MADE EASY

India's Best Institute for IES, GATE & PSUs

Detailed Solutions

ESE-2021 Mains Test Series

Civil Engineering Test No : 14

Q.1 (a) Solution:

(i) **Bleeding :** It is defined as the autogenous flow of mixing water within or emergence to the surface from freshly placed concrete which is usually due to excessive vibration imparted to concrete to achieve full compaction. However well the concrete may have been compacted, the force of gravity tends to pull the heavy solid particles downward, the lighter water being displaced upwards. This upward migration of water is known as bleeding. It ceases either when the solid particles touch each other and cannot settle any more or when the concrete stiffens due to cement hydration and prevents further movement.

(ii) **Laitance:** Laitance defined as cement and water slurry coming on top and setting on the surface. It is very dangerous since the top surface will weather out fast with larger shrinkage cracks. If laitance is formed in a lift, it should be removed before next lift is placed.

(iii) **Segregation:** Segregation usually implies separation of (a) coarse aggregate from fine aggregate (b) paste from coarse aggregate, or water from the mix and the ingredients of the fresh concrete no longer remain uniformly distributed. It can be reduced by increasing small size coarse aggregate air entrainment, using dispersing agents and pozzolana.

The causes of segregation are dropping concrete from heights, badly designed mixes, concrete carried over long distances—pumping, belt conveyor system etc. over vibrations, and during concrete finishing extra floating and tamping. Segregation mainly occurs in dry non-sticky concrete mixes.

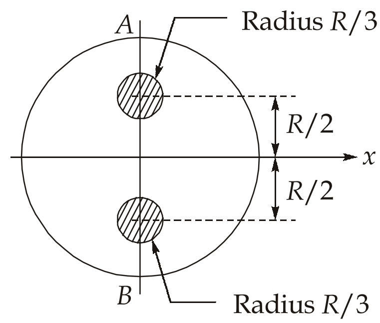
Q.1 (b) Solution:

The moment of inertia of a solid circular cross-section about a diametrical axis is $\frac{\pi R^4}{4}$.

Using this value for solid section and subtracting the moments of inertia of each the holes about the same diameter axis (from the parallel axis theorem) we have,

$$I = \frac{\pi R^4}{4} - 2\left\{\frac{\pi}{4}\left(\frac{R}{3}\right)^4 + \pi\left(\frac{R}{3}\right)^2 \times \left(\frac{R}{2}\right)^2\right\}$$

$$\Rightarrow \quad I = 0.591R^4$$



The bending stress in the uppermost and lower most fibres, denoted by points A and B, respectively will be the maximum.

$$\sigma_{max} = \frac{M}{Z} \qquad \text{where } Z = \frac{I}{R} = 0.591R^3$$

$$\Rightarrow \quad 600 \times 10^6 = \frac{P \times 1.5}{0.591R^3}$$

Substituting $R = 0.1$ m

We get, $P = 236.4 \times 10^3 \text{ N} = 236.4 \text{ kN}$

Q.1 (c) Solution:

We have, $E = 2G(1 + \mu)$

$$\Rightarrow \quad \mu = \frac{E}{2G} - 1 = \frac{1.9 \times 10^5}{2 \times 0.75 \times 10^5} - 1 = 0.267$$

Also,

$$K = \frac{E}{3(1-2\mu)} = \frac{1.9 \times 10^5}{3(1-2 \times 0.267)} = 1.359 \times 10^5 \text{N/mm}^2$$

For maximum percentage error in the derived value of μ, the error in the values of E and G should be of different signs. Let percentage error in E be +1 and that in G be –1.

Now, $\mu = \frac{E}{2G} - 1$

Hence, $\mu' = \frac{E'}{2G'} - 1$

where, E' = Incorrect value of E = $(1.9 \times 10^5) \times 1.01$

G' = Incorrect value of G = $(0.75 \times 10^5) \times 0.99$

μ' = Computed incorrect value of μ

$$\therefore \quad \mu' = \frac{1.9 \times 10^5 \times 1.01}{2(0.75 \times 10^5 \times 0.99)} - 1 = 0.2923$$

$$\text{Percentage error in } \mu = \frac{\mu' - \mu}{\mu} \times 100$$

$$= \frac{0.2923 - 0.267}{0.267} \times 100 = 9.48\%$$

Q.1 (d) Solution:

Let us assume: V = SF at any section x-x

M = BM at any section x-x

w_x = Loading intensity

$$\therefore \quad w_x = \frac{dV}{dx}; \; V = \frac{dM}{dx}$$

Now, from SFD

(i) At A: There is a sudden increase in negative SF, suggesting a downward acting point load of magnitude 2 kN at A.

(ii) Between A and B: SF is constant, and hence there is no load.

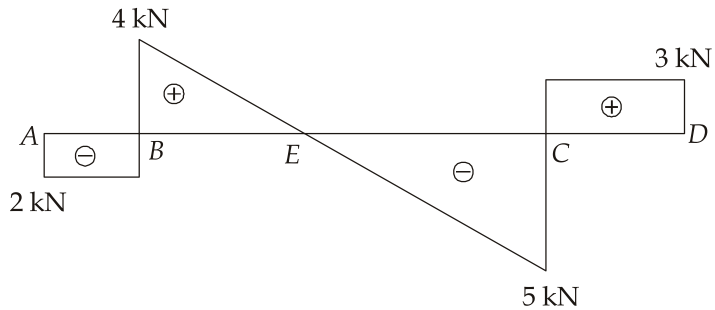
(iii) At B: There is a sudden increase in SF from –2 kN to +4kN, suggesting a support, the value of reaction being equal to 2 + 4 = 6 kN.

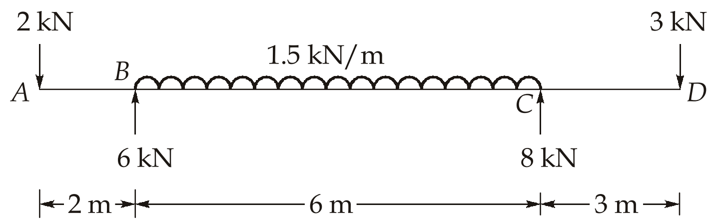
(iv) Between B and C: The SF decreases linearly from +4 kN to –5 kN, suggesting the existence of UDL of value $\frac{4-(-5)}{6} = 1.5$ kN/m

(v) At C: There is sudden change in SF from –5 kN to +3 kN, hence the other support is at C, the value of reaction being equal to 5 + 3 = 8 kN.

(vi) Between C and D: SF is constant and there is no load.

(vii) At D: The SF decreases from +3 to zero, hence a point load of 3 kN acts at the free end D in the downward direction.





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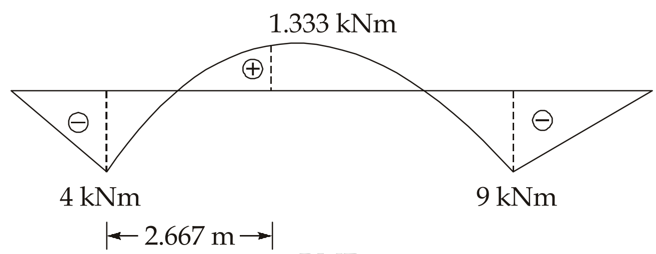


BMD

Check: Total reaction $= 6 + 8 = 14$ kN

Total load $= 2 + 1.5 \times 6 + 3 = 14$ kN

BM Diagram:

Let shear force is zero at distance, $\frac{x}{4} = \frac{6-x}{5}$ from B.

$$5x = 24 - 4x$$

$$9x = 24$$

$$x = \frac{8}{3}\text{ m} = 2.667 \text{ m}$$

Hence

$$M_{max}(+) = 6 \times 2.667 - 2(2 + 2.667) - \frac{1.5(2.667)^2}{2}$$

$$= +1.333 \text{ kNm}$$

Also, $M_B = -2 \times 2 = -4$ kNm and $M_c = -3 \times 3 = -9$ kNm

Between A and B, SF is –ve and constant,

$\therefore$ Slope of BMD is –ve and constant between B to E, SF is +ve and decreasing.

$\therefore$ Slope of BMD is +ve and decreasing between E to C, SF is –ve and incresing

$\therefore$ Slope of BMD is –ve and increasing between C to D SF is +ve and constant.

$\therefore$ Slope of BMD is +ve and constant.

Q.1 (e) Solution:

Factors affecting alkali-aggregate reaction

1. High alkali content in cement

   Alkali content in cement is usually restricted to 0.6%.

2. Reactive nature of aggregates.

3. Availability of moisture

   Water is needed for alkali-aggregate reaction.

   Deterioration of concrete due to alkali-aggregate reaction usually does not occur in the interior of bulk concrete.

4. Temperature

   Temperature in the range of 10 to 40°C promotes alkali-aggregate reaction.

Control of alkali-aggregate reaction:

1. By controlling the voids in cement
2. By controlling the moisture content and temperature
3. By the use of low alkali cement
4. By the use of non-reactive aggregate
5. By use of admixtures

Q.2 (a) Solution:

(i) Nominal dimension : Nominal size of bricks is the size including the thickness of the mortar. The thickness of the mortar in the brickwork should not exceed 10 mm.

Size	Ordinary bricks	
	Metric (cm)	FPS (inch)
Actual	$19 \times 9 \times 9$	$8\frac{7}{8} \times 8\frac{3}{8} \times 2\frac{3}{4}$
Nominal	$20 \times 10 \times 10$	$9 \times 4\frac{1}{2} \times 3$

Weight of brickwork: The average weight of the brick is about 3 to 3.5 kg depending on its denseness.

(ii) According to percentage of calcium oxide and clayey impurities in it, lime can be classified as lean, hydraulic and fat lime.

1. Lean or poor lime : It consists of CaO and MgO where CaO is about 80-85% with MgO less than 5% and clayey impurities of about more than 7% in the form of silica, alumina and iron-oxide. It sets on absorbing CO_2 from atmosphere.

Characteristics:

- Slaking requires more time and so it hydrates slowly. Its expansion is less than that of fat lime.
- It makes thin paste with water.
- Setting and hardening is very slow.
- Its colour varies from yellow to grey.

Uses: It gives poor and inferior mortar and is recommended for less important structures.

2. Hydraulic lime : It is the product obtained by moderate burning (900°C – 1100°C) of raw limestone which contains small proportions of clay (silica and alumina, about 5-30%) and iron oxide in chemical combination with calcium oxide (CaO + MgO with CaO of about 70-80% and MgO less than 5%). Depending upon percentage of clay it is classified as:

Feebly hydraulic lime	Moderately hydraulic lime	Eminently hydraulic lime
< 5 - 10% of silica and alumina i.e. impurities	10 - 20% impurities	20 - 30% impurities
Slaking time 5 - 15 min	Slaking time 1 - 2 hours	Slakes very slowly
Setting time 21 days	Setting time 7 days	Setting time - 2 hours
Used in damp places	Used in damp places	Used in damp places & for all structural purposes

3. Pure, Rich or Fat Lime : It is a soft lime (CaO + MgO where CaO is more than 85% with MgO less than 4%) obtained by calcination of nearly pure limestone, marble, chalk powder, oolitic limestone and calcareous tufa.

Also known as white washing lime, does not have impurities of clay and stones more than 5%. Fat lime is nearly pure calcium oxide and when it is hydrated with the required amount of water, the solid lumps form a soft fine powder of $Ca(OH)_2$ and high heat of hydration produces a cloud of steam.

Characteristics:

- Slaking is vigorous and the volume becomes 2-3 times.
- It sets slowly in contact with air and hence not suitable for thick walls or in wet climate.

- Specific gravity of pure lime is about 3.4.
- If kept under water, fat lime does not lose its plasticity and consequently does not set and gets hard.

Uses: Fat lime finds extensive use in making mortar, matrix for concrete, base for distemper and in whitewash, manufacturing of cement and in metallurgical industry.

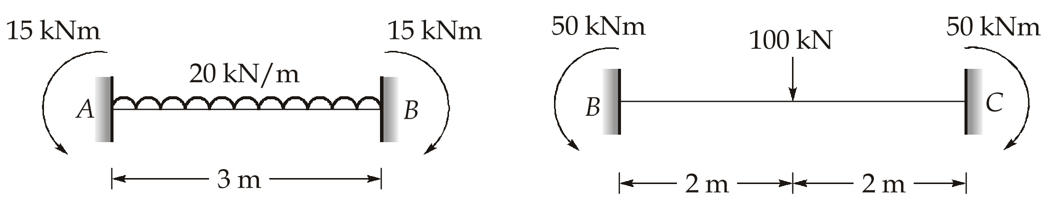
Q.2 (b) Solution:

Fixed end moments:

$$M_{FAB} = -\frac{20\times 3^2}{12} = -15 \text{ kNm}$$

$$M_{FBA} = +\frac{20\times 3^2}{12} = +15 \text{ kNm}$$

$$M_{FBC} = -\frac{100\times 4}{8} = -50 \text{ kNm}$$



Fixed end moments

Equivalent joint loads:

$$M_B = M_{B,\, nodal} - (M_{FBA} + M_{FBC})$$

$\Rightarrow$ $$M_B = 0 - (15 - 50) = +35 \text{ kNm}$$

We know that

$$[P] = [P]_L + [P]_\Delta$$ $[P]_L$ is joint load at joint B

$\Rightarrow$ $$\begin{bmatrix} 0 \\ 80 \end{bmatrix} = \begin{bmatrix} -35 \\ 50 \end{bmatrix} + [P]_\Delta$$

$\Rightarrow$ $$[P]_\Delta = \begin{bmatrix} 35 \\ 30 \end{bmatrix}$$

Stiffness matrix:

$$k_{11} = \frac{4EI}{3} + \frac{4EI}{4} = \frac{7EI}{3}$$

$$k_{21} = \frac{2EI}{4} = \frac{EI}{2}$$

$$k_{22} = \frac{4EI}{4} = EI$$

$$k_{12} = \frac{2EI}{4} = \frac{EI}{2}$$

$$[P]_\Delta = \begin{bmatrix} 35 \\ 30 \end{bmatrix}$$

$$\therefore \quad k = EI\begin{bmatrix} 7/3 & 1/2 \\ 1/2 & 1 \end{bmatrix}$$

$$\Rightarrow \quad \begin{bmatrix} \theta_B \\ \theta_C \end{bmatrix} = k^{-1}\begin{bmatrix} 35 \\ 50 \end{bmatrix} = \frac{12}{EI \times 25} \times \begin{bmatrix} 1 & -\frac{1}{2} \\ -\frac{1}{2} & \frac{7}{3} \end{bmatrix}\begin{bmatrix} 35 \\ 30 \end{bmatrix}$$

$$\Rightarrow \quad \begin{bmatrix} EI\theta_B \\ EI\theta_C \end{bmatrix} = \begin{bmatrix} 9.6 \\ 25.2 \end{bmatrix}$$

$$M_{AB} = -15 + \frac{2EI}{3}[0 + \theta_B]$$

$$\Rightarrow \quad M_{AB} = -15 + \frac{2}{3} \times 9.6 = -8.6 \text{ kNm}$$

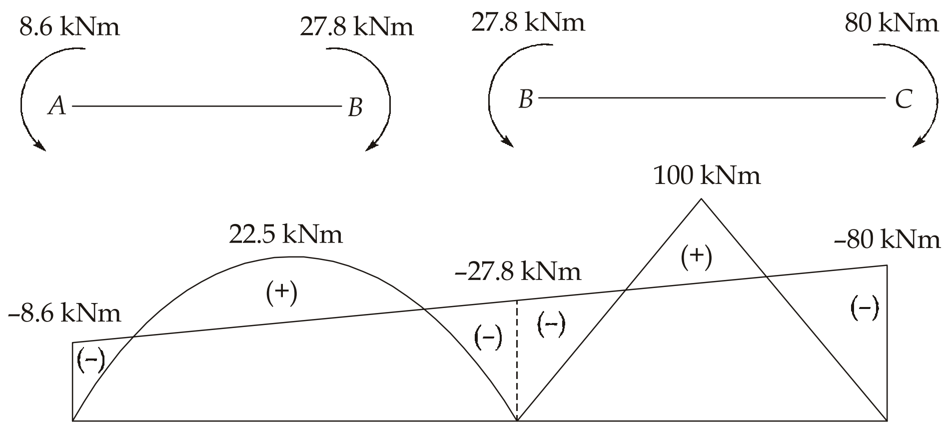
$$M_{BA} = 15 + \frac{2EI}{3}[2\theta_B]$$

$$\Rightarrow \quad M_{BA} = 15 + \frac{2}{3} \times 2 \times 9.6 = 27.8 \text{ kNm}$$

$$M_{BC} = -50 + \frac{2EI}{4}(2\theta_B + \theta_C)$$

$$\Rightarrow \quad M_{BC} = -50 + \left(9.6 + \frac{25.2}{2}\right) = -27.8 \text{ kNm}$$

$$M_{CB} = 50 + \frac{2EI}{4}[2\theta_C + \theta_B] = 50 + \left[25.2 + \frac{9.6}{2}\right] = 80 \text{ kNm}$$



BMD

Q.2 (c) Solution:

The third principal stress (σ) may be the highest, the lowest or the intermediate one. Hence the range is $\sigma_3 \le \sigma \le \sigma_1$.

(i) Maximum principal stress theory

$$\sigma_1 = f_y = 210 \text{ N/mm}^2$$
$$\sigma_3 = f_y' = -210 \text{ N/mm}^2$$

Hence the range is $-210 \le \sigma \le 210 \text{ N/mm}^2$

(ii) Maximum strain theory

$$\sigma_3 - 0.25\,(130 + 90) = f_y = 210 \text{ N/mm}^2$$
$$\Rightarrow \quad \sigma_3 = 210 + 55 = 265 \text{ N/mm}^2$$
or
$$\sigma_3 - 0.25\,(130 + 90) = f_y' = -210$$
$$\sigma_3 = -210 + 55 = -155 \text{ N/mm}^2$$
$$\therefore \quad -155 \le \sigma_3 \le 265 \text{ N/mm}^2$$

(iii) Maximum shear stress theory

$$\sigma_1 - 90 = f_y = 210$$
$$\Rightarrow \quad \sigma_1 = 210 + 90 = 300 \text{ N/mm}^2$$
Also,
$$130 - \sigma_3 = f_y = 210$$
$$\Rightarrow \quad \sigma_3 = 130 - 210 = -80 \text{ N/mm}^2$$

Hence, the range is $-80 \le \sigma \le 300 \text{ N/mm}^2$

(iv) Maximum strain energy theory

$$\sigma^2 + 90^2 + 130^2 - 2 \times 0.25\,(130\sigma + 90\sigma + 90 \times 130) = (210)^2$$
$$\Rightarrow \quad \sigma^2 - 110\sigma - 19150 = 0$$

We get, $\sigma_1 = 203.9\ \text{N/mm}^2$

And $\sigma_3 = -93.9\ \text{N/mm}^2$

Hence the range is $-93.9 \le \sigma \le 203.9\ \text{N/mm}^2$

(v) Maximum distortion energy theory

$(\sigma - 90)^2 + (\sigma - 130)^2 + (130 - 90)^2 = 2(210)^2$

$\Rightarrow \quad \sigma^2 - 220\sigma - 30800 = 0$

We get, $\sigma_1 = 317.1\ \text{N/mm}^2$

And $\sigma_3 = -97.1\ \text{N/mm}^2$

Hence the range is $-97.1 \le \sigma \le 317.1\ \text{N/mm}^2$

Q.3 (a) Solution:

Since parabolic arch is given,

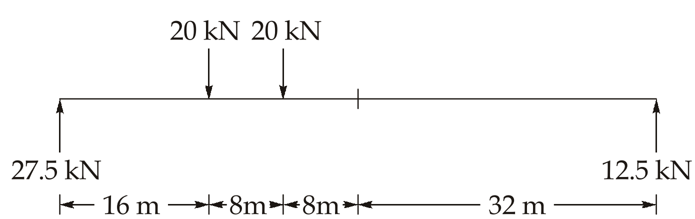
Equation of arch is, $$y = \frac{4hx}{l^2}(l-x)$$

$$= \frac{4\times 12.8}{64^2}\left(64x - x^2\right) = \frac{1}{80}\left(64x - x^2\right)$$

$$H = \frac{-\int \frac{Mydx}{EI}}{\int \frac{y^2dx}{EI}}$$

Calculating $\int My\,dx$:

Here M is defined for equivalent beam moment.



$M_x = 27.5x \qquad (0 < x < 16\text{ m})$

$M_x = 27.5x - 20(x - 16) \qquad (16\text{ m} < x < 24\text{ m})$

$= 7.5x + 320$

$M_x = 27.5x - 20(x - 16) - 20(x - 24) \qquad (24\text{ m} < x < 64\text{ m})$

$= 800 - 12.5x$

So,
$$\int \frac{Mydx}{EI} = \int_0^{16}(27.5x)\times\frac{(64x-x^2)}{80}dx + \int_{16}^{24}(7.5x+320)\frac{(64x-x^2)}{80}dx + \int_{24}^{64}(800-12.5x)\frac{(64x-x^2)}{80}dx = \frac{0.17894\times10^6}{EI}$$

Now,
$$\int \frac{y^2dx}{EI} = \int_0^{64}\frac{\left[\frac{1}{80}(64x-x^2)\right]^2}{EI} = \frac{5592.4053}{EI}$$

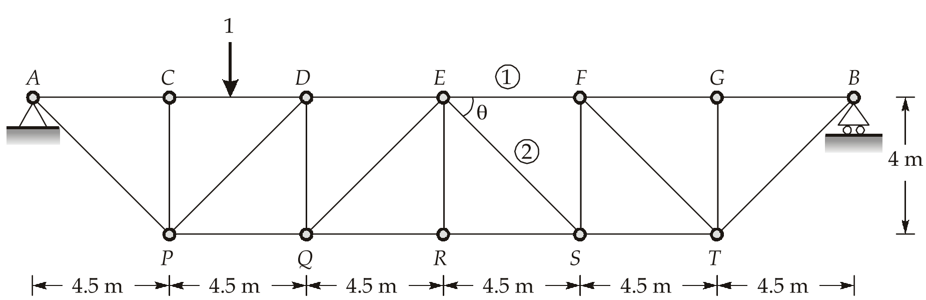
∴ Horizontal reaction,
$$H = -\frac{0.17894\times10^6}{5592.4053} = -31.997\text{ kN} \simeq -32\text{ kN}$$

Q.3 (b) Solution:

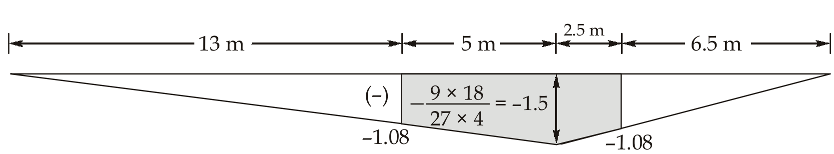
Design force in member '1': The axial force is compressive under dead load and live load.

$$d = \sqrt{4^2+4.5^2} = 6.0208\text{ m}$$

$$\frac{d}{h} = \frac{6.0208}{4} = 1.5052$$



(a) Loading diagram



(b) IL for force in member '1'

For member 1

Taking $x = 0$ at A, keeping unit load behind F,

$$\Sigma M_s = 0$$

$$\Rightarrow \quad N_1 = -\frac{V_B \times 4.5 \times 2}{4} = -2.25 V_B \text{ (Compressive)}$$

Design load calculation for member (1)

1. Due to dead load $= -\frac{1}{2} \times 4.5 \times 6 \times 20 \times 1.5 = -405$ kN
2. Due to live load

To achieve maximum load$= \frac{w_1 x_1}{l_1} = \frac{w_2 x_2}{l_2}$

But $\quad w_1 = w_2$

$$\therefore \quad \frac{x_1}{4.5 \times 4} = \frac{x_2}{4.5 \times 2}$$

$$\Rightarrow \quad x_1 = 2x_2$$

And total length of live load is 7.5 m

So, $\quad x_1 = 5$ m, $\quad x_2 = 2.5$ m

$$\therefore \quad \text{Design live load} = -40 \times \left[\left(\frac{1.08+1.5}{2}\right) \times 5 + \left(\frac{1.08+1.5}{2}\right) \times 2.5\right]$$

$$= -387 \text{ kN}$$

Total design force, $\quad N_1 = -405 - 387 = -792$ kN

For member (2)

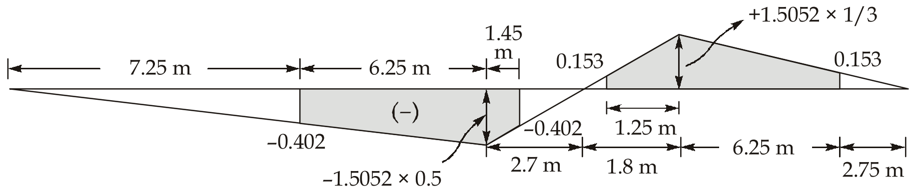
When load is left to point R

We can write $\quad F_2 \times \sin\theta = V_B$

$$F_2 = \frac{V_B}{\sin\theta} = 1.505 V_B \quad \text{(Compressive)} \left[\tan\theta = \frac{4}{4.5}\right]$$

When load is ahead of point S

$$F_2 = \frac{V_A}{\sin\theta} = 1.505\, V_A \quad \text{(Tensile)}$$



(c) IL for force in member '2'

Design load calculation for member (2):

1. Maximum compressive force

$$\text{Due to dead load} = -20\times\left[\frac{0.75\times16.2}{2}-\frac{10.8\times0.5}{2}\right] = -67.5\text{ kN}$$

2. Due to live load

Compressive force,

$$\frac{wx_1}{3\times4.5} = \frac{wx_2}{2.7}$$

$$\Rightarrow \quad x_1 = 5x_2 \qquad (\because w_1 = w_2)$$

$$\text{But} \quad x_1 + x_2 = 7.5\text{ m}$$

$$\therefore \quad x_2 = \frac{7.5}{6} = 1.25\text{ m}$$

$$x_1 = 6.25\text{ m}$$

$$\therefore \quad \text{Design live load} = -40\times\left[\left(\frac{0.75+0.402}{2}\right)\times6.25+\left(\frac{0.75+0.402}{2}\right)\times1.45\right]$$

$$= -177.408\text{ kN}$$

$$\text{Total design force} = -67.5 - 177.408 = -244.908\text{ kN}$$

$$\text{For maximum tensile force} = \frac{wx_1}{1.8} = \frac{wx_2}{4.5\times2}$$

$$\Rightarrow \quad 5x_1 = x_2$$

$$\text{But} \quad x_1 + x_2 = 7.5\text{ m}$$

$$\therefore \quad x_1 = 1.25\text{ m}, \quad x_2 = 6.25\text{ m}$$

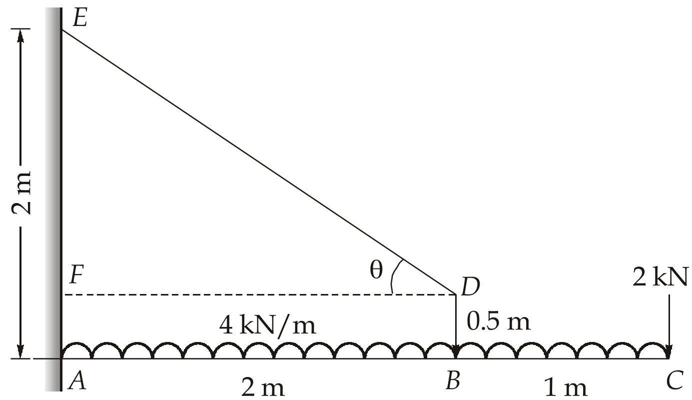
$$\therefore \quad \text{Live load} = 40\times\left[\left(\frac{0.153+0.5}{2}\right)\times1.25+\left(\frac{0.153+0.5}{2}\right)\times6.25\right]$$

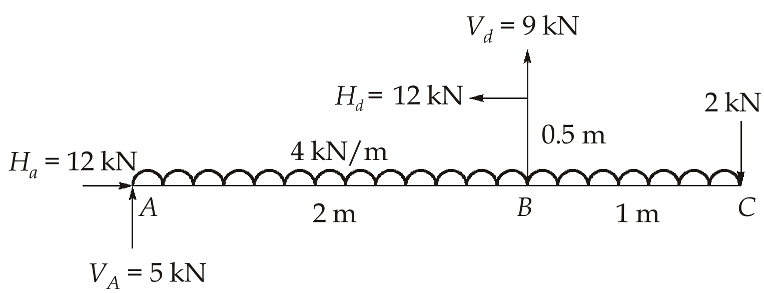
$$= 97.95\text{ kN}$$

$$\therefore \text{ Total design force}, \quad N_2 = -67.5 + 97.95 = 30.45\text{ kN} \qquad \text{(Tensile)}$$

Q.3 (c) Solution:

Let V_e and H_e be the vertical and horizontal reactions at E. Let V_a and H_a be the vertical and horizontal reactions at A.





For the equilibrium of the whole structure, taking moments about E, we get

$$H_a \times 2 = \frac{4\times 3^2}{2} + 2\times 3$$

$$\Rightarrow \qquad H_a = 12 \text{ kN}$$

Resolving the forces on the structure horizontally, we get,

$$H_e = H_a = 12 \text{ kN}$$

Let the tension in the tie *ED* be *T*

Resolving the forces at *E* horizontally and vertically, we get

$$T\cos\theta = H_e$$

$$T\sin\theta = V_e$$

$$\therefore \qquad \tan\theta = \frac{V_e}{H_e}$$

In ΔEDF,

$$\tan\theta = \frac{EF}{DF} = \frac{1.5}{2} = \frac{3}{4}$$

$$\therefore \qquad V_e = \frac{3}{4}H_e = \frac{3}{4}\times 12 = 9 \text{ kN}$$

Resolving the forces on the whole structure vertically, we get,

$$V_a = 4 \times 3 + 2 - 9 = 5 \text{ kN}$$

Now consider the equilibrium of the beam ABC.

This part is in equilibrium under the action of the following forces:

(i) External loading on the beam

(ii) Vertical reaction at $A = V_1 = 5$ kN

(iii) Horizontal reaction at $A = H_1 = 12$ kN

(iv) Vertical component of the tension T at $D = V_d = T \sin\theta = 9$ kN

(v) Horizontal component of the tension T at $D = H_a = \text{T} \cos\theta = 12$ kN

The effect of the forces $H_d = 12$ kN and $V_d = 9$ kN is the same as that of an upward force of 9 kN and an anticlockwise couple of $12 \times 0.5 = 6$ kNm at B.

SF Calculations:

At any section in AB distant x from A the SF is given by

$$S = 5 - 4x$$

At $x = 0$, $\quad S = +5$ kN

At $x = 2$ m, i.e., just on the left side of B,

$$S = 5 - 4 \times 2 = -3\text{kN}$$

S.F. is zero at a distance x from A given by the condition $5 - 4x = 0$

i.e., at $\quad x = \frac{5}{4}$ m

SF just on the right side of $B = -3 + 9 = +6$ kN

From B to C the SF will change uniformly from +6 kN to +2 kN

BM Calculation:

At any section in AB distant x from A the BM is given by

$$M = 5x - \frac{4x^2}{2} = 5x - 2x^2$$

At A, i.e., at $x = 0$, $M = 0$

At $x = 2$ m i.e., just on the left side of B.

$$M = 5 \times 2 - 2 \times 2^2 = +10 - 8 = +2\text{kNm}$$

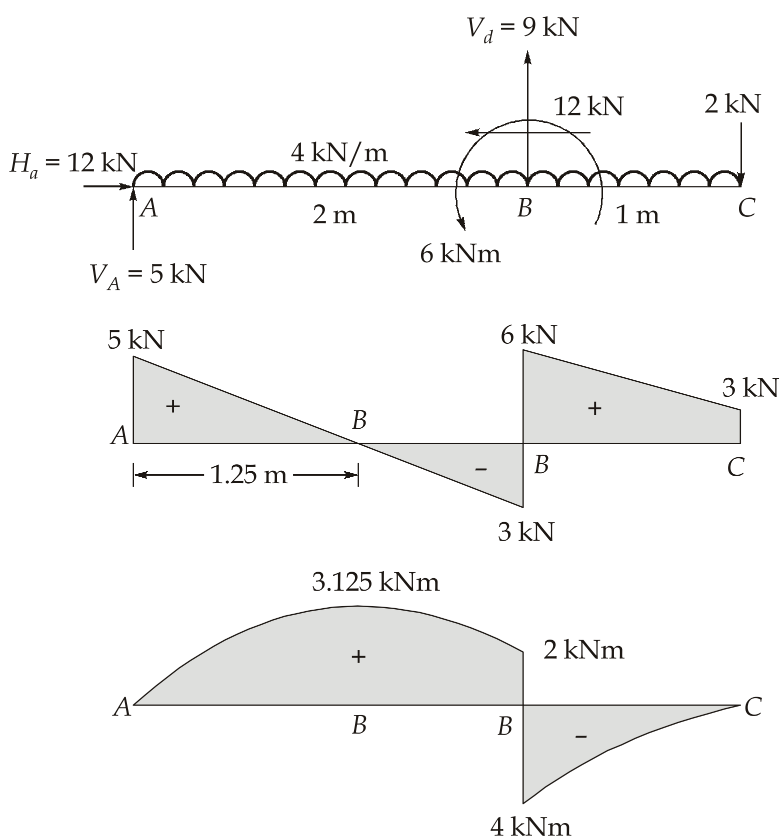
At $x = \frac{5}{4}$ m i.e., where is SF is zero,

$$\text{BM} = \frac{5\times5}{4} - 2\times\frac{25}{16} = 3.125 \text{ kNm}$$

BM just on the right side of B = +2 – 6 = –4 kNm or alternatively,

$$\text{BM on right of } B = (2\times1)+\left(4\times1\times\frac{1}{2}\right) = 4 \text{ kNm}$$

BM at C = 0.



Q.4 (a) Solution:

(i) Deterioration of stones:

The various natural agents such as rain, heat, etc. and chemicals deteriorate the stones with time.

Rain: Rain water acts both physically and chemically on stones. The physical action is due to the erosive and transportation powers and the latter due to the decomposition, oxidation and hydration of the minerals present in the stones.

Physical action: Alternate wetting by rain and drying by sun causes internal stresses in the stones and consequent disintegration.

Chemical action: In industrial areas, acidic rain water reacts with the constituents of stones leading to its deterioration.

Decomposition: The disintegration of alkaline silicate of alumina in stones is mainly because of the action of chemically active water. The hydrated silicate and the carbonate forms of the alkaline materials are very soluble in water and are removed in solution leaving behind a hydrated silicate of alumina (Kaolinite). The decomposition of felspar is represented as

$$\underbrace{K_2Al_2O_3.6H_2O}_{\text{(Orthoclase)}} + CO_2 + nH_2O \longrightarrow \underbrace{K_2CO_2}_{\text{(Alkaline carbonate)}} + \underbrace{Al_2O_3.2SiO_2.2H_2O}_{\text{(Kaolinite)}} + \underbrace{4SiO_2.nH_2O}_{\text{(Hydrated silicate)}}$$

Oxidation and hydration: Rock containing iron compounds in the forms of peroxide, sulphide and carbonate are oxidised and hydrated when acted upon by acidulated rain water. As an example of peroxide—FeO is converted into ferric oxide—Fe_2O_3 which combines with water to form $FeO.nH_2O$. This chemical change is accompanied by an increase in volume and results in a physical change manifested by the liberation of the neighbouring minerals composing the rocks. As another example, iron sulphide and siderite readily oxidize to limonite and liberates sulphur, which combines with water and oxygen to form sulphuric acid and finally to sulphates.

(ii) Causes and remedies of timber decay

Decay due to fungal and Bacterial attack : Wood is essentially organic substance made up of skeleton of cellulose impregnated with lignin. Fungi are system of plant organisms which live in and attack timber causing rot and decay. Fungi reproduce through spores which send out mycelia which in turn destroy the wood tissues by secretions of solvents and enzymes. Basic requirements for the existence of fungi are moisture, suitable temperature and food supplies.

Control of fungal and bacterial attack :

- Dryness of timber and it should not be subjected to alternate wet and dry conditions.
- Felled trees should be air dried as early as possible and sawn timber should be kiln seasoned properly in accordance with good air-seasoning practice.
- Adequate ventilation around the timber to prevent fungal attack.

Damages due to insects :

Termites : White ants are the most destructive termites. They completely excavate the wood at the centre leaving the outer shell intact. They also attack furniture and wood work in houses and railway sleepers etc.

Beetles : These are small insects and they cause rapid decay of timber by converting them into fine powder. Usually, the outer shell of timber remains intact and hence the timber looks sound until the timber fails completely.

Carpenter ants : They are usually black in colour and vary in size within the same nest. They do not eat wood but merely tunnel it out for habitation. They normally attack slightly rolled or water softened wood.

Control of insects :

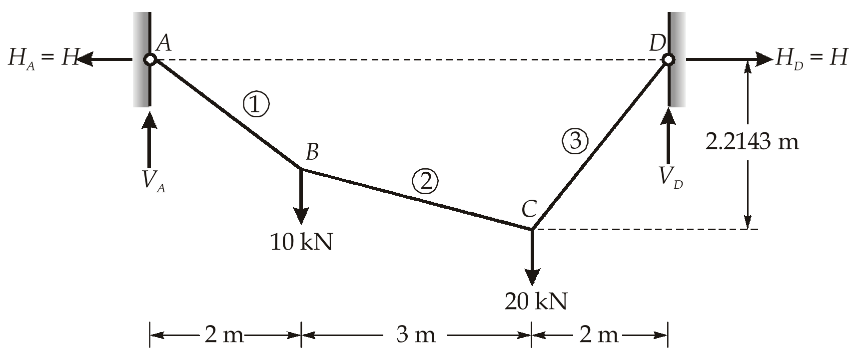
- Large scale fumigation is carried out using powerful hydrocyanic acid gas.
- The best alternative is turpentine mixed with a small quantity of orthodichlorobenzene. This vapour is very deadly to insects.

Damage due to rodents : They are more serious in food handling establishments.

Control of Rodents :

The guiding principle is to close all openings or passages and making doors and windows capable of closure in a tight manner by fixing metal sheets over the lower part of doors.

Q.4 (b) Solution:



Support reactions:

Consider the overall free-body.

$$\Sigma M_A = 0$$

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

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

$$\Sigma F_y = 0$$

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

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

Consider 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 $\Sigma F_y = 0$,

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

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

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

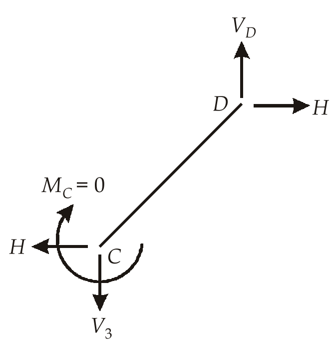
The axial tension in the i^{th} segment is given by

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

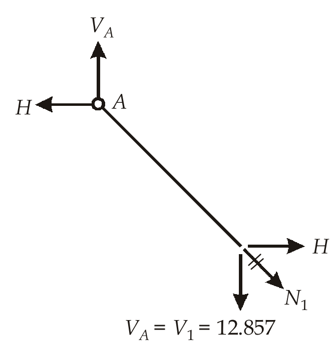
$$\therefore \quad N_1 = \sqrt{(12.857)^2 + (15.484)^2} = 20.126 \text{ kN}$$

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

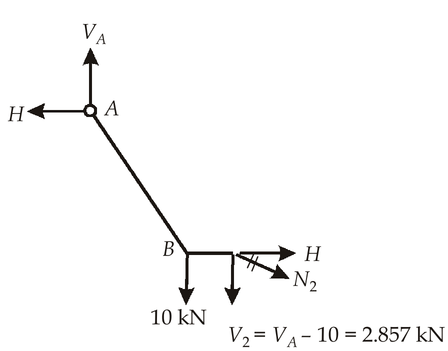
$$N_3 = \sqrt{(17.143)^2 + (15.484)^2} = 23.101 \text{ kN}$$



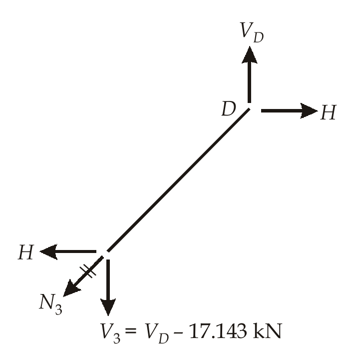
(a) Free body DC[+]



(b) Section through segment 1



(c) Section through segment 2



(d) Section through segment 3

Sag at B

Considering the free-body of the segment AB,

$$M_B = 0$$

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

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

Length of the cable

Total cable length, $S = S_1 + S_2 + S_3$

$$= \sqrt{(2)^2 + (1.6607)^2} + \sqrt{(3)^2 + (2.2143 - 1.6607)^2} + \sqrt{(2)^2 + (2.2143)^2}$$

$$= \sqrt{6.758} + \sqrt{9.306} + \sqrt{8.903} = 8.634 \text{ m}$$

Q.4 (c) Solution:

Support reactions:

$$\Sigma M_A = 0$$

$$\Rightarrow \quad V_B \times l = 2W \times \frac{l}{2} + W \times \left(l + \frac{l}{4}\right)$$

$$\Rightarrow \quad V_B \times l = Wl + \frac{5Wl}{4}$$

$$\Rightarrow \quad V_B = \frac{9W}{4}$$

$$\Rightarrow \quad \Sigma F_y = 0$$

$$\Rightarrow \quad V_A + V_B = 2W + W$$

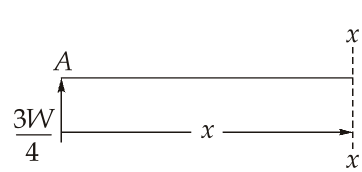
$$\Rightarrow \quad V_A = 3W - \frac{9W}{4}$$

$$\Rightarrow \quad V_A = \frac{3W}{4}$$

Bending moment diagram:

Span AD:

Take a section at x m from end A



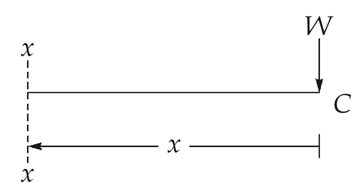
$$\therefore \quad BM_x = +\frac{3W}{4}\times x$$

$$\therefore \quad BM_A = 0 \qquad (\because x = 0)$$

$$BM_D = \frac{3W}{4}\times\frac{l}{2} = \frac{3Wl}{8} \qquad \left(\because x = \frac{l}{2}\text{m}\right)$$

Span BC:

Take a section at x m from end C



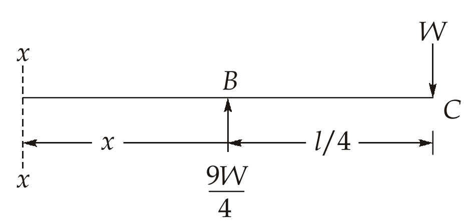
$$\therefore \quad BM_x = -Wx$$

$$\therefore \quad BM_C = 0 \qquad (\because x = 0)$$

$$BM_B = -W\times\frac{l}{4} = -\frac{Wl}{4} \qquad \left(\because x = \frac{l}{4}\text{ m}\right)$$

Span DB:

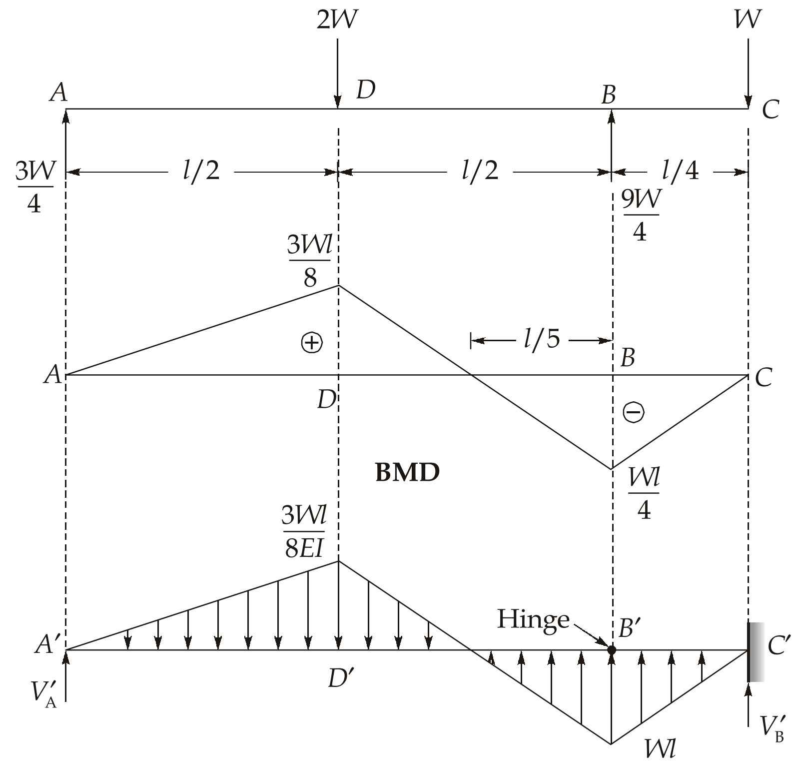
Take a section at x m from end B



$$BM_x = +\frac{9Wx}{4} - W\left(x+\frac{l}{4}\right)$$

$$\therefore \quad BM_B = -\frac{Wl}{4} \qquad (\because x = 0)$$

$$BM_D = +\frac{9W}{4}\left(\frac{l}{2}\right) - W\left(\frac{3l}{4}\right) = \frac{3}{8}Wl \qquad \left(\because x = \frac{l}{2}\text{m}\right)$$



Conjugate beam

$$\Sigma M_{B'} \text{(about hinge)} = 0$$

$$\Rightarrow \quad V'_A l + \frac{1}{2} \times \frac{l}{5} \times \frac{Wl}{4EI} \times \frac{1}{3} \times \frac{l}{5} = \frac{1}{2} \times \frac{l}{2} \times \frac{3}{8} \frac{Wl}{EI} \times \left(\frac{l}{2} + \frac{1}{3} \times \frac{l}{2} \right) + \frac{1}{2} \times \frac{3l}{10} \times \frac{3Wl}{8EI} \left[\frac{l}{5} + \frac{2}{3} \times \frac{3l}{10} \right]$$

$$\Rightarrow \quad V'_A = \frac{Wl^2}{EI} \left[\frac{1}{16} + \frac{9}{400} - \frac{1}{600} \right]$$

$$\Rightarrow \quad V'_A = \frac{Wl^2}{12EI}$$

Now,
$$V'_B = \frac{1}{2} \times \left(\frac{l}{2} + \frac{3l}{10} \right) \times \frac{3Wl}{8EI} - \frac{1}{2} \times \left(\frac{l}{5} + \frac{l}{4} \right) \times \frac{Wl}{4EI} - \frac{Wl^2}{12EI}$$

$$\Rightarrow \quad V'_B = \frac{3}{20} \frac{Wl^2}{EI} - \frac{9}{160} \frac{Wl^2}{EI} - \frac{Wl^2}{12EI}$$

$$= \frac{72Wl^2 - 18Wl^2 - 40Wl^2}{480EI}$$

$$= \frac{14Wl^2}{480EI} = \frac{7Wl^2}{240EI}$$

$\therefore$ Slope at A = SF at A in conjugate beam

$$\Rightarrow \quad \theta_A = V'_A = \frac{Wl^2}{12EI}$$

Slope at B = SF at B in conjugate beam

$$\Rightarrow \quad \theta_B = \frac{Wl^2}{12EI} + \frac{1}{2} \times \frac{l}{5} \times \frac{Wl}{4EI} - \frac{1}{2} \times \left(\frac{l}{2} + \frac{3l}{10}\right) \times \frac{3Wl}{8EI}$$

$$= \frac{Wl^2}{12EI} + \frac{Wl^2}{40EI} - \frac{3Wl^2}{20}$$

$$= \frac{10Wl^2 + 3Wl^2 - 18Wl^2}{120EI} = -\frac{Wl^2}{24EI}$$

Slope at C = SF at C in conjugate beam

$$\frac{Wl^2}{12EI} + \frac{1}{2}\left(\frac{l}{5} + \frac{l}{4}\right)\frac{Wl}{4EI} - \frac{1}{2}\left(\frac{l}{2} + \frac{3l}{10}\right)\frac{3Wl}{8EI}$$

$$\Rightarrow \quad \theta_C = \frac{Wl^2}{12EI} + \frac{9Wl^2}{160EI} - \frac{3Wl^2}{20EI}$$

$$= \frac{40Wl^2 + 27Wl^2 - 72Wl^2}{480EI} = -\frac{5Wl^2}{480EI} = -\frac{Wl^2}{96}$$

Deflection at D = Bending moment at D in conjugate beam

$$\Rightarrow \quad \delta_D = \frac{Wl^2}{12EI} \times \frac{1}{3} \times \frac{l}{2} - \frac{1}{2} \times \frac{l}{2} \times \frac{3Wl}{8EI} \times \frac{1}{3} \times \frac{l}{2}$$

$$= \frac{Wl^3}{72EI} - \frac{Wl^3}{64EI} = -\frac{Wl^3}{576EI}$$

Deflection at C = Bending moment at C in conjugate beam

$$\Rightarrow \quad \delta_C = \frac{Wl^2}{12EI} \times \frac{5l}{4} + \frac{1}{2} \times \frac{l}{4} \times \frac{Wl}{4EI} \times \frac{2}{3} \times \frac{l}{4} + \frac{1}{2} \times \frac{l}{5}$$

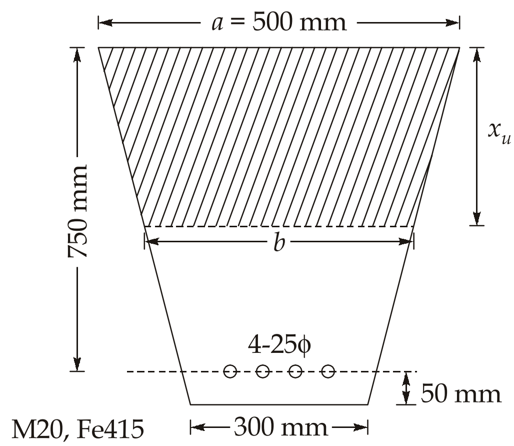
$$\times \frac{Wl}{4EI}\left(\frac{l}{4} + \frac{1}{3} \times \frac{l}{5}\right) - \frac{1}{2} \times \frac{3l}{10} \times \frac{3Wl}{8EI}\left(\frac{2}{3} \times \frac{3l}{10} + \frac{9l}{20}\right)$$

$$-\frac{1}{2} \times \frac{l}{2} \times \frac{3Wl}{8EI} \times \left(\frac{1}{3} \times \frac{l}{2} + \frac{3l}{4}\right)$$

$$\Rightarrow \qquad \delta_C = \frac{5Wl^3}{48EI} + \frac{Wl^3}{192EI} + \frac{19Wl^3}{2400EI} - \frac{5.85Wl^3}{160EI} - \frac{33Wl^3}{384EI}$$

$$= -\frac{Wl^3}{192EI}$$

Q.5 (a) Solution:



$$b = 300 + \frac{500 - 300}{800} \times 590$$

$$= 447.5 \text{ mm}$$

Assume depth of NA, $x_u = 210$ mm

Distance of centroid of reinforcement from NA

$$= 750 - 210 = 540 \text{ mm}$$

$$\therefore \qquad \varepsilon_s = 0.0035 \times \frac{540}{210} = 0.009 > 0.0038$$

(the yield strain of Fe415 steel)

∴ Steel will yield first.

Compression in concrete:

Area of compression block $= A_c$

$$A_c = \text{Average width} \times 210 \text{ mm}^2$$

$$\text{Average width} = \frac{1}{2}(500 + 447.5) = 473.75 \text{ mm}$$

$$A_c = 473.75 \times 210 = 99487.5 \text{ mm}^2$$

Average compressive stress in concrete stresses

$$= 0.36 f_{ck}$$

$$\therefore \qquad C = 0.36 f_{ck} A_c$$

$$= 0.36 \times 20 \times 99487.5 \times 10^{-3} \text{ kN}$$

$$= 716.31 \text{ kN}$$

Tension in steel, $T = 0.87 f_y A_{st}$

$$= 0.87 \times 415 \times 4 \times \frac{\pi}{4}(25)^2 \times 10^{-3} \text{ kN}$$

$$= 708.92 \text{ kN} \simeq 709 \text{ kN}$$

$\therefore$ $C \simeq T$ (Approx)

Let C.G. of compression stress block is at a distance 'y' from top fibre

$$\therefore \quad y = \frac{h}{3}\left[\frac{a+2b}{a+b}\right] = \frac{210}{3}\left[\frac{500+2\times 447.5}{500+447.5}\right] = 103.6 \text{ mm}$$

$\therefore$ Lever arm $= 750 - 103.6$

$\Rightarrow$ $Z = 646.94$ mm

Moment of resistance, $M_u = T \times Z = 709 \times 646.94 \times 10^{-3} = 458.68$ kNm

Q.5 (b) Solution:

Given: Steel grade Fe410 (E250)

$$f_y = 250 \text{ N/mm}^2$$
$$f_u = 410 \text{ N/mm}^2$$

Site welding

$\therefore$ $\gamma_{mw} = 1.5$

The design strength of the member is governed by yielding of gross-section.

$$T_{dg} = \frac{f_y}{\gamma_{m0}} A_g = \left(\frac{250}{1.1} \times 978\right) \times 10^{-3} \text{ kN} = 222.27 \text{ kN}$$

Therefore, the weld will be designed to transmit a force equal to 222.27 kN.

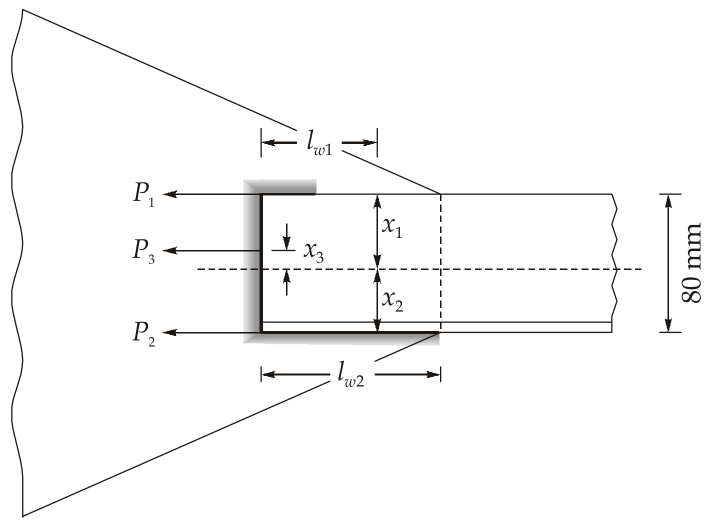
Size of weld:

Minimum size of weld for 12 mm thick plate = 5 mm

Maximum size of weld for 8 mm rounded edge = $\frac{3}{4} \times 8 = 6$ mm

$\therefore$ Adopt size of weld, $S = 6$ mm

$\therefore$ Throat thickness, $t_t = kS = 0.7 \times 6 = 4.2$ mm



$$x_2 = 27.3 \text{ mm}$$

$$x_1 = 80 - 27.3 = 52.7 \text{ mm}$$

$$x_3 = 40 - 27.3 = 12.7 \text{ mm}$$

P_1, P_2 = Load resisted by side fillet welds

P_3 = Load resisted by end fillet weld

$$\therefore \quad P_3 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} l_{w3} t_t$$

$$= \frac{410}{\sqrt{3} \times 1.5} \times 80 \times 4.2 \times 10^{-3} \text{ kN}$$

$$= 53.024 \text{ kN}$$

Since, $P_1 + P_2 + P_3 = T_{dg}$

$$\Rightarrow \quad P_1 + P_2 = 222.27 - 53.024 = 169.246 \text{ kN} \quad \text{...(i)}$$

Since welded joint is a rigid joint,

For moment free connection

$$P_1 x_1 + P_3 x_3 = P_2 x_2$$

$$\Rightarrow \quad P_2 x_2 - P_1 x_1 = P_3 x_3$$

$$\Rightarrow \quad -P_1 (52.7) + P_2 (27.3) = 53.024 \times 12.7 = 673.405 \quad \text{...(ii)}$$

Solving eq. (i) and (ii), we get

$$P_1 = 49.34 \text{ kN}$$

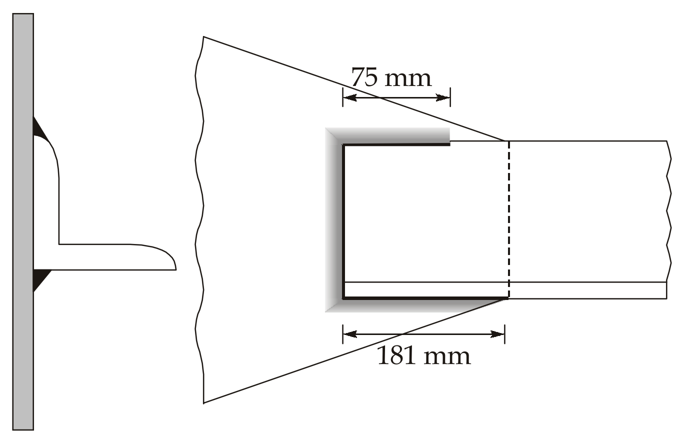
$$P_2 = 119.91 \text{ kN}$$

$$\therefore \quad P_{dw_1} = P_1 = \frac{410}{\sqrt{3}\times 1.5}\times l_{w_1}\times 4.2 = 49.34\times 10^3$$

$$\Rightarrow \quad l_{w1} = 74.44 \simeq 75 \text{ mm}$$

$$P_{dw_2} = P_2 = \frac{410}{\sqrt{3}\times 1.5}\times l_{w_2}\times 4.2 = 119.91\times 10^3$$

$$l_{w_2} = 180.91 \text{ mm} \simeq 181 \text{ mm}$$



Q.5 (c) Solution:

Rate Analysis : The method of determination of rate per unit of a particular item of work considering the cost of quantities of materials, the cost of labourers, hire of tools and plants, overhead charges, water charges, contractor's profit etc. is known as rate analysis.

Purpose of rate analysis :

- To determine the current rate per unit of an item of work at the locality.
- To examine the viability of rates quoted by the contractors.
- To ascertain the quantity of materials and labour strengths required to complete the project.
- To revise the schedule of rates due to increase in the cost of materials and labour or due to changed situations.

Factors affecting rate analysis: The following factors affect the rate of a particular item of work:

- Specifications of work and materials, quality of materials, proportion of mix, method of construction, operation etc.
- Quantities of materials and their rates.
- Number of different types of labour and their rates.

- Location of site of work and its distance from the sources of materials and rates of transport.
- Availability of water.
- Miscellaneous and overhead expenses of contractor.
- Location of site and its conditions.

Q.5 (d) Solution:

Total height of building, $h = 3.2 + 3.2 + 4.0 = 10.4$ m

$\therefore$ Natural period, $T = 0.075\, h^{3/4}$

$= 0.075\,(10.4)^{3/4} = 0.434$ sec

As $0.4 \leq T \leq 40$

$$\therefore \quad S_a/g = \frac{1}{T} = \frac{1}{0.434} = 2.3$$

As building is in zone-IV

$\therefore$ $Z = 0.24$

Importance factor, $I = 1.0$ (for residential building)

Response reduction factor, $R = 5.0$ (for RCC special moment resisting frame)

Base shear, $V_B = A_h \times W$

W = Seismic weight of building

$= W_1 + W_2 + W_3 = 294.3 + 1863.9 + 1079.1$

$= 3237.3$ kN

$$A_h = \left(\frac{Z}{2}\right)\left(\frac{I}{R}\right)\left(\frac{S_a}{g}\right)$$

$$= \left(\frac{0.24}{2}\right) \times \left(\frac{1}{5}\right) \times 2.3 = 0.0552$$

$\therefore$ $V_B = 0.0552 \times 3237.3 = 178.699 \text{ kN} \simeq 178.7$ kN

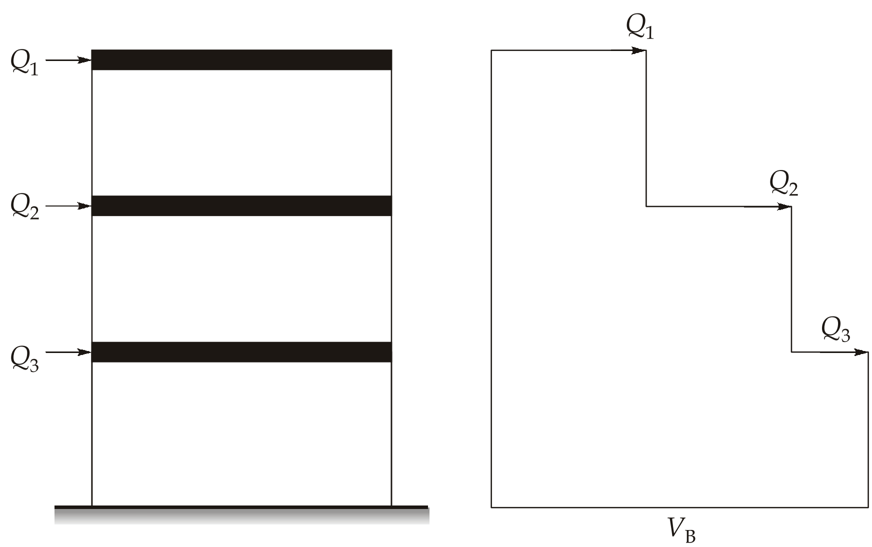
Computation of lateral forces (Q_i) at each storey:

Storey	Weight W_i (in kN)	h_i (m) from base	$W_i h_i^2$	$\frac{W_i h_i^2}{\Sigma W_i h_i^2} = \alpha_i$	$Q_i = \alpha_i V_B$ (kN)
Roof	294.3	10.4	31831.488	0.218	38.96
Second storey	1863.9	7.2	96624.576	0.663	118.48
First storey	1079.1	4	17265.6	0.118	21.09
Σ	3237.3		145721.664	1.00	178.53

$\therefore$ Base shear, $V_B = 178.7$ kN

and the lateral forces at the storey levels are:

$$Q_1 = 38.96 \text{ kN}$$
$$Q_2 = 118.48 \text{ kN}$$
$$Q_3 = 21.09 \text{ kN}$$



Q.5 (e) Solution:

$$A = 2\times\frac{1}{2}(2d)\times\frac{d}{2} = d^2$$

$$I_{xx} = 2\times\frac{2d}{12}\times\left(\frac{d}{2}\right)^3 = \frac{d^4}{24}$$

$$I_{yy} = 2\times\frac{1}{12}(d)\times d^3 = \frac{d^4}{6}$$

$$I_{\min} = I_{xx} = \frac{d^4}{24}$$

$$r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{d^4/24}{d^2}} = \frac{d}{\sqrt{24}}$$

Slenderness ratio, $$\lambda = \frac{l_{eff}}{r_{\min}} = \frac{3500}{d/\sqrt{24}} = \frac{17146.43}{d}$$

$$\therefore \qquad f_{cc} = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{17146.43}{d}\right)^2} = 6.714 \times 10^{-3} d^2$$

$$\sigma_{ac} = \frac{P}{A} = \frac{400 \times 10^3}{d^2}$$

Also,

$$\sigma_{ac} = \frac{0.6 f_{cc} f_y}{\left[(f_{cc})^n + (f_y)^n\right]^{1/n}}$$

$$\Rightarrow \qquad (\sigma_{ac})^n\left[(f_{cc})^n + (f_y)^n\right] = (0.6 f_y . f_{cc})^n$$

$$\Rightarrow \left(\frac{400 \times 10^3}{d^2}\right)^{1.4}\left[\left(6.714 \times 10^{-3} d^2\right)^{1.4} + (250)^{1.4}\right] = \left[0.6 \times 250 \times 6.714 \times 10^{-3} d^2\right]^{1.4}$$

After solving we get, $d = 102.7 \simeq 103$ mm

Q.6 (a) Solution:

Given: DL of the slab = 0.15 × 25 = 3.75 kN/m^2

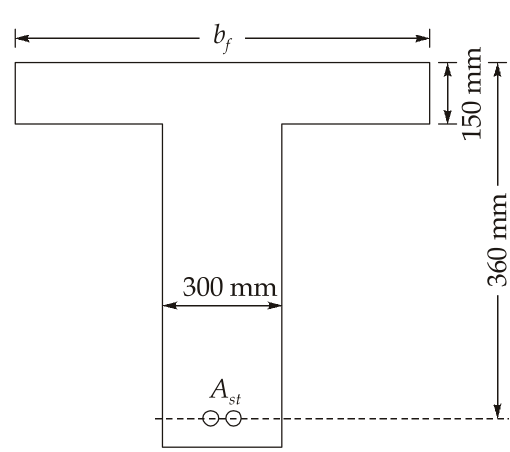
Superimposed load = 5 kN/m^2

∴ Total load on slab = 8.75 kN/m^2

Load per meter run of beam = Load on slab per unit area × c/c distance between beam

= 8.75 × 3.5 = 30.6 kN/m

Intermediate T-beam section is shown below:



Effective flange width, b_f = $\min.\begin{cases} \dfrac{l_0}{6}+b_w+6D_f = \dfrac{6000}{6}+300+6\times 150 = 2200 \text{ mm} \\ \dfrac{3500}{2}+300+\dfrac{3500}{2} = 3800 \text{ mm} \end{cases}$

= 2200 mm

Now, deal load of web of beam

DL = 0.30× 0.25 × 1 × 25 = 1.875 kN/m

∴ Total load on beam per m run = 30.6 + 1.875 = 32.475 kN/m

∴ Factored load on beam = 1.5 × 32.475 = 48.7125 kN/m ≃ 48.713 kN/m

Assuming NA lies in the flange

BM_u = MOR

= $0.36 f_{ck}\, x_u b_f\, (d - 0.2x_u)$

$$\Rightarrow \quad \frac{48.713\times(6)^2}{8}\times 10^6 = 0.36 \times 20 \times x_u \times 2200\,(360 - 0.42x_u)$$

$$\Rightarrow \quad 0.42x_u^2 - 360x_u + 13838.92 = 0$$

After solving, we get

x_u = 40.33 mm < 150 mm (OK)

Our assumption is correct and NA lies in flange.

$$\therefore \quad A_{st} = \frac{0.5 f_{ck} b_f d}{f_y}\left(1-\sqrt{1-\frac{4.6BM_u}{f_{ck}b_f d^2}}\right)$$

$$= \frac{0.5\times 20\times 2200\times 360}{250}\left(1-\sqrt{1-\frac{4.6\times\dfrac{48.713\times 6^2}{8}\times 10^6}{20\times 2200\times 360^2}}\right)$$

= 2937.153 mm^2

Provide 5-28 mm ϕ bars.

$$\therefore \quad (A_{st})_{\text{provided}} = 5\times\frac{\pi}{4}\times(28)^2 = 3078.76 \text{ mm}^2 > \left(A_{st,reqd.}\right) \quad \text{(OK)}$$

Minimum area of tension steel,

$$(A_{st})_{\min} = \frac{0.85}{f_y} b_w d$$

$$= \frac{0.85}{250} \times 300 \times 360 = 367.2 \text{ mm}^2$$

Maximum area of tension steel $= 0.04\, b_w D$

$= 0.04 \times 300 \times 400 = 4800 \text{ mm}^2$ (OK)

Design of shear reinforcement:

Factored shear force, $V_u = \dfrac{w_u L}{2} = \dfrac{32.475 \times 6}{2} = 97.425 \text{ kN}$

Nominal shear stress, $\tau_v = \dfrac{V_u}{b_w d} = \dfrac{97.425 \times 10^3}{300 \times 360}$

$= 0.902 \text{ N/mm}^2 < \tau_{c,\max} = 0.631\sqrt{f_{ck}}$

$= 0.631\sqrt{20} = 2.82 \text{ N/mm}^2$ (OK)

Now for design shear strength of concrete,

Percentage of tensile reinforcement,

$$P_t(\%) = \frac{A_{st}}{b_w d} \times 100 = \frac{5 \times \frac{\pi}{4} \times (28)^2}{300 \times 360} \times 100 = 2.85\%$$

From table given:

$$\tau_c = 0.82 \text{ N/mm}^2$$

$\because \tau_v > \tau_c$, hence shear reinforcement is provided for a shear force of

$$V_{us} = V_u - V_c$$

$$= 97.425 - (0.82 \times 300 \times 360) \times 10^{-3}$$

$$= 8.865 \text{ kN}$$

Using 2-legged 8 mm ϕ stirrups

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$$

$$S_v = \frac{0.87 \times 250 \times 2 \times \frac{\pi}{4}(8)^2 \times 360}{8.865 \times 10^3} = 887.94 \text{ mm}$$

As per minimum shear reinforcement

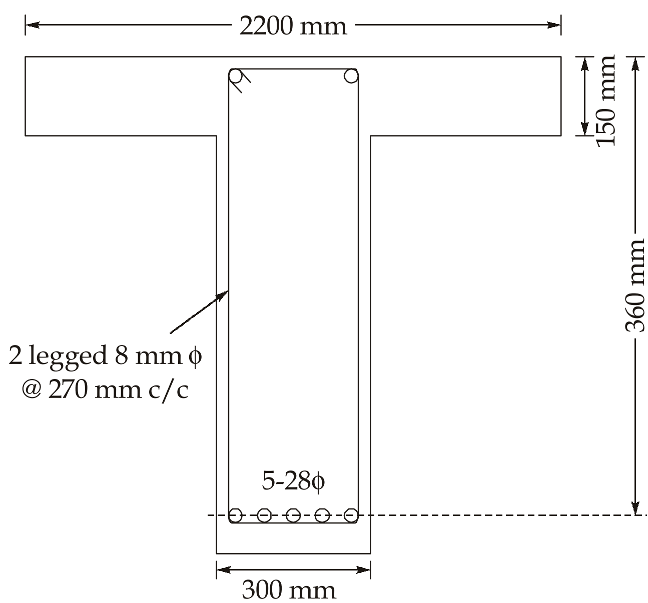
$$\frac{A_{sv}}{b_w S_v} > \frac{0.4}{0.87 f_y}$$

$$\Rightarrow \qquad S_v < \frac{2\times\frac{\pi}{4}(8)^2\times 0.87\times 415}{0.4\times 300} = 302.47 \text{ mm}$$

$$\text{Also} \qquad S_{v,\max} = \begin{cases} 0.75d = 0.75\times 360 = 270 \text{ mm} \\ = 300 \text{ mm} \end{cases}$$

Use minimum of all above i.e., S_v = 270 mm

$\therefore$ Provide 2-legged 8 mm ϕ stirrups @ 270 mm c/c



Q.6 (b) Solution:

Description	Purpose	Application	Capacity	Remarks
1. Backhoe	For excavation below the ground (lower elevation)	Cutting of trenches, pits etc., leveling and loading	Struck bucket capacity 0.38 m^3 – 3.25 m^3	Suitable for heavy positive cutting
2. Shovel or front shovel	For excavation above its own tank or wheel level	For cutting and for loading	Struck bucket capacity 0.38 m^3 – 3.25 m^3	Suitable for heavy positive cutting in all types of dry soil
3. Dragline	For bulk excavation in loose soils below its own track level	For canals and pits excavation and cutting of ditches	0.38 m^3 – 3.06 m^3	Suitable for loose soils, marshy land and area containing water
4. Clamshell or Grab	For deep confined cuttings in pits, trenches	Such as shafts, pits wells	0.38 m^3 – 3.06 m^3	Consists of a hydraulically controlled bucket suspended from a lifting arm

5. Dozers	For moving earth upto a distance of about 100 m, shallow excavation and acting as a towing tractor and pusher to scraper machines	Clearing & grubbing sites, excavation of surface earth, and maintaining road	Blade capacity $1.14\ m^3 - 6.11\ m^3$	Lifting arm can be track mounted or wheel mounted
6. Roller compactor	For compaction of earth or other materials	Used for large works of highways, canals and airports	i) 8-10*t* ii) 4-17*t* iii) 11-25*t* iv) 2.5-11.5*t* capacity may be increased by ballasting	Comes in different varieties: i) Smooth wheeled tractor ii) Vibrating roller iii) Pneumatic tyred iv) Sheep foot roller
7. Scraper	For site stripping and leveling, loading hauling and discharging over long distances	Towed scraper, two-three axle scrapers	Size varying from $8\ m^3$ and $50\ m^3$	Best suit for haul distance varying between 150 m and 900 m
8. Dumper	For horizontal transportation of materials and off sites	Suitable for hauling on softer subgrades, large capacity dumpers are used in mines and quaries	Load capacity of is about 80t; 20t is common for small dumper	Comes in different varieties with front tipping, side tipping or elevated tipping arrangements
9. Grader	For spreading fill and fine trimming the subgrade grader performs a follow up operation to scraping or bill dazing	Grading & finishing the upper surface of the earthen formations and embankments		Grader usually operates in the forward direction

Q.6 (c) Solution:

$$\text{Area of the connected leg} = (125 - 5)\,10 = 1200\ \text{mm}^2$$

$$\text{Area of the outstanding leg} = (95 - 5)\,10 = 900\ \text{mm}^2$$

Factored tension in the angle member = 450 kN

$$\text{Force in the outstanding leg} = \frac{900}{1200+900}\times 450 = 192.9\ \text{kN}$$

$$\text{Force in the connected leg} = 450 - 192.9 = 257.1\ \text{kN}$$

Force transmitted to the lug angle = 1.2 times the force in the outstanding leg

$$= 1.2 \times 192.9 = 231.48\ \text{kN}$$

The 75 mm leg of the lug angle will be placed as the outstanding leg to be connected to 95 mm outstanding leg of the main angle.

Given size of weld = 6 mm

Strength of the weld per mm length

$$= \frac{f_u}{\sqrt{3}\gamma_{mw}}(0.7\,s) = \frac{410}{\sqrt{3}\times 1.25}[0.7\times 6] = 795.4\ \text{N/mm}$$

∴ Length of weld required for connecting the 100 mm leg of the lug angle to the gusset plate

$$= \frac{231.48 \times 10^3}{795.4} = 291 \text{ mm} \simeq 295 \text{ mm (say)}$$

∵ Force in the connected leg of the main angle = 257.1 kN

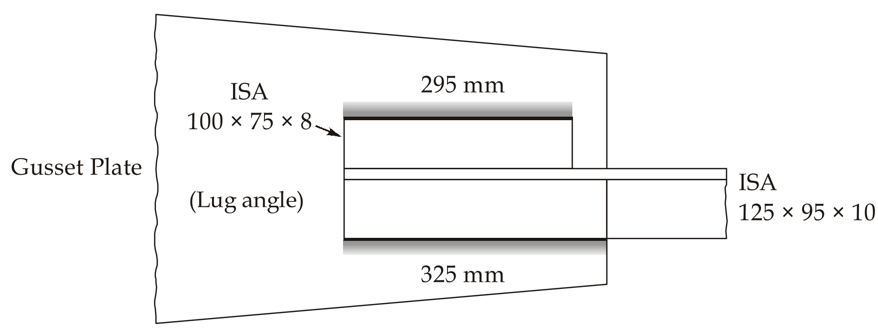
∴ Length of weld required to connect the main angle to the gusset plate

$$= \frac{257.1 \times 10^3}{795.4} = 323.2 \text{ mm} \simeq 325 \text{ mm (say)}$$

Length of weld required to connect the outstanding leg of the lug angle to the main angle

$$= \frac{\text{1.4 times the load share of outstanding leg of main angle}}{\text{Strength of weld per mm length}}$$

$$= \frac{1.4 \times 192.9 \times 10^3}{795.4} \simeq 340 \text{ mm}$$

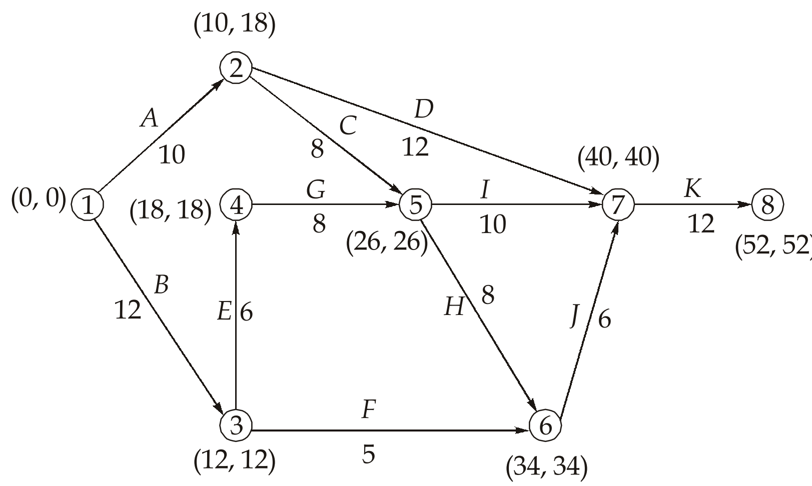


Q.7 (a) Solution:

Determination fo critical path:

Critical Path	Duration (in days)
1-2-7-8	34
1-2-5-7-8	40
1-2-5-6-7-8	44
1-3-4-5-7-8	48
1-3-4-5-6-7-8	52
1-3-6-7-8	35

Hence critical path is 1-3-4-5-6-7-8 i.e., (B-E-G-H-J-K) with project completion duration of 52 days.



Activity	Duration	EST	EFT	LST	LFT	Total float	Free float	Independent float
1-2	10	0	10	8	18	8	0	0
1-3	12	0	12	0	12	0	0	0
2-5	8	10	18	18	26	8	8	0
2-7	12	10	22	28	40	18	18	10
3-4	6	12	18	12	18	0	0	0
3-6	5	12	17	29	34	17	17	17
4-5	8	18	26	18	26	0	0	0
5-6	8	26	34	26	34	0	0	0
5-7	10	26	36	30	40	4	4	4
6-7	6	34	40	34	40	0	0	0
7-8	12	40	52	40	52	0	0	0

Q.7 (b) Solution:

Given: Axial load, $P_a = 2000$ kN

Unsupported length, $L = 3$ m

End conditions: Both the ends of the columns are braced against sidesway.

$$L_{eff} = L = 3 \text{ m}$$

Material used: M20, Fe415

Percentage of longitudinal steel = 2% of A_g

Factored axial load on column,

$$P_u = 1.5 \times 2000 = 3000 \text{ kN}$$

Since, section size is not given so, slenderness ratio and minimum eccentricity can't be determined.

So assuming column as short and axially loaded.

$$P_u = 0.4 f_{ck} A_g + \left(0.67 f_y - 0.4 f_{ck}\right) A_{sc}$$

$$\Rightarrow \quad 3000 \times 10^3 = 0.4 \times 20 \times A_g + (0.67 \times 415 - 0.4 \times 20)\ 0.02 A_g$$

$$\Rightarrow \quad A_g = 223863.89 \text{ mm}^2$$

Assuming one of column dimensions as 400 mm

$$\therefore \quad \text{Other dimension} = \frac{223863.89}{400} = 559.66 \text{ mm}$$

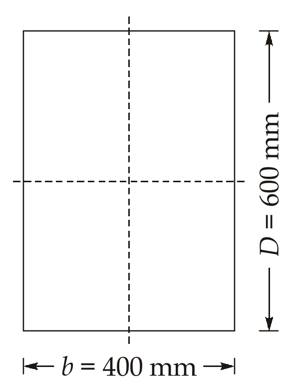
Let us adopt 400 mm × 600 mm as rectangular column

Step-1: Determine whether column is short or long.

$$\text{Slenderness ratio,} \quad \lambda_{max} = \frac{L_{eff}}{D} = \frac{3000}{400} = 7.5 < 12$$

$\therefore$ Column is short.

Step-2: Check for minimum eccentricity



$$(e_{min})_x = \max.\begin{cases} \dfrac{L_{\text{unsupported}}}{500} + \dfrac{D}{30} \\ 20 \text{ mm} \end{cases}$$

$$= \max.\begin{cases} \dfrac{3000}{500} + \dfrac{600}{30} = 26 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

$$= 26 \text{ mm} \leq 0.05 \times 600 = 30 \text{ mm} \qquad \text{(OK)}$$

$$(e_{min})_y = \max.\begin{cases} \frac{3000}{500} + \frac{400}{30} = 19.33 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

$$= 20 \text{ mm} \leq 0.05 \times 400 = 20 \text{ mm} \quad \text{(OK)}$$

Hence column is axially loaded.

Step-3: Longitudinal reinforcement.

$$P_u = 0.4 f_{ck} A_g + \left(0.67 f_y - 0.4 f_{ck}\right) A_{sc}$$

$$\Rightarrow \quad 3000 \times 10^3 = 0.4 \times 20 \times (400 \times 600) + (0.67 \times 415 - 0.4 \times 20) A_{sc}$$

$$\Rightarrow \quad A_{sc} = 3999.26 \text{ mm}^2$$

$$A_{sc,\min} = \frac{0.8}{100} \times 400 \times 600 = 1920 \text{ mm}^2 \quad \text{(OK)}$$

Provide 4-28 ϕ bars at corners and 4-25 ϕ bars on faces of column,

$$(A_{sc})_{\text{provided}} = 4 \times \frac{\pi}{4} \times (28)^2 + 4 \times \frac{\pi}{4} \times (25)^2$$

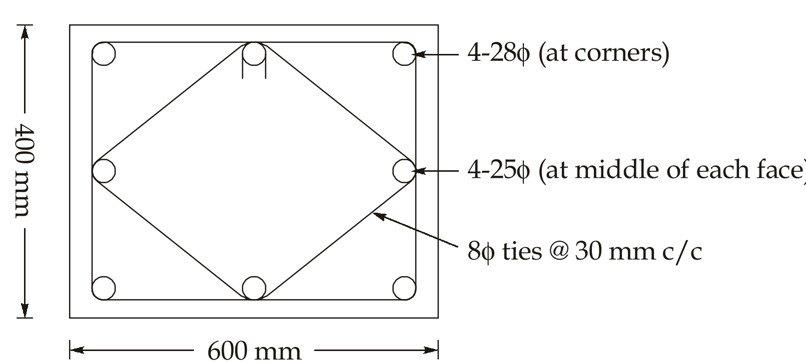
$$= 4426.50 \text{ mm}^2 > (A_{sc})_{\text{reqd.}}$$

Transverse reinforcement;

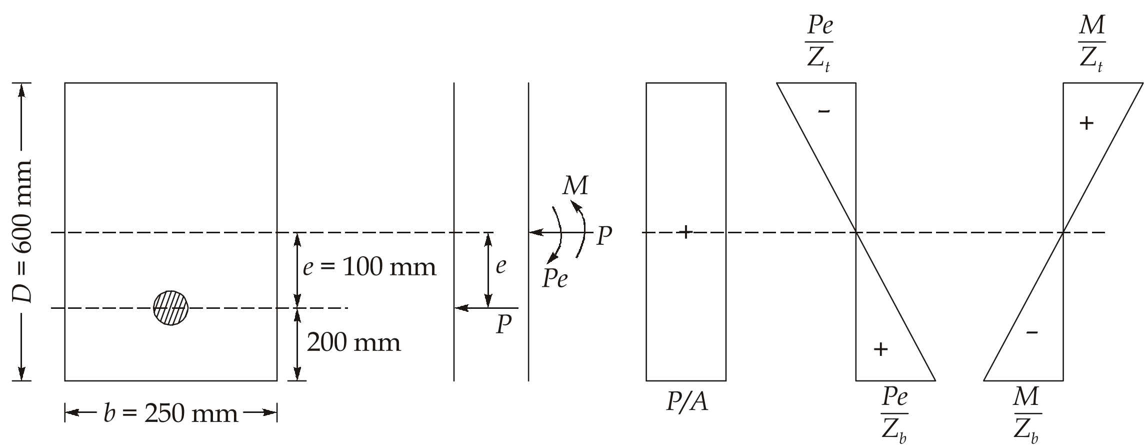
$$\text{Dia. of tie, } \phi_t = \max.\begin{cases} \frac{28}{4} = 7 \text{ mm} \\ 6 \text{ mm} \end{cases} = 7 \text{ mm} \simeq 8 \text{ mm}$$

$$\text{Spacing} = \min.\begin{cases} 400 \text{ mm} \\ 16 \times 25 = 400 \text{ mm} \\ 300 \text{ mm} \end{cases} = 300 \text{ mm}$$

$\therefore$ Provide 8 mm ϕ ties @ 300 mm c/c



Q.7 (c) Solution:



Eccentricity of prestressing force,

$$e = 100 \text{ mm}$$

At final stage,

∴ Stress at bottom, $$f_b = \frac{P}{A} + \frac{Pe}{Z_b} - \frac{M}{Z_b}$$

$$P = 830 \text{ kN} \qquad \text{(At final stage)}$$

(i) When stress at the bottom edge reaches zero i.e,

$$f_b = 0$$

$$\Rightarrow \quad \frac{P}{A} + \frac{Pe}{Z_b} = \frac{M}{Z_b}$$

$$\Rightarrow \quad \frac{830 \times 10^3}{250 \times 600} + \frac{830 \times 10^3 \times 100}{\frac{250 \times (600)^2}{6}} = \frac{M \times 10^6}{\frac{250 \times (600)^2}{6}}$$

$$\Rightarrow \quad M = 165.995 \simeq 166 \text{ kNm}$$

Let the maximum UDL the beam can carry be w kN/m

$$\therefore \quad \frac{wL^2}{8} = 166$$

$$\Rightarrow \quad \frac{w \times (8)^2}{8} = 166$$

$$\Rightarrow \quad w = 20.75 \text{ kN/m}$$

(ii) Stress at the bottom edge reaches a cracking tensile stress of 4 N/mm^2

$$\frac{P}{A} + \frac{Pe}{Z_b} - \frac{M}{Z_b} = -4$$

$$\Rightarrow \quad \frac{830\times10^3}{250\times600}+\frac{830\times10^3\times100}{\frac{250\times(600)^2}{6}}-\frac{M\times10^6}{\frac{250\times(600)^2}{6}} = -4$$

$$\therefore \quad M = 226 \text{ kNm}$$

Let the maximum uniformly distributed load for this condition will be w kN/m

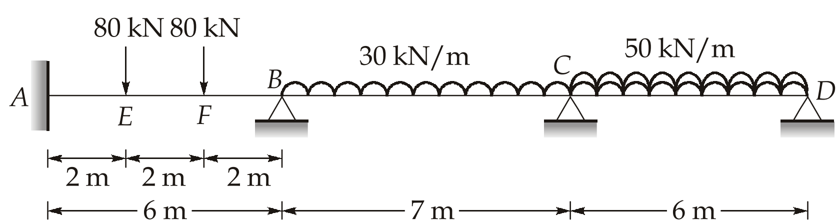
$$\therefore \quad \frac{wL^2}{8} = 226$$

$$\Rightarrow \quad w = \frac{226\times8}{(8)^2}$$

$$\Rightarrow \quad w = 28.25 \text{ kN/m}$$

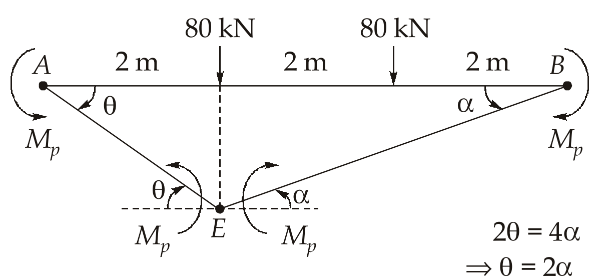
Q.8 (a) Solution:

Given: Continuous beam



Span AB:

Mechanism-1: Plastic hinges at A, B and E



By the principle of virtual work

$$W_i = W_E$$

$$\Rightarrow \quad 2M_p(\theta+\alpha) = 80\times2\theta+80\times2\alpha$$

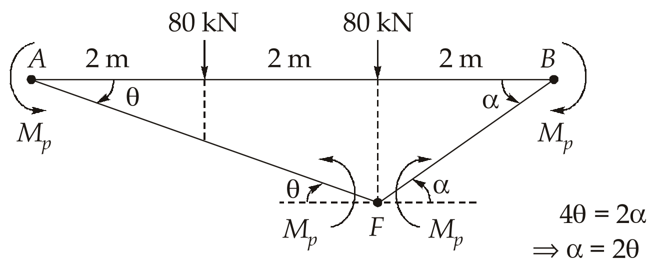
But $$\alpha = \frac{\theta}{2}$$

$$\therefore \quad 2M_p\left(\theta+\frac{\theta}{2}\right) = 160\theta+80\times2\times\frac{\theta}{2}$$

$$\Rightarrow \quad 3M_p = 240$$

$$\Rightarrow \quad M_p = 80 \text{ kNm}$$

Mechanism-2: Plastic hinges at A, B and F,



By the principle of virtual work

$$W_i = W_E$$

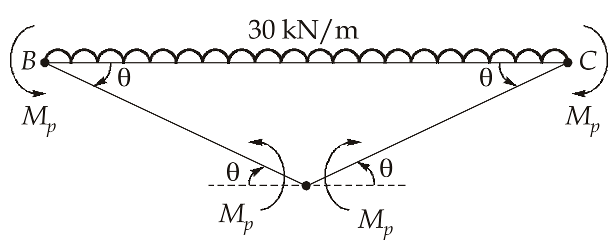
$$\Rightarrow \quad 2M_p(\theta + \alpha) = 80 \times 4\theta + 80 \times 2\theta$$

$$\Rightarrow \quad 6M_p = 480$$

$$\Rightarrow \quad M_p = 80 \text{ kNm} \qquad \text{...(i)}$$

Span BC:

Plastic hinges at B, C and at mid point of BC.



By the principle of virtual work

$$W_i = W_E$$

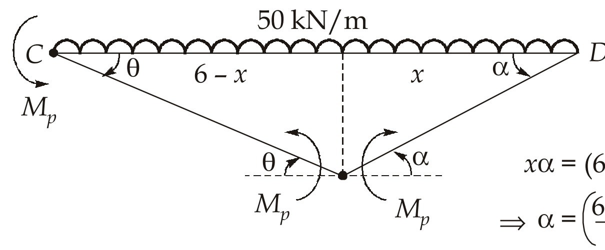
$$\Rightarrow \quad 4M_p\theta = 30 \times \left\{\frac{1}{2} \times 3.5\theta \times 7\right\}$$

$$\Rightarrow \quad M_p = 91.875 \text{ kNm}$$

Span CD:

Plastic hinges at C and in between C and D.

Let second plastic hinge forms at a distance x from D.



$$x\alpha = (6 - x)\theta$$

$$\Rightarrow \alpha = \left(\frac{6 - x}{x}\right)\theta$$

By the principle of virtual work

$$W_i = W_E$$

$$\Rightarrow \quad 2M_p\theta + M_p\alpha = 50 \times \left\{\frac{1}{2} \times (6 - x)\theta \times 6\right\}$$

$$\Rightarrow \quad 2M_p\theta + M_p\left(\frac{6 - x}{x}\right)\theta = 50 \times 3(6 - x)\theta$$

$$\Rightarrow \quad M_p\left(\frac{2x + 6 - x}{x}\right) = 150(6 - x)$$

$$\Rightarrow \quad M_p = \frac{150x(6 - x)}{6 + x}$$

For M_p to be maximum

$$\frac{dM_p}{dx} = 0$$

$$\Rightarrow \quad (6 + x)(6 - 2x) - x(6 - x) = 0$$

$$\Rightarrow \quad 36 - 12x + 6x - 2x^2 - 6x + x^2 = 0$$

$$\Rightarrow \quad -x^2 - 12x + 36 = 0$$

$$\Rightarrow \quad x^2 + 12x - 36 = 0$$

$$\therefore \quad x = \frac{-12 \pm \sqrt{(12)^2 - 4 \times 1 \times (-36)}}{2 \times 1}$$

$$= \frac{-12 \pm \sqrt{288}}{2}$$

$$= \frac{-12 \pm 12\sqrt{2}}{2} = -6 + 6\sqrt{2} \qquad \text{(Ignore –ve sign)}$$

$$\therefore \quad x = 6\left(\sqrt{2} - 1\right) = 0.414 \times 6 = 2.484 \text{ m}$$

$$\therefore \qquad M_p = \frac{150 \times 2.484(6-2.484)}{6+2.484} = 154.42 \text{ kNm}$$

$\therefore$ Plastic moment of the continuous beam is

$$M_p = \text{Maximum of (i), (ii) and (iii)}$$
$$= 154.42 \text{ kNm}$$

Q.8 (b) Solution:

Clear height of wall above opening

$$= 6 - 2.75 = 3.25 \text{ m}$$

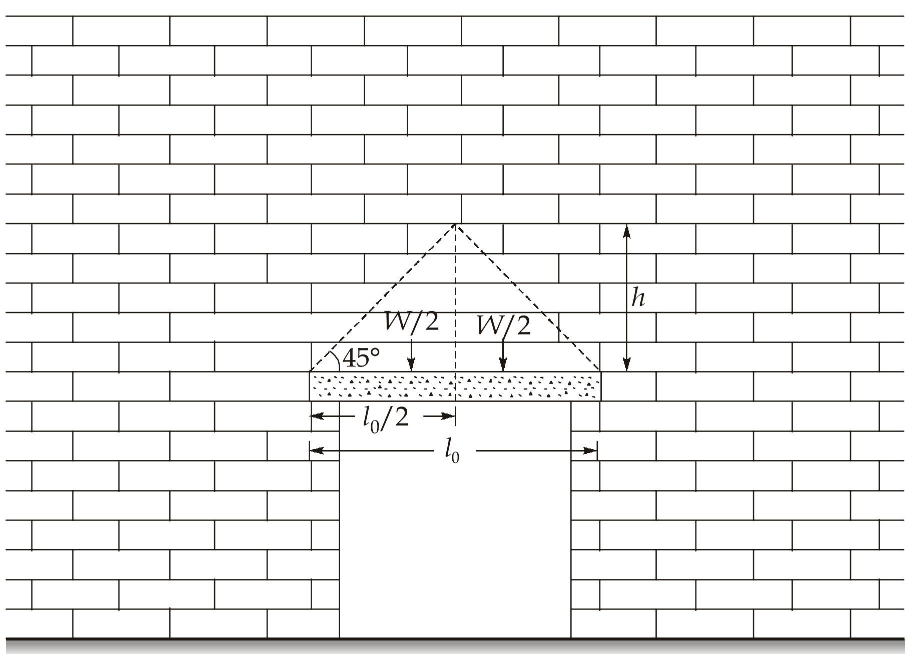
Given, overall depth of lintel = 150 mm

Effective cover = 20 mm

$\therefore$ Effective depth of lintel = 150 – 20 = 130 mm

Effective span of lintel, l_0 = 2.5+ 0.15 = 2.63 m

As load over opening is transferred through arch action, therefore, load acting on a lintel is triangular with base angle = 45° (given)



Let, W = Weight of masonry within triangle

$$\therefore \text{ Total load on lintel, } \quad W = \frac{1}{2} \times 2.63 \times \left(\frac{2.63}{2}\right) \times 0.40 \times 19 = 13.14 \text{ kN}$$

Maximum bending moment at mid span due to the triangular load, is

$$BM = R_A \times \frac{l}{2} - \frac{W}{2} \times \left(\frac{l_0}{2} \times \frac{1}{3}\right)$$

Also, $$R_A = \frac{W}{2}$$

$\therefore$ $$BM = \frac{Wl}{4} - \frac{Wl}{12} = \frac{Wl}{6} = \frac{13.14 \times 2.63}{6} = 5.76 \text{ kNm}$$

Maximum shear force at support,

$$R_A = \frac{W}{2} = \frac{13.14}{2} = 6.57 \text{ kN}$$

$$\text{DL of lintel} = 0.15 \times 0.40 \times 25 = 1.5 \text{ kN/m}$$

Maximum bending moment at mid span due to uniformly distributed dead load

$$BM = \frac{Wl^2}{8} = \frac{1.5 \times (2.63)^2}{8} = 1.3 \text{ kNm}$$

Maximum shear force at support due to dead load

$$= 1.5 \times \frac{2.5}{2} = 1.875 \text{ kN}$$

$$\text{Total factored bending moment} = 15\ (5.76 + 1.30) = 10.60 \text{ kNm}$$

$$\text{Total factored shear force} = 1.5\ (6.57 + 1.875) = 12.67 \text{ kN}$$

Check for beam depth:

$$BM_u = M_{u,\, lim}$$

$\Rightarrow$ $$10.60 \times 10^6 = 0.138 f_{ck}\, bd^2$$

$\Rightarrow$ $$10.60 \times 10^6 = 0.138 \times 20 \times 400 \times d^2$$

$\Rightarrow$ $$d = 97.99 \text{ mm} < 130 \text{ mm}$$

As provided depth is more than that required for balanced section, section is under-reinforced.

$\therefore$ $$A_{st} = \frac{0.5 f_{ck} bd}{f_y}\left(1 - \sqrt{1 - \frac{4.6 BM_u}{f_{ck} bd^2}}\right)$$

$$= \frac{0.5 \times 20 \times 400 \times 130}{415}\left(1 - \sqrt{1 - \frac{4.6 \times 10.6 \times 10^6}{20 \times 400 \times 130^2}}\right)$$

$$= 251.11 \text{ mm}^2$$

$$A_{st,\min} = \frac{0.12}{100} \times 400 \times 150 = 72 \text{ mm}^2$$

$$\therefore \text{No. of 10 mm f bars required} = \frac{251.11}{\frac{\pi}{4}(10)^2} = 3.2 \simeq 4 \text{ bars}$$

$\therefore$ Provide 4-10 mm ϕ bars,

Cut-off 2- 10 mm ϕ bars at 0.08 l = 0.08 × 2500 = 200 mm from the face of support, provide 2 additional 10 mm ϕ bars or continue the main bars near the top at the support in a length equal to 0.1 $l \simeq$ 250 mm for resisting BM due to partial fixity.

Check for shear:

Factored shear stresses, $$\tau_v = \frac{V_u}{bd} = \frac{12.67 \times 1000}{400 \times 130} = 0.24 \text{ N/mm}^2$$

Percentage of tension steel provided,

$$P_t\% = \frac{A_{st}}{bd} \times 100 = \frac{4 \times \frac{\pi}{4} \times (10)^2}{400 \times 130} \times 100 = 0.604\%$$

From table given, $$\tau_c = 0.48 + \frac{0.56 - 0.48}{0.75 - 0.50} \times (0.604 - 0.50) = 0.513 \text{ N/mm}^2$$

$\because$ $\tau_v < \tau_c$ $\therefore$ No shear reinforcement is necessary

Check for development length at support:

MOR offered by 2-10 mm ϕ bars at support,

$$= 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times (10)^2 \left[130 - \frac{0.42 \times 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 10^2}{0.36 \times 20 \times 400}\right]$$

$$= 6.904 \times 10^6 \text{ Nmm}$$

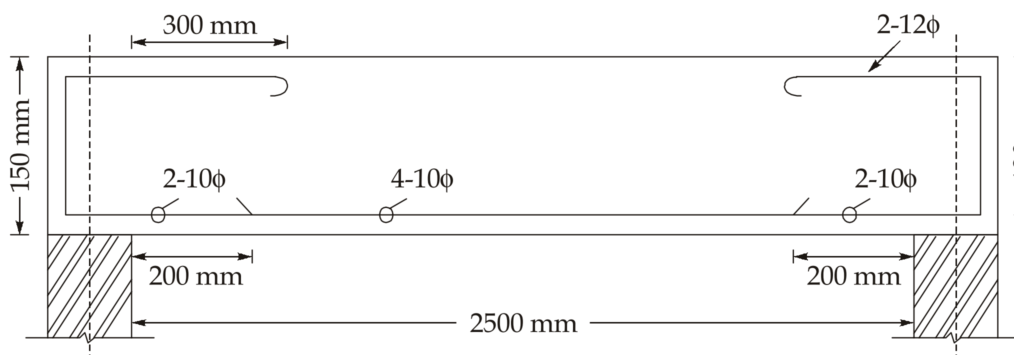
$$V_u = 12.67 \text{ kN}$$

$$L_d \leq \frac{1.3 M_u}{V_u} + L_0$$

But $$L_d = \frac{\phi \sigma_{st}}{4 \tau_{bd}} = \frac{10 \times 415 \times 0.87}{4 \times (1.2 \times 1.6)} = 470.12 \text{ mm}$$

Also, $$\frac{1.3 M_u}{V_u} + L_0 = 1.3 \times \frac{6.904 \times 10^6}{12.67 \times 10^3} + 125 = 833.38 \text{ mm}$$

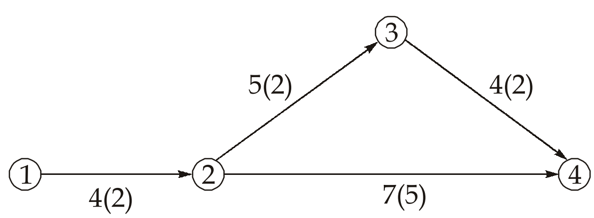
$$\therefore \qquad L_d \leq \frac{1.3M_u}{V_u} + L_0 \qquad \text{(OK)}$$



Reinforcement Details

Q.8 (c) Solution:

(i)



Critical path is 1-2-3-4 with project completion duration of (4 + 5 + 4) = 13 weeks

Total normal cost of project = Rs. 15600

Activity	Cost - slope (Rs./week)
1-2	4000
2-3	1500
2-4	1200
3-4	2500

To get optimum duration, apply project crashing.

First stage crashing:

Considering minimum cost slope activity in the critical path i.e., activity 2-3 having cost slope of Rs. 1500 per week.

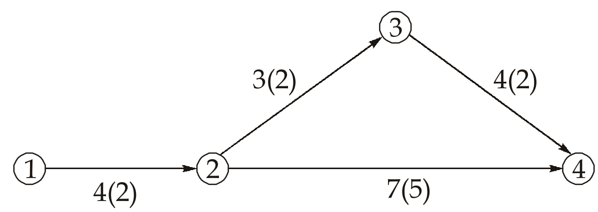
Crash activity 2-3 by two weeks

$\therefore$ Project duration = 13 – 12 = 11 weeks

Crashing cost = 1500 × 2 = Rs. 3000

$\therefore$ Total cost of project = 15600 + 2000 × 11 + 3000 = Rs. 40600

Network diagram:



Now, we have two critical paths viz. 1-2-3-4 and 1-2-4

Second stage crashing:

Considering minimum cost slope along critical paths, possible crashing of activities can be

2-3 and 2-4, crashing cost = Rs. 2700/week

3-4 and 2-4; crashing cost = Rs. 3700/week

Crashing activity 2-4 by 1 week.

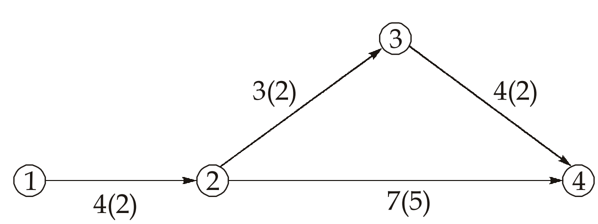
$$\text{Project duration} = 10 \text{ weeks}$$

$$\text{Total cost of project} = 15600 + 2000 \times 10 + (3000) + 2700$$

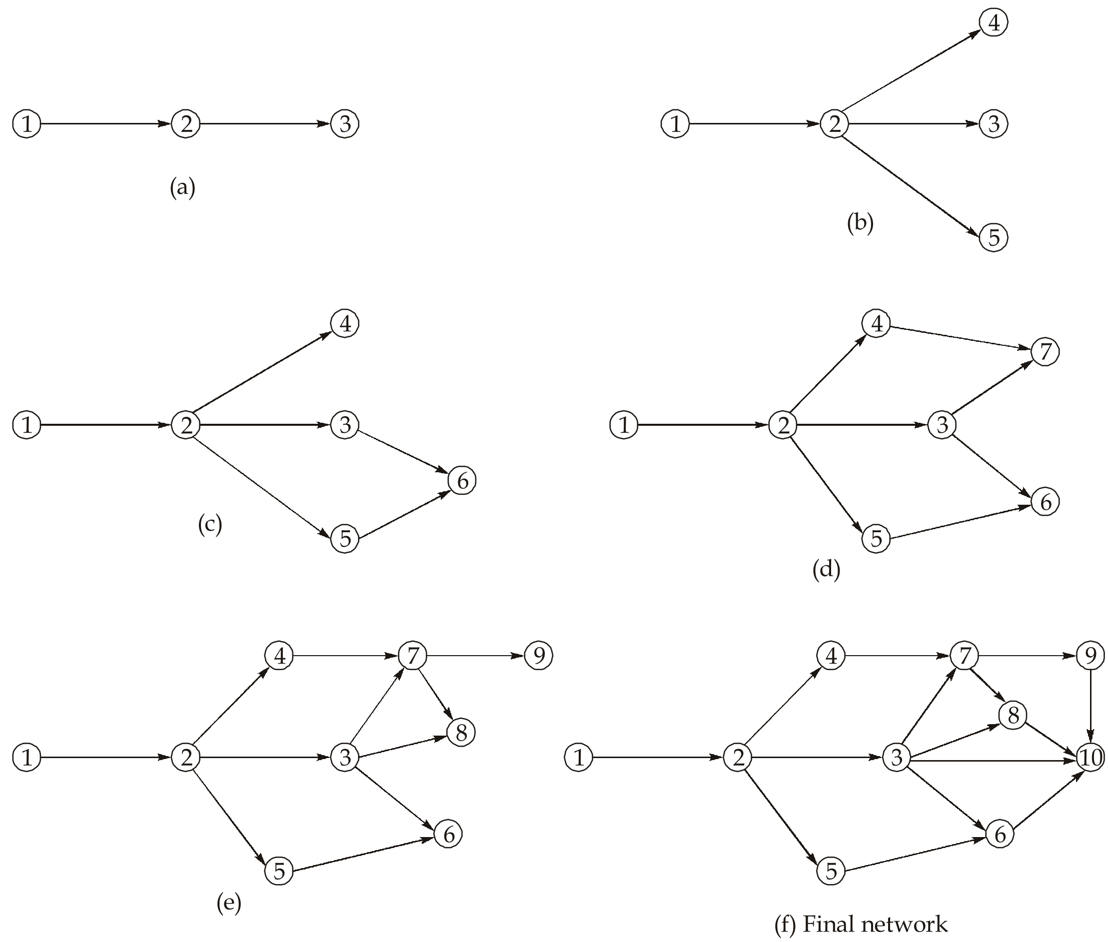
$$= \text{Rs. } 41300$$

Here we observe that for a project duration of 10 weeks, cost of project is Rs. 41300 whereas for a project duration of 11 weeks, cost of project is Rs. 40600. Therefore from the total cost curve, we can conclude that optimum project duration is 11 weeks is as shown below.

Least cost network



(ii) The step by step development of the network is shown in figure.



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