the whole section is as effective as the plastic section

$$m_p = \frac{32406.25\times10^3\times410}{10^6} = 13286.5 \text{ kN-m}$$

Hence reduction in capacity due to slendreness.

$$= \frac{13286.5-10845.8}{13286.5}\times100 = 18.4\%$$

Thus 18.4% of the capacity of the cross section could not be utilized, due to the slenderness of the cross section.

36. As the load is acting on the joint, there will be no fixed end moments. However, due to side away, moments will be induced at joint A, B and C.

Distribution Factors (Table) :

Joint	Member	Relative Stiffness	Sum	D.F.
B	BA	$\frac{I}{4}$	$\frac{2I}{4}$	0.5
	BC	$\frac{I}{4}$		0.5
C	CB	$\frac{I}{4} = \frac{4I}{16}$	$\frac{7I}{16}$	0.57
	CD	$\frac{3}{4} \times \frac{I}{4} = \frac{3I}{16}$		0.43

Side sway under the action of the 10 kN load, there will be side sway to the right and the columns AB and CD will rotate in a clockwise direction. Thus negative moments will be induced at A, B and C in these columns. As the end A is fixed and D is hinged, the ratio of moments will be :

$$\frac{M_{BA}}{M_{CD}} = \frac{2I_1 / L_1^2}{I_2 / L_2^2} = \frac{2I / 4^2}{I / 4^2} = \frac{2}{1}$$

Also, $M_{BA} = M_{AB}$

Let us, first of all, assume arbitrary values of these moments and find out the corresponding sway force.

Let $M_{CD} = -5$ kN-m

$M_{BA} = M_{AB} = -10$ kN-m

A Moment Distribtion (Table) :

A	B		C		D	
	0.5	0.5	0.57	0.43		F.E.M.
–10.0	–10.0	–	–	–5.0	0	Balance
–	+ 5.0	+5.0	+2.86	+2.14	–	
+2.50	–	–1.43	+2.50	–	–	Carry over
–	–0.72	–0.71	–1.43	–1.07	–	Balance
–0.36	–	–0.72	–0.36	–	–	Carry over
–	+0.36	+0.36	+0.21	+0.15	–	Balance
+0.18	–	+0.10	+0.18	–0.08	–	Carry over
–	–0.05	–0.05	–0.10	–	–	Balance
–0.03	–	–0.05	–0.03	–	–	Carry over
+0.01	+0.03	+0.02	+0.02	+0.01	–	Balance and Carry over
–7.70	–5.38	+5.38	+3.85	–3.85	0	Final moment

Horizontal reaction at

$$A = -\frac{7.70 - 5.38}{4} = \frac{13.08}{4} = 3.27 \text{ kN}(-)$$

Horizontal reaction at $D = -\frac{3.84}{4} = 0.963 \text{ kN}(+)$

The sway force causing the assumed moments

$$= 3.27 + 0.963 = 4.233 \text{ kN}(\rightarrow)$$

But actual sway force is 10 kN;

Hence the moments will be increased praportionately in the ratio of $\frac{10}{4.233}$, as shown in table.

	A	B	B	C	C	D
Sway = 4.233 kN	–7.70	–5.48	+5.38	+3.85	–3.85	0
Sway = 10 kN	–18.18	–12.73	+12.73	+9.09	–0.09	0

The horizontal reaction at

$$A = \frac{3.27}{4.233} \times 10 = 7.72 \text{ kN}(\leftarrow)$$

The horizontal reaction at

$$D = \frac{0.963}{4.233} \times 10 = 2.28 \text{ kN}(\leftarrow)$$

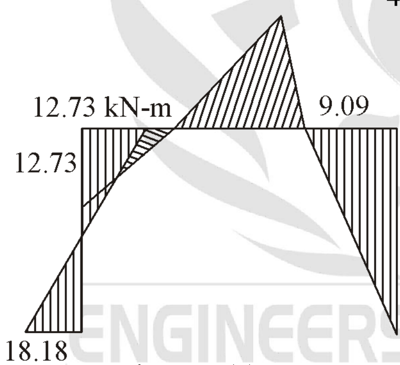


Figure : (a) B.M.D.

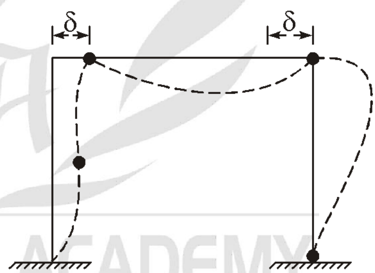


Figure : (b) Deflected Shape

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

37. The column may be taken to be restrained against rotation and translation both at top and bottom. It conforms to the case (a) in table.

Hence, effective length

$$I_e = 0.651 = 0.65 \times 5 = 3.25 \text{ m}$$

Permissible stresses are

$$\sigma_{cc} = 5 \text{ N/mm}^2$$

$$\sigma_{sc} = 190 \text{ N/mm}^2$$

Intially, the column may be assumed to be short and the section dimensions worked out. Slenderness ratio may then be calculated to see weather the column is short. Otherwise, the design may be revised as for a long column.

(i) Square section with lateral ties.

Axial load bearing capacity of a short column is

$$p = \sigma_{cc}A_c + \sigma_{sc}A_{sc}$$
$$= \sigma_{cc}(A_g - A_{sc}) + \sigma_{sc}A_{sc}$$
$$= \sigma_{cc}A_g + (\sigma_{sc} - \sigma_{cc})A_{sc}$$
$$= \{\sigma_{cc} + (\sigma_{sc} + \sigma_{cc})p\}A_g$$

Where, $p = \frac{A_{sc}}{A_g}$ is the proportion of longitudinal rein-forcement.

Let the self weight of the column be 12 kN. Then, total design axial load P = 630 + 12 = 642 kN.

Keeping minimum reinforcement (0.8%) in the column,

$$p = 0.008$$

Sub-stituting the values in equation

$$642 \times 10^3 = \{5 + (190 - 5) \times 0.008\}A_g$$
$$A_g = 99074 \text{ mm}^2$$
$$\text{Length of side} = \sqrt{99074} = 314.76 \text{ mm}$$

Adopt a (315 × 315 mm) cross-section.

$$\text{Slenderness ratio} = \frac{3.25 \times 1000}{315} = 10.32 < 12$$

Hence, the assumption that the column is short is justified.

Longitudinal reinforcement

$$A_g = 0.008 \times 315^2 = 793.8 \text{ mm}^2$$

Provide 4 nos, 16 mm dia bars, giving $A_{sc} = 804.3 \text{ mm}^2$

Transverse reinforcement in the form of lateral ties will be provided.

Diameter of tie bar

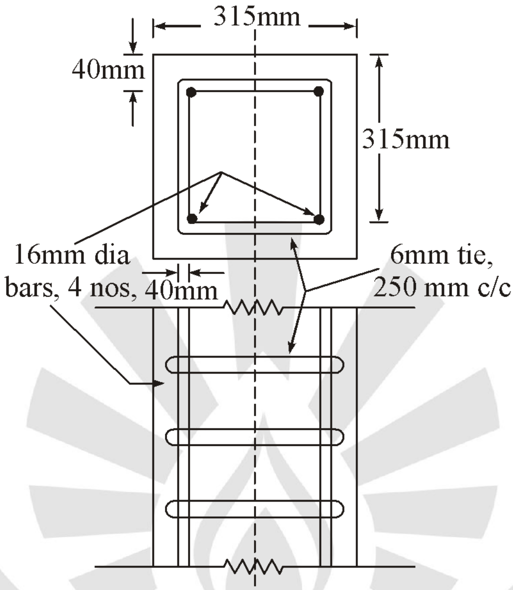
$$\nless \frac{1}{4} \times 16 \text{ i.e., } 4$$
$$\nless 5 \text{ mm}$$

Adopt 6 mm diameter M.S. ties. Spacing of ties

$$\ngtr 315 \text{ mm}$$
$$\ngtr 16 \times 16 \text{ i.e., } 250 \text{ mm}$$
$$\ngtr 48 \times 6 \text{ i.e., } 288 \text{ mm}$$

Adopt a spacing of 250 mm c/c througout.

A cover of 40 mm is kept on the longitudinal bars.
Hence, the cover on lateral ties will be 34 mm, which is O.K.
Figure shows the details of the reinforcement in the column.



38. ***Penetration Tests :***

These tests involves the mesurements of the reistance to penetration of a sampling spoon a cone or other shaped tool under, dynamic or static loadings. The resistance is empirically correlated with some of the engineering properties of soil, such as density index, bearing capacity etc. two commonly used penetration tests are :

(i) Standard penteration test

(ii) Dutch come test

Standard Penetration Test :

The test (IS : 2131 - 1963) is performed in a clean hale, 55 to 150 mm in diameter. A casing or drilling mud may be used to support the side of the hole. A thick wall split tube sampler, 50.8 mm outer diameter and 35 mm internal diameter is driven into the undisturbed soil at the bottom of the hole under the blows of 65 kg drive weight with 75 cm free fall. The minimum open length of the sampler should be 60 cm. The sampler is first driven through 15 cm as a seating drive. It is further driven through 30 cm and the number of blows required for this are counted. This number of blows is termed as penetration resistance N.

In very fine, or silty, saturated sand, an apparent increase in resistance occurs. Terzaghi and Peck have recommended the use an equivalent pentration resistance N_e, in place of the actually observed value of N_1 is greater than 15. N_e is given by the following relation.

$$N_e = 15 + \frac{1}{2}(N-15)$$

Terzaghi and Peck's empirical charts for determining net bearing pressure P for footing on sand depend on B and N value, to limit maximum settlement of individual footing to 2.5 cm and differential sattlement of 2 cm, assuming that a differential sattlement of 2 cm can be tolerated by most of the oridinary structures. The empirical relations are represented by the following equation :

$$q_p = 3.5(N - 3)\left[\frac{B+0.3}{2B}\right]^2 R_{w2} R_d$$

Where q_p = Allowable net increase in soil pressure over exising soil pressure for settlement of 2.5 cm $\left(\text{in } \frac{t}{m^2}\right)$

(where 1 $t/m^2 \approx 10$ kN/m^2)

N = Standard penetration number, with applicable, corelations

B = Width of footing (or least lateral dimension) in meters.

R_{w2} = Water reduction factor = $0.5\left[1+\frac{Z_{w1}}{B} \le 1\right]$

Z_{w1} = Depth of water table below the level of footing.

If the water table is above the base of footing, R_{w1} should be taken as 0.5

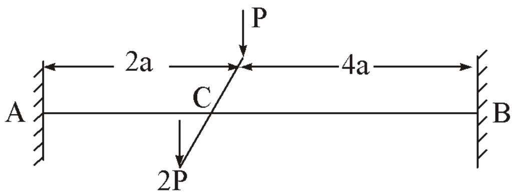
R_d = Depth factor = $\left(1+\frac{0.2D}{B}\right) \le 1.20$

Dutch Cone Test :

This test is used for getting a continuous record of the resistance of soil by pentrating steadily under static pressure a cone with a of 10 cm^2 (3.6 cm in diameter) and an angle of 60° at vertex. The cone is carried at the lower end of a steel driving rod which passes through a steel tube (mantle) with external diameter equal to the base of the cone either the cone or the tube, or both together can be force into the soil by means of jack. To know the cone resistance, the cone along is first force down for a distance of 8 cm and the maximum value of resistance is recorded. The steel tube is then pushed down upto the cone, and both together and fucther penetreted through a depth of 20 cm to give the total of cone resistance and the frictional resistance clong the tube.

The cone test is considered very useful in determining the bearing capcity of pits in coheonless soils, particularly in fine sands of varyigly equal to 5 to 10 times the penetration resistance N.

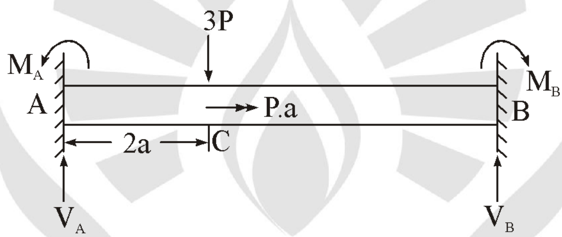
39. Calculation of Moments :



$$M_A = \frac{-3P \times (2a) \times (4a)^2}{(6a)^2}$$

$$M_A = -\frac{8Pa}{3}$$ (–ve) indicates moments acts.

In Anti-clockwise direction



$$M_B = +\frac{3P \times (2a)^2 \times (4a)}{(6a)^2} = +\frac{4Pa}{3}$$

(+ve) indicate moments act in clockwise direction.

Calculation of Reactions :

$$V_A + V_B = 3\ P$$

Taking moment about A, we get

$$\Sigma M_A = 0$$

$$\Rightarrow (V_A \times 6a) + M_A - M_B = 0$$

$$\Rightarrow V_B.6a + \frac{8Pa}{3} - \frac{4Pa}{3} - 3P(2a) = 0$$

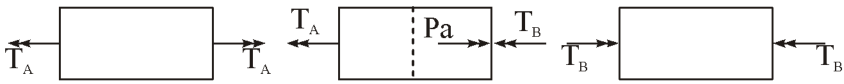
$$\Rightarrow V_B \times 6a + \frac{4Pa}{3} - 6Pa = 0$$

$$V_B = \frac{7P}{9}$$

$$V_A = \frac{20P}{9}$$

Calculation of torsional moments :

Drawing the FBD for torsion



Equlibrium Condition

$$T_A + T_B = Pa \quad \ldots(1)$$

Compatibility condition

$$\theta_{AC} + \theta_{CB} = 0$$

$$-\frac{T_A.2a}{GI_p} + \frac{T_B.4a}{GI_p} = 0$$

$$T_A = 2T_B$$

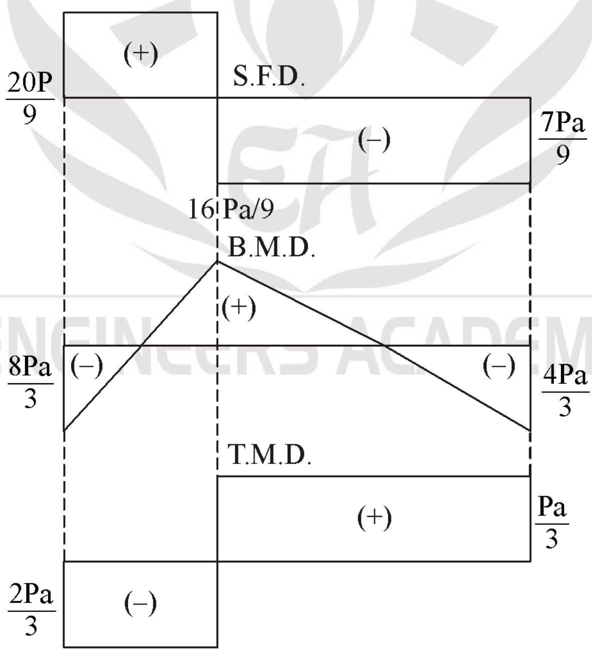
From equation (1), we get

$$T_A + T_B = Pa$$

$$2T_B + T_B = Pa$$

$$T_B = \frac{Pa}{3}$$

$$T_A = 2T_B = \frac{2Pa}{3}$$



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