DETAILS EXPLANATIONS

1. **(A)** Let vertical distance of point 'X' is y

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

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

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

$BM_x = 14.4$ kN-m

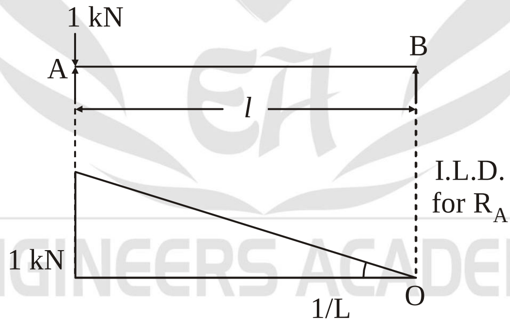
(B) For two hinged semi-circular arch with load 'w' applied at any section, the radius vector corresponding to which makes an angle θ with horizontal

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

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

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

(C)



(D) 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.

2. **Fixed end moments:**

$$M_{FAB} = -\frac{Wl^2}{12} = -\frac{8 \times 4 \times 4}{12}$$

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

$$M_{FBA} = +\frac{Wl^2}{12} = +\frac{8\times4\times4}{12} = +10.67 \text{ kN-m}$$

$$M_{FBC} = -\frac{WL}{8} = -\frac{12\times6}{8} = -9.0 \text{ kN-m}$$

$$M_{FCB} = +\frac{WL}{8} = +\frac{12\times6}{8} = +9.0 \text{ kN-m}$$

Distribution factors, at joint 'B'

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

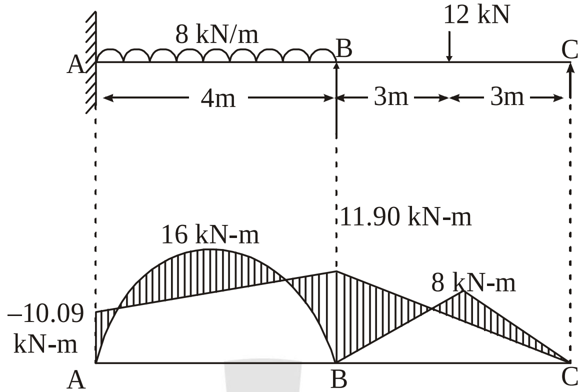
For BC $\Rightarrow \frac{3I}{4L} = \frac{3I}{4\times6} = \frac{I}{8}$

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

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

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

S.No.	A	B (2/3)	B (1/3)	C	Remarks
1.	–10.67	+10.67	–9.0	9	FEM
	0	–1.11	–5.6	–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**



3. **(A) 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 :

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

(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) $\dfrac{\text{Diameter of Core}}{6}$

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

$= 0.36 \times 20 \times 400 \times (0.48 \times 600)$

$(d - 0.42 \times 0.48 \times 600)$

$MR_{lim} = 397135.872$ N-mm

$MR_{lim} = 397.13$ kN-m

4. Let the effective cover be 40 mm

So total size = 500 × (700 × 40)

B × D = 500 × 740

So, Self weight of the beam

= 0.5 × 0.7 × 1 × 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 × 482.34 = 732.51 kN-m

Now for design,

$(BM)_u = MR_{lim}$

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

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

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

$MR_{lim} = 675.68$ kN-m

$BM_V = 732.51$ kN-m

$BM_V > MR_{lim}$

$\rightarrow$ So, Design over reinforced-section.

Let $MR_{lim} = M_1$

$M_1 = 675.68 \times 10^6$

$= 0.87\ f_y A_{st_1}\ (d - 0.42\ x_{u_{lim}})$

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

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

$\Rightarrow M_2 = BM_u - M_1$

$\Rightarrow M_2 = 732.51 - 675.68$

$= 56.83$ kN-m

$\Rightarrow M_2 = 56.83 \times 10^6$

$= 0.87\ f_y A_{st_2}(d - d')$

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

$= 238.49\ mm^2$

$A_{st} = A_{st_1} + A_{st_2}$

$= 3588.69\ mm^2$

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

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

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

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

5. **(A)** The weld strength $\Rightarrow F = (t_t \times L) \times f_S$

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

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

Resisted-Twisting moment $= F.l.\frac{D}{2} = 485.49 \times 150\pi \times 75$

$= 17158632.95$ N-mm

$= 17.15$ kN-m

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

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

(C) *Black Bolts :*

(i) Hexagonal black bolts are generally used in steel works.

(ii) They are made from low or medium carbon steels.

(iii) They are designated as black bolts M × d × *l*

Where, d = diameter and *l* = length of bolt.

Precision and Semi Precision Bolts :

(i) They are also known as close tolerance bolts.

(ii) Sometimes to prevent excessive slip, close tolerance bolts are provided in holes of 0.15 to 0.2 mm oversize this may cause difficulty in alignment and delay in the progress of work.

6. ***As per IS : 800 - 1984:***

Unless the outer edge of each stiffner is continously stiffened, the outstand of all stiffners from the web should net be more than

$$\frac{256t}{\sqrt{f_y}}.$$

Where vertical stiffners are required, they should be provided throughout the length of the girder at a distance greater than 1.5 d_1 and not less than 0.33 d_1.

When horizontal stiffners are provided d_1 be should be taken as clear distance between the horizontal stiffner and tension flange ignoring fillets.

The moment of inertia(I) of a pair of vertical-stiffeners about the center of web or a single stiffner about the face of the web should be,

$$I \geq \frac{1.5d_1^3t^3}{C^2}$$

where, t = Minimum required thickness of web

C = The maximum permitted clear distance between vertical stiffner for thickness 't'.

Sometimes vertical stiffners are subjected to external-forces and therefore the moment of inertia of the stiffner should be increased as described below.

(a) Bending moment on stiffner due to eccentricity of vertical loading with respect to vertical axis of web.

$$I = \frac{150MD^2}{Et_w} cm^4$$

(b) Lateral loading on stiffner

$$I = \frac{0.3VD^3}{Et_w} cm^4$$

For first horizontal stiffner at $\left(\frac{2}{5}\right)^{th}$ of the distance between compression flange and neutral axis for the compression flange.

$$I \geq 4\ C\ t^3$$

where, t = Minimum thickness of web required.

C = Actual distance between vertical stiffner.

I = Moment of inertia of horizontal stiffner pair.

For second horizontal stiffner at neutral axis:

$$I \geq D_2 t^3$$

Stiffners are not connected to web to withstand a shearing force not less than $\frac{125t_w^2}{h} kN/m$

where, h = Outstand of stiffner in mm.

7. When the back fill consists more than one layer, the lateral pressure distribution for each of the layer 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 $K_A \rightarrow$ Active earth pressure coefficient.

$\Rightarrow$ Active earth pressure distribution for the upper layer.

at $Z = 0$ m; Vertical pressure $\Rightarrow P_v = 0$

Active pressure $\Rightarrow P_a = 0$

at $Z = 5$ m;

Vertical pressure

$$\Rightarrow P_v = 18 \times 5$$

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

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 $Z = 5\text{m}$;

Vertical pressure

$P_v = \gamma.h \Rightarrow P_v = 90 \text{ kN/m}^2$

$\therefore$ Active pressure

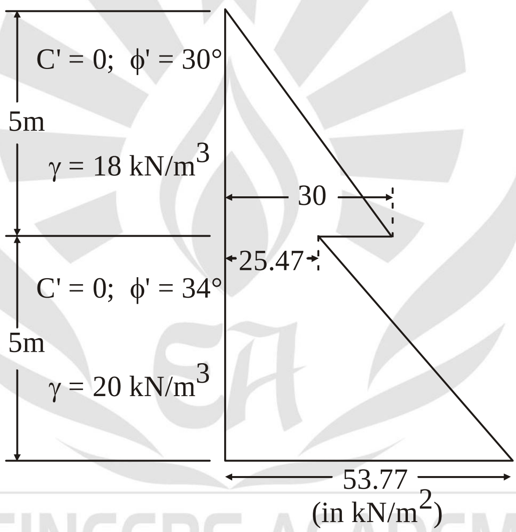
$\Rightarrow \quad P_A = K_2.P_V$

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

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

$\therefore$ 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.

8. **(A)** $K_P = \dfrac{1+\sin\phi}{1-\sin\phi}$

$$= \frac{1+\sin 60°}{1-\sin 60°} = \frac{2+\sqrt{3}}{2-\sqrt{3}}$$

$K_P = 13.92$

(B) Change in void ratio = $\Delta e = e_0 - e$

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

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

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

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)

$$m_v = \frac{a_v}{1+l_0}$$

$$= \frac{0.4}{1+1.2}$$

$$= 0.18 \text{ cm}^2/\text{kgf}$$

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

$$S_C = H_0\frac{\Delta e}{(1+e_0)}$$

$$= \frac{3\times0.10}{1+1.20}$$

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