DETAILS EXPLANATIONS

CE : Paper-1 (Paper-2) [Full Syllabus]

[PART : A]

1. Bulk modulus $K = \dfrac{P}{\left(\dfrac{\Delta V}{V}\right)}$

$$\Delta V = \frac{1280 \times 7800}{1.33 \times 10^6} = 7.50 \text{ cc}$$

2. Modulus of Resilience

$$U = \frac{\sigma^2}{2E}; \; E = \frac{fL}{\Delta l}$$

$$\therefore \quad U = \frac{f\Delta l}{2L} = \frac{200 \times 2.2}{2 \times 1.2 \times 10^3} = 0.18 \text{ units}$$

3. Equivalent bending moment

$$= \frac{M + \sqrt{M^2 + T^2}}{2}$$

Equivalent-Torsion $= \sqrt{M^2 + T^2}$

4. Slenderness Ratio, $\lambda = \dfrac{l}{r}$

where, r = Radius of Gyration $= \sqrt{\dfrac{I}{A}} = \dfrac{d}{4}$

$$\Rightarrow \frac{l}{d} = \frac{150}{4} = 37.5$$

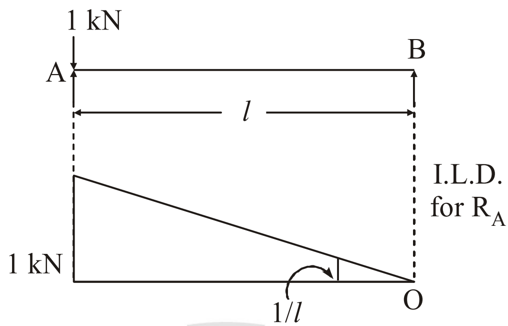
5. For two hinged semicircular arch with load 'W' applied at any section, the radius vector corresponding to which makes an angle θ with horizontal

$$H = \frac{W}{\pi}\sin^2\theta$$

with load at crown $\theta = \dfrac{\pi}{2}$

$$H = \frac{W}{\pi}$$

6.



7. The carry over factor is $\frac{1}{2}$.

So, moment developed is $\frac{M}{2}$.

8. As per Maxwell reciprocal theorem, the deflection at any point 'A' due to some load at another point 'B' will be equal to the deflection at point 'B' due to the same amount of load at 'A'.

9. Maximum dia. $= \frac{\text{slab thickness}}{8} = \frac{75}{8} = 9.37$ mm

So, available diameter = 8 mm

10. The section in which steel and concrete reach at their permissible limits simultaneously is called a 'balanced-section'.

11. $P_L = \theta m f_c$

Where, $\theta \rightarrow$ Creep coefficient

$m \rightarrow$ Modular ratio

$f_c \rightarrow$ Stress in concrete at the level of steel

12. The maximum spacing should be the lesser of

(i) Nominal maximum size of aggregate + 5mm

(ii) Greater diameter of bar used as main bar

13. Strain at failure $= \varepsilon = \frac{0.87\ f_y}{E_S} + 0.002$

$$\varepsilon = \frac{0.87 \times 415}{2 \times 10^5} + 0.002$$

$$\varepsilon = 0.0038$$

14. The strength of the joint = 5000 kg (minimum of all)

$$\therefore \text{ Number of Rivets } = \frac{100\times10^3}{5000} = 20 \text{ rivets}$$

15. It is a location at which curvature is infinite and moment is equal to plastic moment capacity.

16. The weld strength $\Rightarrow F = (t_t \times L)\times f_s$

$$F = (0.707 \times 7) \times 1 \times (10 \times 9.81)$$
$$= 485.49 \text{ N/mm}$$

Length of Weld = $\pi D = 150\ \pi$ mm

$$\text{Resisted-Twisting moment} = F.l.\frac{D}{2} = 485.49 \times 150\pi \times 75$$
$$= 17158632.95 \text{ N-mm} = 17.15 \text{ kN-m}$$

17. Degree of saturation is the ratio of volume of water to volume of voids.

18. According to Darcy

$V = Ki$

K = coefficient of permeability

i = hydraulic gradient

V = velocity of flow through voids

19.
$$q_u = \gamma D_F\left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

$D_F \rightarrow$ Depth of foundation

20.
$$K_P = \frac{1+\sin\phi}{1-\sin\phi}$$
$$= \frac{1+\sin 60°}{1-\sin 60°} = \frac{2+\sqrt{3}}{2-\sqrt{3}}$$
$$K_P = 13.92$$

[PART : B]

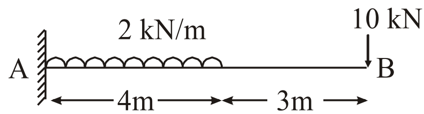
21. Hoop stress is more than longitudinal stress.

$$\text{Hoop-stress} = \frac{Pd}{2t} = \text{ultimate strength}$$

Given $d = 6$ cm, $t = 3$ mm $= 0.3$ cm

$$P = \frac{3600\times2\times0.3}{6} = 360 \text{ kg/cm}^2$$

22.



Bending moment at A = $(BM)_A$

$$= -\left\{10\times(4+3)+\left(2\times4\times\frac{4}{2}\right)\right\}$$

$$(BM)_A = -\{70 + 16\}$$

$$(BM)_A = -86 \text{ kN-m}$$

23. Maximum deflection for the given beam

$$\delta = \frac{Wl^4}{8EI}$$

$$\delta = \frac{6\times(4\times10^3)^4}{8\times45\times10^{12}}$$

$$= 4.27 \text{ mm}$$

$$\delta = 4.27 \text{ mm}$$

24. Let vertical distance of point 'X' is y

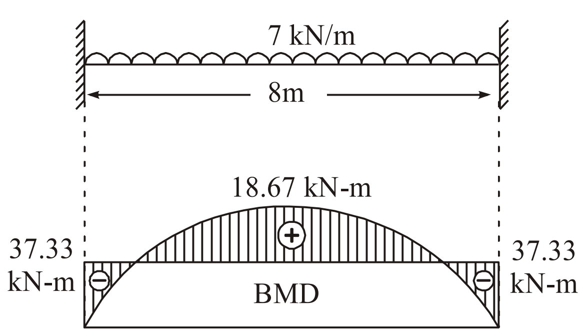
$$y = \frac{4hx}{l^2}(l-x)$$

$$y = \frac{4\times4\times3}{(10)^2}(10-3) = 3.36 \text{ m}$$

$$\therefore \quad BM_X = (100 \times 3) - (85 \times 3.36)$$

$$BM_X = 14.4 \text{ kN-m}$$

25.



Fixed end moment

$$= (-)\frac{Wl^2}{12} = (-)\frac{7\times8\times8}{12}$$

$$= (-)\ 37.33\ \text{kN-m}$$

Moment at center

$$= \frac{Wl^2}{24} = \frac{7\times8\times8}{24}$$

$$= 18.67\ \text{kN-m}$$

26. Minimum reinforcement in an RCC-column as per IS : 456-2000 is 0.8% of gross-area.

$$A_{sc} = \frac{0.8}{100}\times300\times500$$

$$A_{sc} = 1200\ \text{mm}^2$$

27.

$$MR_{lim} = 0.36 f_{ck} B x_{u_{lim}} (d - 0.42\, x_{u_{lim}})$$

$$= 0.36\times20\times 400 \times (0.48 \times 600)\ (d - 0.42 \times 0.48 \times 600)$$

$$MR_{lim} = 397135872\ \text{N-mm}$$

$$MR_{lim} = 397.13\ \text{kN-m}$$

28. Effective Span of Cantilever RC-Beam

The effective span of a cantilever shall be taken as it's length to the face of support plus half the effective depth except where it forms the end of a continuous-beam where the length to the center of support shall be taken.

29. Shape factor is the ratio of plastic section modulus to elastic section modulus of any beam.

$$\text{Shape-factor} = \frac{Z_p}{Ze}$$

Section	**Shape-factor**
Rectangular	1.5
Circular	1.33

30. A rigid joint doesn't permit any rotation. So moment rotation curve will be a vertical line.

While flexible and pin joints can not take any moment. So, moment-rotation curve will be a horizontal-line.

31. Types of transported Soil

(i) Alluvial Soil : Soils, that are deposited from suspension in running water.

(ii) Lacustrine Soil : Soils, that have been deposited from suspension in still water.

(iii) Marine Soil : Soil, deposited from suspension in sea-water.

(iv) Aeolian-Deposit : Soils that have been deposited by transportation of wind.

(v) Glacial-Deposit : Deposits, transported by ice.

32. The plasticity properties are mostly found in clay soils. And the minerals causing these properties are:-

(i) Mont montmorillonite

(ii) Illite

(iii) Kaolinite

[PART : C]

33. Fixed end moments:

$$M_{FAB} = -\frac{Wl^2}{12} = -\frac{8\times4\times4}{12}$$

$$= -10.67 \text{ kN-m}$$

$$M_{FBA} = +\frac{Wl^2}{12} = +\frac{8\times4\times4}{12}$$

$$= +10.67 \text{ kN-m}$$

$$M_{FBC} = -\frac{WL}{8} = -\frac{12\times6}{8}$$

$$= -9.0 \text{ kN-m}$$

$$M_{FCB} = +\frac{WL}{8} = +\frac{12\times6}{8}$$

$$= +9.0 \text{ kN-m}$$

Distribution factors, at joint 'B' :

For BA $\Rightarrow \frac{I}{L} = \frac{I}{4}$

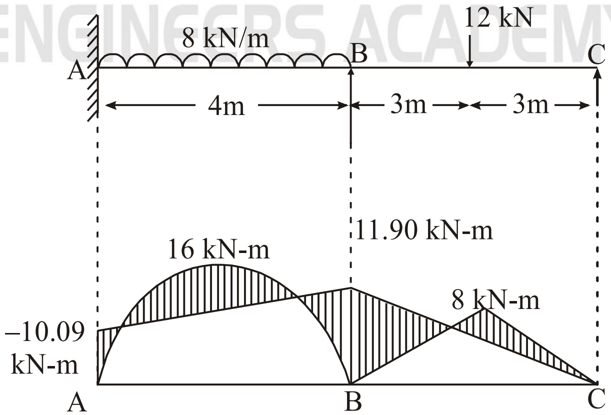
For BC $\Rightarrow \frac{3I}{4L} = \frac{3I}{4\times6} = \frac{I}{8}$

$$\text{D.F.} = \frac{I}{4} + \frac{I}{8} = \frac{3I}{8}$$

$$DF_1 = \frac{I/4}{3I/8} = \frac{2}{3}$$

$$DF_2 = \frac{I/8}{3I/8} = \frac{1}{3}$$

S.No.	A	B (2/3)	B (1/3)	C	Remarks
1.	–10.67	+10.67	–9.0	9	FEM
	0	–1.11	–0.56	–9	Balance
2.	–0.55	0	–4.5	–2.8	Carry-over
	0	3	+1.5	+2.8	Balance
3.	1.5	0	+1.4	0.75	Carry-over
	0	–0.933	–0.466	–0.75	Balance
4.	–0.46	0	–0.38	–0.233	Carry-over
	0	+0.253	–0.127	+0.233	Balance
5.	+0.13	0	+0.12	0.06	Carry-over
	0	0.08	–0.04	–0.06	Balance
6.	+0.04	0	–0.03	0.02	Carry-over
	0	0.02	+0.01	–0.02	Balance
	–10.09	11.90	–11.90	0	**Total**



34. Types of Column :

There are three types of RC column

(i) Long-column

(ii) Short-column

(iii) Pedestal

Codal-Provisions

(a) Minimum number of bars used in RC column is 4 for Rectangular columns and 6 for circular-columns.

(b) Minimum diameter of bar = 12 mm

(c) Minimum longitudinal Reinforcement

$= 0.8\%$ of A_g

(d) Maximum longitudinal Reinforcement

$4\% \rightarrow$ When bars are lapped

$6\% \rightarrow$ When bars are not lapped

(e) Maximum spacing between main bars

= 300 mm

(f) Lateral ties are used for preventing the buckling of main reinforcement

For Lateral Reinforcement (Tie bars):

(g) Diameter of Bar used :

$$\left.\begin{array}{l} \phi_L = 6\text{mm} \\ \dfrac{\phi_{main}}{4} \end{array}\right\} \text{more}$$

(h) Maximum spacing between the ties shall be lesser of the following :

(i) 48 × dia of tie bars

(ii) 16 × dia of main bars

(iii) least lateral dimension

(iv) 300 mm

(i) Helical Reinforcement

Minimum Pitch : It shall not be less than the more of the following

(i) 25 mm

(ii) 3 × dia of bar used for helix

(j) Maximum Pitch : The pitch of the helix shall be limited to the lesser of following :

(i) 75 mm

(ii) $\frac{\text{Diameter of Core}}{6}$

35. For Fe410 grade of steel f_u = 410 MPa

For bolts of grade 4.6, f_{ub} = 400 N/mm^2

γ_{mb} = Partial safety factor for material of bolt

= 1.25

The relevant properties of ISHB150@300.19 N/m are

Gauge, g = 90 mm

Flange - thickness, t_f = 9.0 mm

The factored end reaction is transmitted on to the two bracket plates as shown in fig. Thus the load for which the bracket connection is to be designed is

$$P = \frac{400}{2} = 200 \text{ kN}$$

Let us provide 20 mm diameter bolts of grade 4.6

For 20 mm dia bolt

$$d_0 = 20 + 2 = 22 \text{ mm}$$

Edge-Distance $e = 33$ mm

Stress-Area $A_{nb} = 245$ mm^2

Minimum pitch $P = 2.5 \times 20$

= 50 mm (let 60 mm)

Strength of bolt in single shear

$$V_{sb} = A_{nb}\frac{f_{ub}}{\sqrt{3}\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3}\times 1.25}\times 10^{-3}$$

$$= 45.26 \text{ kN}$$

Strength of bolt in bearing

$$V_{Pb} = 2.5\ K_b\ dt\frac{f_u}{\gamma_{mb}}$$

'K'$_b$ is the least of $\frac{e}{3d_0} = \frac{33}{3\times 22} = 0.50$

$$\Rightarrow \quad \frac{P}{3d_0} - 0.25 = \frac{60}{3\times 22} - 0.25 = 0.659$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410}$$

So, $\qquad K_b = 0.5$

$$V_{Pb} = 2.5\times 0.50\times 20\times 9\times \frac{400}{1.25}\times 10^{-3} = 72\ \text{kN}$$

Hence, strength of bolt = V_{sd} = 45.26 kN

Let us provide bolts in two rows

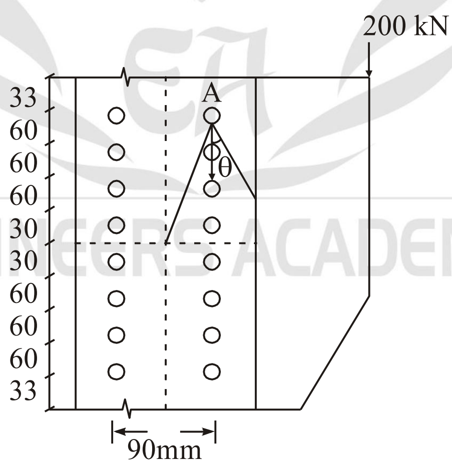
$$n = \sqrt{\frac{6M}{Pn'V_{sd}}} = \sqrt{\frac{6\times 200\times 250}{60\times 2\times 45.26}}$$

$$= 7.32 \simeq 8$$

Provide 20 mm bolts, on each bracket plate with 8 bolts in each row.

Force on critical bolt A

Direct force = $F_1 = \frac{P}{n} = \frac{200}{16} = 12.5$ kN



The force in bolt due to torque

$$F_2 = \frac{Pl_0 r_n}{\Sigma r^2}$$

$\Rightarrow \qquad l_0 = 250$ mm

$$r_n = \sqrt{210^2 + 45^2} = 214.76 \text{ mm}$$

$$\cos\theta = \frac{45}{\sqrt{210^2 + 45^2}}$$

$$\cos\theta = 0.21$$

$$\Sigma r^2 = 4[(210^2 + 45^2) + (150^2+45^2) + (90^2 + 45^2) + (30^2 + 45^2)]$$

$$= 334,800 \text{ mm}^2$$

$$F_2 = \frac{200 \times 250 \times 214.76}{334800} = 32.07 \text{ kN}$$

$$F = \sqrt{(12.5)^2 + (32.07)^2 + (2 \times 12.5 \times 32.07 \times 0.21)}$$

$= 36.78$ kN < 45.26 kN

36. Let the effective cover be 40 mm

So total size = 500 × (700 + 40)

$B \times D = 500 \times 740$

So, Self weight of the beam

$= 0.5 \times 0.7 \times 1 \times 25$

$= 8.75$ kN/m

So, Bending moment due to self weight

$$BM = \frac{Wl^2}{8} = \frac{8.75 \times 5^2}{8} = 27.34 \text{ kN-m}$$

So, Total Bending Moment

$= 27.34 + 455 = 482.34$ kN-m

Ultimate Bending Moment

$= 1.5 \times 482.34 = 732.51$ kN-m

Now for design,

$$(BM)_u = MR_{lim}$$

$$MR_{lim} = 0.36\, f_{ck} B x_{u_{lim}} (d - 0.42\, x_{u_{lim}})$$

$$= 0.36 \times 20 \times 500 \times (0.48 \times 700)$$

$$(d - 0.42 \times 0.48 \times 700)$$

$$MR_{lim} = 675.68 \text{ kN-m}$$

$$BM_U = 732.51 \text{ kN-m}$$

$$BM_U > MR_{lim}$$

→ So, Design over reinforced-section.

Let $MR_{lim} = M_1$

$$M_1 = 675.68 \times 10^6 = 0.87\, f_y A_{st_1} (d - 0.42\, x_{u_{lim}})$$

$$\Rightarrow \quad A_{st_1} = \frac{675.68 \times 10^6}{0.87 \times 415 \times (700 - 0.42 \times 0.48 \times 700)}$$

$$\Rightarrow \quad A_{st_1} = 3350.21 \text{ mm}^2$$

$$\Rightarrow \quad M_2 = BM_u - M_1$$

$$\Rightarrow \quad M_2 = 732.51 - 675.68 = 56.83 \text{ kN-m}$$

$$\Rightarrow \quad M_2 = 56.83 \times 10^6 = 0.87\, f_y A_{st_2}(d - d')$$

$$A_{st_2} = \frac{56.83 \times 10^6}{0.87 \times 415(700 - 40)} = 238.49 \text{ mm}^2$$

$$A_{st} = A_{st_1} + A_{st_2} = 3588.69 \text{ mm}^2$$

$$M_2 = (f_{sc} - 0.45\, f_{ck}) A_{sc} (d - d')$$

$$A_{sc} = \frac{56.83 \times 10^6}{(0.87 \times 415 - 0.45 \times 20)(700 - 40)}$$

$$A_{sc} = 244.58 \text{ mm}^2$$

Provide 14 mm Bars in compression and 25 mm bars in tension.

37. When the back fill consists more than one layer, the lateral pressure distribution for each of the layers is worked out.

$$K_{A_1} \text{ for upper layer} = \frac{1 - \sin 30°}{1 + \sin 30°} = \frac{1}{3} = 0.33$$

$$K_{A_2} \text{ for lower layer} = \frac{1 - \sin 34°}{1 + \sin 34°} = 0.283$$

where K_A → Active earth pressure coefficient.

⇒ Active earth pressure distribution for the upper layer.

at $Z = 0$ m; Vertical pressure ⇒ $P_v = 0$

Active pressure ⇒ $P_a = 0$

at $Z = 5$ m;

Vertical pressure

$$\Rightarrow \quad P_v = 18 \times 5 = 90 \text{ kN/m}^2$$

Active pressure

$$\Rightarrow \quad P_a = K_A \cdot P_v$$

$$p_a = 0.33 \times 90 = 30 \text{ kN/m}^2$$

Now, active earth pressure for the lower level

at $Z = 5m$;

Vertical pressure

$$P_v = \gamma.h$$

$$\Rightarrow \quad P_v = 90 \text{ kN/m}^2$$

$\therefore$ Active pressure

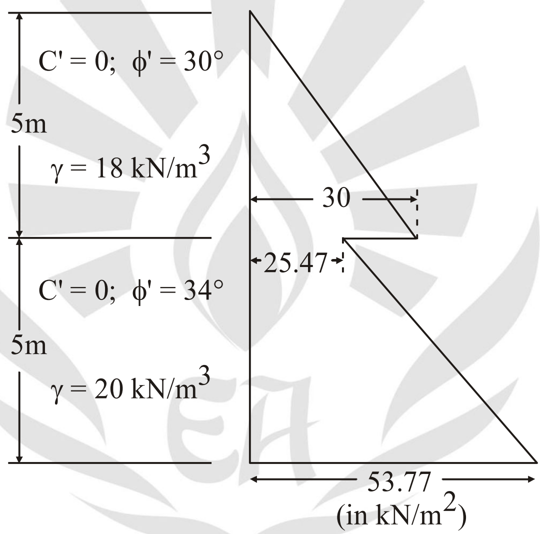
$$\Rightarrow \quad P_A = K_2.p_V$$

$$P_A = 0.283 \times 90 = 25.47 \text{ kN/m}^2$$

at $Z = 10m$, $p_v = 90 + (20 \times 5) = 190 \text{ kN/m}^2$

$\therefore$ Active-pressure $\Rightarrow P_A = 0.283 \times 190 = 53.77 \text{ kN/m}^2$

The active earth pressure distribution is given below



In reality, there can not be a sudden change in lateral pressure since shear stresses which develop along the interface have not been considered. But this does not introduce any serious error in the magnitude and the direction of the resultant thrust.

38. Change in void ratio = $\Delta e = e_0 - e$

$$\Rightarrow \quad \Delta e = 1.20 - 1.10 = 0.10$$

Change in effective stress $\Rightarrow \Delta\bar{\sigma} = \bar{\sigma}_2 - \bar{\sigma}_1$

$$\Delta\bar{\sigma} = 0.50 - 0.25 = 0.25 \text{kgf/cm}^2$$

Coff. of compressibility $(a_v) = \dfrac{\Delta e}{\Delta\bar{\sigma}}$

$$a_v = \frac{0.10}{0.25} = \frac{2}{5} = 0.4 \text{ cm}^2/\text{kgf}$$

Coff. of volume compressibility - (m_v)

$$m_v = \frac{a_v}{1+e_0} = \frac{0.4}{1+1.2} = 0.18 \text{ cm}^2/\text{kgf}$$

C_v = Coefficient of consolidation

$C_v = 10 \text{ m}^2/\text{year}$

$$C_v = \frac{10\times10^4}{1\times365\times24\times60\times60} = 3.17 \times 10^{-3} \text{ cm}^2/\text{sec}$$

For coefficient of permeability :

Consolidation Equation

$$K = C_v m_v \gamma_w$$

$$K = 3.17\times10^{-3}\times0.18\times1000\times10^{-6}$$

$$K = 5.7 \times 10^{-7} \text{ cm/sec}$$

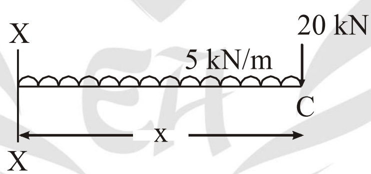
Consolidation-Settlement :

$$S_c = H_0\frac{\Delta e}{(1+e_0)} = \frac{3\times0.10}{1+1.20}$$

$$S_c = 0.136 \text{ m}$$

39. For shear force and Bending Moment

(i) In Section BC



Shear force at X – X section

$$SF_x = (20) + (5 \times x)$$

$$SF_x = 5x + 20$$

at $x = 0,\ (SF)_C = 20$ kN

at $x = 6$ m, $(SF)_B = (5 \times 6) + 20 = 50$ kN

For B.M. at X – X Section

$$BM_X = (-20x) - \left(5x\cdot\frac{x}{2}\right)$$

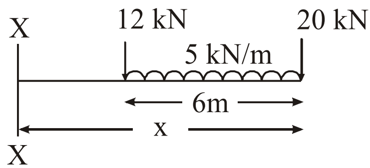
$$BM_X = -20x - 2.5x^2$$

at $x = 0,\ (BM)_C = 0$

at $x = 6\text{m},\ (BM)_B = -120 - 90 = -210$ kN-m

(ii) In Section AB :

Shear force :



$$SF_X = +20 + (5 \times 6) + 12$$

SF for AB section is constant

$$(SF)_{A-B} = 62 \text{ kN}$$

Bending Moment

$$(BM)_X = (-20 \times x) - 5 \times (6)(x-3) - 12(x-6)$$

For AB Section

at $x = 6m$

$$(BM)_B = (-20 \times 6) - (5 \times 6 \times 3) - 12(0)$$

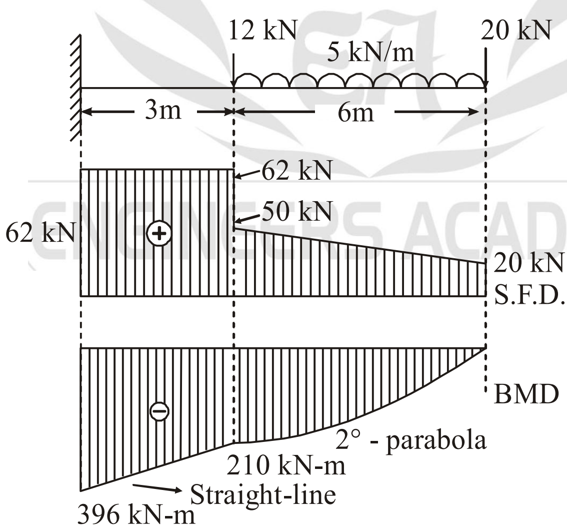
$$BM = -210 \text{ kN-m}$$

at $x = 9m$

$$(BM)_A = (-20 \times 9) - (5 \times 6 \times 6) - (12 \times 3)$$

$$(BM)_A = -396 \text{ kN-m}$$

By the above calculated data:



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