MADE EASY

India's Best Institute for IES, GATE & PSUs

Detailed Solutions

ESE-2018 Mains Test Series

Civil Engineering Test No : 10

Section A

Q.1 (a) Solution:

Tempered glass is a kind of safety glass, made from the normal glass by strengthening it by a simple physical process. In the tempering process, the flag glass is reheated to its softening temperature then taken out of the furnace and rapidly cooled with jets of cold air directed to the surface of glass. This process produces a state of compression in the glass surfaces while the core of the glass is in the state of compensating tension.

Properties :

- It is 4-5 times stronger than ordinary annealed glass.
- In the event of breakage this, glass does not form the sharp shards or edges which may cause cutting and piercing injuries to humans, as it will disintegrate into small granules or blunt particles that will cause practically no injury to humans.
- It can withstand a very wide range of rapid temperature variations. The heat shock endurance of tempered glass is two times more as compared with ordinary glass and can generally bear difference in temperature of more than 150°C.

Applications :

- In patio and entrance doors
- tub and shower enclosures
- It is commonly used for windows of commercial/residential buildings.

- For displays and partitions
- Storefronts and handrails

Advantages :

The tempered glass has enhanced impact resistance, mechanical strength and thermal stability or resistance to thermal cracking. Although tempered glass is four to five times as strong as annealed glass of the same thickness, but it can disintegrate under impact into innumerable cellular pieces or small granules, i.e., fragments of more or less cubical shape when the external impact force exceeds its strength. It is thus a kind of safety glass. There are two distinct heat-treated glass products viz. heat-strengthened glass and fully tempered glass. However, all cutting and fabricating in glass must be done before tempering.

Disadvantages :

A disadvantages with tempered flat and bend glass is that, unlike annealed, wired or laminated glass, it cannot be cut from a larger sheet of glass. Tempered glass must be pre-cut to size, and then tempered. It might take time to get a custom order for tempered glass filled. This would suggest a need for standardizing glass sizes, and stocking replacements.

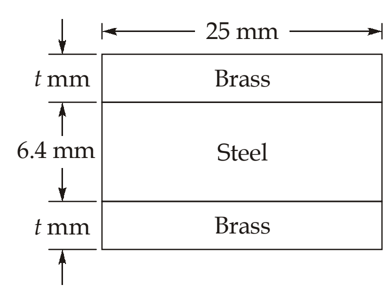
Q.1 (b) Solution:

(i)

Let the axial pull on the compound bar be P Newton.

Let the stresses in brass and steel be p_b and p_s N/mm^2 respectively.

Strain in steel = Strain in brass



$$\Rightarrow \quad \frac{p_s}{E_s} = \frac{p_b}{E_b}$$

$$\Rightarrow \quad p_s = \frac{E_s}{E_b} \cdot p_b = \frac{2.07}{1.14} p_b$$

$$\Rightarrow \quad p_s = 1.816\, p_b$$

But load on steel + load on brass = Total load

$$\Rightarrow \quad p_s A_s + p_b A_b = P$$

$$1.816\, p_b (25 \times 6.4) + p_b \times 2t \times 25 = P \text{ Newton}$$

$$\Rightarrow \quad P = p_b (290.56 + 50t) \text{ Newton}$$

Area of the composite section,

$$A = 25 \times 6.4 + 25 \times 2t \text{ mm}^2 = (160 + 50t) \text{ mm}^2$$

Apparent Young's modulus, $E = 1.57 \times 10^5$ N/mm²

$$\therefore \quad \text{Strain, } \varepsilon = \frac{P}{AE} = \frac{p_b(290.56 + 50t)}{(160+50t)1.57\times 10^5}$$

This must be equal to the strain of brass or steel i.e.,

$$\frac{p_b}{E_b} = \frac{p_b}{1.14\times 10^5}$$

$$\Rightarrow \quad \frac{p_b(290.56+50t)}{(160+50t)1.57\times 10^5} = \frac{p_b}{1.14\times 10^5}$$

$$\Rightarrow \quad \frac{290.56+50t}{160+50t} = \frac{1.57}{1.14} = 1.377$$

$$\Rightarrow \quad 290.56 + 50t = 220.32 + 68.85t$$

$$\Rightarrow \quad 18.85t = 70.24$$

$$\Rightarrow \quad t = 3.726 \text{ mm}$$

(ii)

Since $p_s = 1.816\, p_b$ and since the stress in either brass or steel should not exceed 157 N/mm².

Let, $p_s = 157$ N/mm² $\quad (\because p_b$ will always be less than $p_s)$

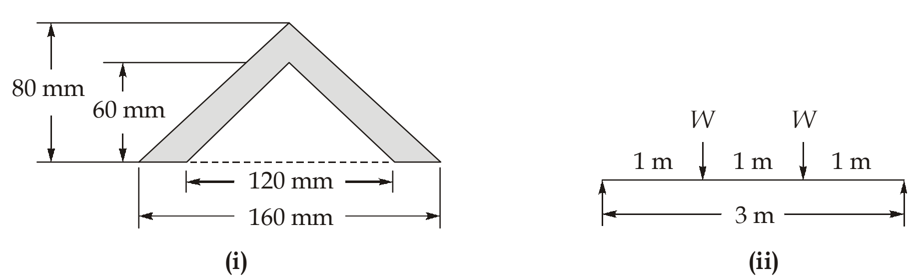
$$\therefore \quad p_b = \frac{157}{1.815} = 86.5 \text{ N/mm}^2$$

$\therefore$ Axial pull on the bar, $P = p_s A_s + p_b A_b$

$$= 157 \times 25 \times 6.4 + 86.5 \times 25 \times 2 \times 3.726$$

$$= 25120 + 16114.95 = 41234.95 \text{ N} = 41.235 \text{ kN}$$

Q.1 (c) Solution:



Area of section, $A = 160\times\frac{80}{2} - 120\times\frac{60}{2} = 2800$ mm²

Height of neutral axis above the base

$$\bar{y} = \frac{\frac{6400\times80}{3} - \frac{3600\times60}{3}}{2800} = 35.2 \text{ mm}$$

Moment of inertia about the base

$$I_b = \frac{BH^3}{12} - \frac{bh^3}{12} = \frac{1}{12}[160\times80^3 - 120\times60^3]$$

$$= 466.67 \times 10^4 \text{ mm}^4$$

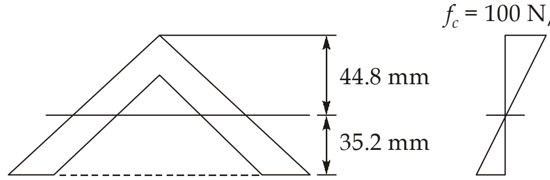
But $\quad I_b = I_{xx} + A\bar{y}^2$

$\Rightarrow \quad 466.67 \times 10^4 = I_{xx} + 2800 \times 35.2^2$

$\Rightarrow \quad I_{xx} = 119.74 \times 10^4 \text{ mm}^4$

Suppose the maximum tensile stress is allowed to reach 120 N/mm^2 and thus the corresponding maximum stress in compression

$$= \frac{120}{35.2} \times 44.8 = 152.7 \text{ N/mm}^2 > 100 \text{ N/mm}^2$$



But the maximum permissible compressive stress is only 100 N/mm^2. Hence the maximum tensile stress should not be allowed to reach 120 N/mm^2.

Let the maximum compressive stress be allowed to reach 100 N/mm^2.

$\therefore$ Corresponding maximum tensile stress

$$= \frac{35.2}{44.8} \times 100 = 78.57 \text{ N/mm}^2 < 120 \text{ N/mm}^2 \qquad \text{(OK)}$$

Now consider the beam carrying the two point loads,

$\therefore$ Maximum B.M., $\quad M = W \times 1 \text{ Nm} = W \times 1 \times 1000 \text{ Nmm}.$

But $\quad \frac{M}{I} = \frac{f}{y}$

$\Rightarrow \quad \frac{W\times1\times1000}{119.74\times10^4} = \frac{100}{44.8}$

$\Rightarrow \quad W = 2672.8 \simeq 2673 \text{ N}$

Q.1 (d) Solution:

(i) Lateral strain without lateral pressure,

$$\in_{\text{lateral}} = \frac{\mu\sigma_1}{E}$$

Lateral strain with lateral pressure,

$$\in_2 = \frac{1}{E}[-\sigma_2 + \mu(\sigma_1 + \sigma_3)] = \frac{1}{2}\frac{\mu\sigma_1}{E}$$

$$\Rightarrow \quad -\sigma_2 + \frac{\mu}{2}\sigma_1 + \mu\sigma_3 = 0 \quad \text{...(1)}$$

Also

$$\in_3 = \frac{1}{E}[-\sigma_3 + \mu(\sigma_2 + \sigma_1)] = \frac{1}{3}\times\frac{\mu\sigma_1}{E}$$

$$\Rightarrow \quad -\sigma_3 + \frac{2}{3}\mu\sigma_1 + \mu\sigma_2 = 0 \quad \text{...(2)}$$

On solving (1) and (2) for σ_2 an σ_3 we get,

$$\sigma_2 = \frac{\mu(3+4\mu)}{6(1-\mu^2)}\sigma_1 = 0.178\sigma_1$$

$$\sigma_3 = \frac{\mu(4+3\mu)}{6(1-\mu^2)}\sigma_1 = 0.211\sigma_1$$

(ii)

$$\in_1 = \frac{1}{E}[-\sigma_1 + \mu(\sigma_2 + \sigma_3)]$$

$$= \frac{1}{E}[-\sigma_1 + 0.25(0.178\sigma_1 + 0.211\sigma_1)]$$

$$= -\frac{0.903}{E}\sigma_1 = -4.515\times10^{-6}\sigma_1$$

$$\in_2 = \frac{1}{E}[-\sigma_2 + \mu(\sigma_1 + \sigma_3)]$$

$$= \frac{1}{E}[-0.178\sigma_1 + \mu(\sigma_1 + 0.211\sigma_1)]$$

$$= \frac{0.125}{E}\sigma_1 = 6.25\times10^{-7}\sigma_1$$

$$\in_3 = \frac{1}{E}[-\sigma_3 + \mu(\sigma_1 + \sigma_2)]$$

$$= \frac{1}{E}[-0.211\sigma_1 + \mu(\sigma_1 + 0.178\sigma_1)]$$

$$= \frac{0.083}{E}\sigma_1 = 4.175\times10^{-7}\sigma_1$$

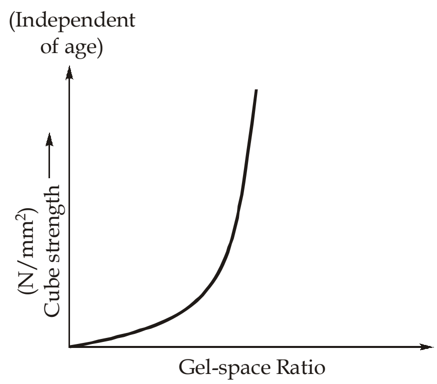
Q.1 (e) Solution:

(i) Gel-space Ratio : Gel-space ratio is defined as the ratio of volume of hydrated cement paste to the sum of the volumes of the hydrated cement and that of the capillary pores. A typical curve relating gel-space ratio and compressive strength of concrete is shown which is given by.

$$S = 240x^3$$

where S = strength of concrete N/mm^2

x = Gel-space ratio



(ii) Concreting in hot weather poses some special problems such as,

- Strength reduction
- Cracking of flat surfaces due to too rapid drying.

Concrete that stiffens before consolidation is caused by too rapid setting of cement and too much absorption and evaporation of mixing water. This leads to difficulty in finishing flat surfaces. Therefore, limitations are imposed on placing concrete during hot weather and on the maximum temperature of the concrete; quality and durability suffer when concrete is mixed, placed and cured at high temperature.

Q.2 (a) Solution:

A. Distribution Factors

Joint	Member	Relative Stiffness	Total Relative Stiffness	Distribution Factors
B	BA	$\frac{I}{3} = \frac{2I}{6}$	$\frac{3I}{6}$	$\frac{2}{3}$
	BC	$\frac{I}{6}$		$\frac{1}{3}$
C	CB	$\frac{I}{6}$	$\frac{2I}{6}$	$\frac{1}{2}$
	CD	$\frac{1}{6}$		$\frac{1}{2}$

(i) Non sway analysis

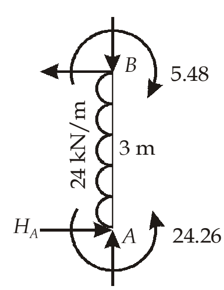
Fixed end moments

$$\bar{M}_{AB} = \frac{-24 \times 3^2}{12} = -18 \text{ kN-m}$$

$$\bar{M}_{BA} = +\frac{24 \times 3^2}{12} = +18 \text{ kN-m}$$

$$\bar{M}_{BC} = \bar{M}_{CB} = \bar{M}_{CD} = \bar{M}_{DC} = 0$$

A	B ($\frac{2}{3}$)	B ($\frac{1}{3}$)	C ($\frac{1}{2}$)	C ($\frac{1}{2}$)	D
–18	+18	0	0	0	0
	–12	–6	0	0	
–6		0	–3		0
	0	0	+1.5	+1.5	
0		+0.75	0		+0.75
	–0.50	–0.25	0	0	
–0.25		0	–0.125		0
	0	0	+0.06	+0.06	
0		+0.03	0		+0.03
	–0.02	–0.01	0	0	
–0.01			–0.005		0
–24.26	**+5.48**	**–5.48**	**–1.57**	**+1.57**	**+0.78**

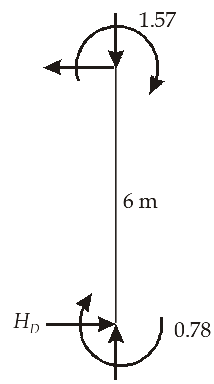


Taking moment about B,

$3H_A + 24.26 - 5.48 + 24 \times 3 \times 1.5 = 0$

$$\text{Horizontal reaction at } A = \frac{-24.26 + 5.48 - 24 \times 3 \times 1.5}{3}$$

$$= -42.26 \text{ kN } (\rightarrow) = 42.26 \text{ kN } (\leftarrow)$$



Taking moment about C,

$$6H_D - 1.57 - 0.78 = 0$$

$$\text{Horizontal reaction at } D = \frac{1.57 + 0.78}{6} = 0.39 \text{ kN } (\rightarrow)$$

Force acting from left to right = Total UDL + Horizontal reaction at D

$$= (24 \times 3) + 0.39 = 72.\,39 \text{ kN}$$

Force acting from right to left = 42.26 kN

$\therefore$ Sway force = 72.39 – 42.26 = 30.13 kN ($\rightarrow$)

(ii) Sway analysis

Since sway of the frame is from left to right, the initial fixing moments due to deflection D will be in anticlockwise direction

$$\bar{M}_{AB} = \bar{M}_{BA} = \frac{-6EI\Delta}{L^2} = \frac{-6EI\Delta}{3^2} = \frac{-6EI\Delta}{9}$$

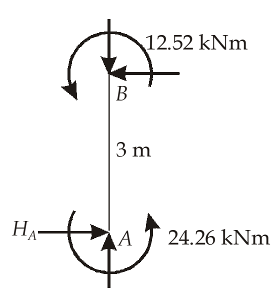
$$\bar{M}_{BC} = \bar{M}_{CB} = 0$$

$$\bar{M}_{CD} = \bar{M}_{DC} = \frac{-6EI\Delta}{L^2} = \frac{-6EI\Delta}{6^2} = \frac{-6\,EI\Delta}{36}$$

$\therefore$	$\bar{M}_{AB}$	:	$\bar{M}_{BA}$	:	$\bar{M}_{BC}$	:	$\bar{M}_{CB}$	:	$\bar{M}_{CD}$	:	$\bar{M}_{DC}$
	$\frac{-1}{9}$	:	$\frac{-1}{9}$	:	0	:	0	:	$\frac{-1}{36}$	:	$\frac{-1}{36}$
	– 4	:	– 4	:	0	:	0	:	– 1	:	– 1
	– 36	:	– 36	:	0	:	0	:	– 9	:	– 9

	A	B		C		D
		$\frac{2}{3}$	$\frac{1}{3}$	$\frac{1}{2}$	$\frac{1}{2}$	
	–36	–36	0	0	–9	–9
		+24	+12	+4.5	+ 4.5	
	+12		+2.25	+6		+2.25
		–1.50	–0.75	–3	–3	
	–0.75		–1.50	–0.38		–1.50
		+1.00	+0.5	+0.19	+0.19	
	+0.5		+0.09	+0.25		+0.10
		–0.06	–0.03	–0.121	–0.13	
	–0.03		–0.06	–0.02		–0.07
		+0.04	+0.02	+0.01	+0.01	
	+0.02		0	+0.01		0
		0	0	0	0	
Col. (a)	**–24.26**	**–12.52**	**+12.52**	**+7.43**	**–7.43**	**–8.22**

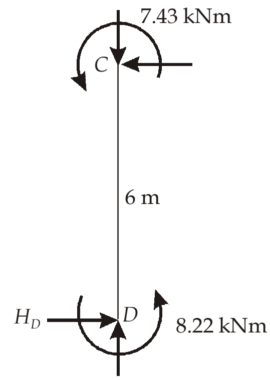
Let the moments shown in col. (a) be due to a sway force S.



Taking moment about B,

$$3H_A + 24.26 + 12.52 = 0$$

$$\text{Horizontal reaction at } A = \frac{-24.26 - 12.52}{3} = -12.26 \text{ kN } (\rightarrow) = 12.26 \text{ kN } (\leftarrow)$$



Taking moment about C,

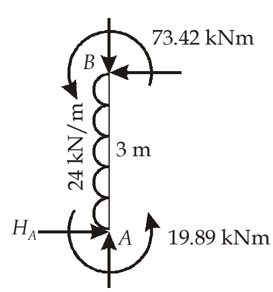
$$6H_D + 8.22 + 7.43 = 0$$

$$\text{Horizontal reaction at } D = \frac{-7.43 - 8.22}{6} = -2.61 \text{ kN } (\rightarrow) = 2.61 \text{ kN } (\leftarrow)$$

$\therefore$ Sway force, $S = 12.26 + 2.61 = 14.87 \text{ kN } (\rightarrow)$

Thus, the moments shown in col. (a) are due to a sway force of 14.87 kN. Hence, for the actual sway force of 30.13 kN the actual sway moments will be $\frac{30.13}{14.87} \times$ col. (a) moments.

	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
col. (a)	– 24.26	– 12.52	+ 12.52	+ 7.43	– 7.43	– 8.22
Actual Sway moments	– 49.16	– 25.37	+ 25.37	+ 15.06	– 15.06	– 16.66
Non-Sway moments	– 24.26	+ 5.48	– 5.48	– 1.57	+ 1.57	+ 0.78
Final moments	– 73.42	– 19.89	+ 19.89	+ 13.50	– 13.50	– 15.88



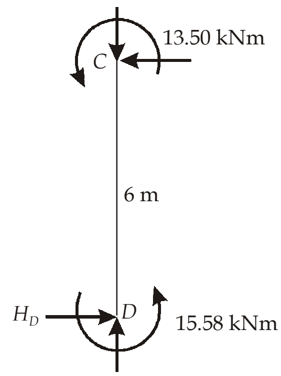
Taking moment about B,

$$3H_A + 19.89 + 73.42 + 24 \times 3 \times 1.5 = 0$$

Actual horizontal reaction at A

$$= \frac{-73.42 - 19.89 - 24 \times 3 \times 1.5}{3}$$

$$= -67.10 \text{ kN } (\rightarrow) = 67.10 \text{ kN } (\leftarrow)$$



Taking moment about C,

$$6H_D + 15.58 + 13.50 = 0$$

Actual horizontal reaction at D

$$= \frac{-13.50 - 15.88}{6} = -4.90 \text{ kN } (\rightarrow) = 4.91 \text{ kN } (\leftarrow)$$

$$\text{Vertical reaction at } A = -\left(\frac{19.89 + 13.50}{6}\right) = -5.57 \text{ kN}(\uparrow) = 5.57 \text{ kN } (\downarrow)$$

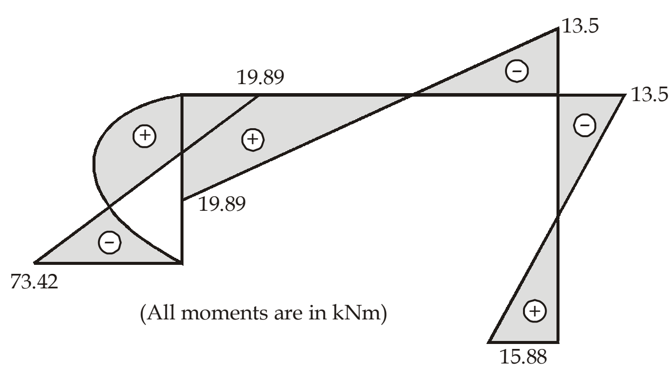
$\therefore$ Vertical reaction at D = 5.57 kN ($\uparrow$)

Bending moment diagram

Taking outer face of the portal frame as reference face.

Simply supported moment for

$$AB = \frac{24 \times 3^2}{8} = 27 \text{ kN-m}$$



Q.2 (b) Solution:

Characteristics of high strength concrete:

1. **Workability :** In early stages of its development, the high strength concrete has a tendency to be sticky and stiff due to large amounts of fines (high cement content, and pozzolana), a low w/c ratio and a normal water reducing admixture. However,

with the advent of superplasticizers, it is possible to have desired high workability without causing segregation even at a lower w/c ratio of 0.30.

2. **Strength:** The most noteworthy point about high strength concrete is its capacity to develop strength at a rapid rate without steam curing. Concrete can develop 20 to 27 MPa on normal curing within 24 hours and the high strength concrete can develop 42 MPa in 12 hours and 64 MPa in 24 hours.
3. **Microstructure, stress-strain relation, creep and fracture:** As a result of reduction in the size and the number of micro-cracks in high strength concrete, its stress strain relationship, creep and fracture behaviour is different from the normal concrete. High strength concrete, having compressive strength in the range of 30-75 MPa behave more like a homogenous material as compared to PCC.
4. **Durability :** It has been found that primarily due to low permeability, high strength concrete exhibits excellent durability to various physical and chemical agents that are normally responsible for concrete deterioration. Due to high cement content, thermal cracking can be problematic in structures using high strength concrete.

Applications of high strength concrete:

- The use of the highest possible strength concrete and minimum steel offers the most economical solution for columns of high rise buildings.
- In industrial application, high strength concrete is limited to structural members that are not exposed to freeze and thaw cycles. Such applications include floors in chemical and food industry and bridge deck overlays that are subjected to severe chemical and physical processes of degradation.
- High strength concrete is being increasingly selected for the construction of islands in ocean due to high durability in sea water environment.

Q.2 (c) Solution:

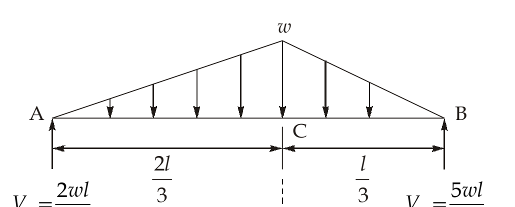


Fig. (i)

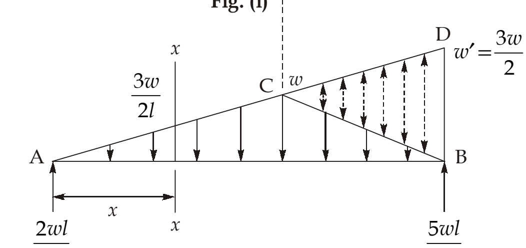


Fig. (ii)

Figure (i) shows the beam carrying the loading mentioned in the problem.

$$V_A + V_B = \frac{1}{2} \times w \times l = \frac{wl}{2}$$

Taking moment about A

$$V_B \times l = \frac{1}{2} \times \frac{2l}{3} \times w \times \frac{2l}{3} \times \frac{2}{3} + \frac{1}{2} \times \frac{l}{3} \times w \times \left(\frac{2l}{3} + \frac{l}{3} \times \frac{1}{3}\right)$$

$$V_B\, l = \left[\frac{4}{27} + \frac{7}{54}\right] wl^2$$

$$\Rightarrow \qquad V_B = \frac{15}{54} wl = \frac{5}{18} wl$$

$$\Rightarrow \qquad V_A = \frac{2}{9} wl$$

In figure (ii) the load diagram is rearranged.

Let the line AC of the load diagram be produced and the point D be located.

$$\text{The ordinate BD} = w = \left(\frac{l}{\frac{2l}{3}}\right) w = \frac{3}{2} w$$

Hence we shall consider that the loading on the beam consists of (i) downward triangular loading whose intensity varies from zero at the left end to $\frac{3w}{2}$ per unit run at the right end, and (ii) an upward triangular loading acting for a distance $\frac{l}{3}$ from the right end whose intensity varies from zero at $\frac{l}{3}$ from the right end to $\frac{3w}{2}$ at the right end.

Now following Macaulay's method, the bending moment at any section distant x from the left end is given by

$$EI \frac{d^2y}{dx^2} = \frac{2}{9} wlx - \frac{1}{2} x \cdot \frac{3w}{2l} x \frac{x}{3} + \left| \frac{1}{2}\left(x - \frac{2l}{3}\right) \frac{\left(x - \frac{2l}{3}\right)}{\frac{l}{3}} \frac{3w}{2} \frac{\left(x - \frac{2l}{3}\right)}{3} \right.$$

$$EI \frac{d^2y}{dx^2} = \frac{2}{9} wlx - \frac{wx^3}{4l} \left| + \frac{3w}{4l}\left(x - \frac{2l}{3}\right)^3 \right.$$

Integrating, we get, $$EI\frac{dy}{dx} = \frac{wlx^2}{9} - \frac{wx^4}{16l} + C_1 \Big| + 3w\frac{\left(x - \frac{2l}{3}\right)^4}{16l}$$

Integrating again, we get, $$EIy = \frac{wlx^3}{27} - \frac{wx^5}{80l} + C_1x + C_2 \Big| + \frac{3w\left(x - \frac{2l}{3}\right)^5}{80l}$$

At $x = 0, y = 0$

$\therefore$ $C_2 = 0$

$\therefore$ $x = l, y = 0$

$$\therefore \quad 0 = \frac{wl^4}{27} - \frac{wl^4}{80} + C_1 l + \frac{3wl^5}{80 \times 81 \times 3l}$$

$$\therefore \quad 0 = \frac{wl^4}{27} - \frac{wl^4}{80} + C_1 l + \frac{wl^4}{6480}$$

$$\therefore \quad C_1 = -\frac{2wl^3}{81}$$

For maximum deflection which will occur between $x = 0$ and $x = \frac{2l}{3}$, equate the expression for slope to zero.

We have, $$0 = \frac{wlx^2}{9} - \frac{wl^4}{16l} - \frac{2wl^3}{81}$$

Putting $x = Kl$, we have

$$\frac{wl}{9}K^2l^2 - \frac{w}{16}K^4l^3 - \frac{2}{81}wl^3 = 0$$

$$\therefore \quad \frac{K^2}{9} - \frac{K^4}{16} - \frac{2}{81} = 0$$

$$\therefore \quad K^4 - \frac{16}{9}K^2 + \frac{32}{81} = 0$$

Solving as a quadratic in K^2, we get

$$K^2 = \frac{\frac{16}{9} \pm \sqrt{\frac{256}{81} - \frac{128}{81}}}{2}$$

$$\therefore \quad K^2 = 1.517, 0.2603$$

$$\Rightarrow \quad K = 1.232, 0.5103$$

Neglecting the value of K = 1.232, maximum deflection occurs at a distance of 0.5103l from the left end A.

Distance of the point of maximum deflection from the middle point

$$= 0.5103l - 0.5l = 0.0103l = 0.011 \text{ (approximately)}$$

$$Ely_{\max} = \frac{wl}{27}(0.5103l)^3 - \frac{w}{80l}(0.5103l)^5 - \frac{2}{81}wl^3(0.5103l)$$

$$= -0.008108\frac{wl^4}{EI}$$

Q.3 (a) Solution:

The principal compounds in portland cement are known as Bogue's compounds. They are as follows:

(i) Tricalcium silicate (C_3S) or Alite

(ii) Dicalcium silicate (C_2S) or Belite

(iii) Tricalcium aluminate (C_3A) or Celite

(iv) Tetracalcium aluminoferrite (C_4AF) or Felite

The reaction of water with C_3A is very fast and in the process **flash setting** i.e. stiffening without strength development can occur because the C-A-H phase prevents the hydration of C_3S and C_2S. To prevent this flash set, gypsum is added at the time of grinding the cement clinker. The hydrated aluminates do not contribute anything to the strength of concrete. On the other hand, their presence is harmful to the durability of concrete particularly, where the concrete is likely to be attacked by sulphates. As it hydrates fast, it may contribute a little to the early strength.

$$C_3A + H_2O \rightarrow C_3AH_6$$

On hydration, C_4AF is believed to form a system of the form C-F-H. A hydrated calcium ferrite of this form is comparatively more stable. This hydrated product also does not contribute anything to the strength. However, the hydrates of C_4AF show a comparatively higher resistance to the attack of sulphates than the hydrates of calcium aluminate.

$$C_3AF + H_2O \rightarrow C_3AH_6 + CFH$$

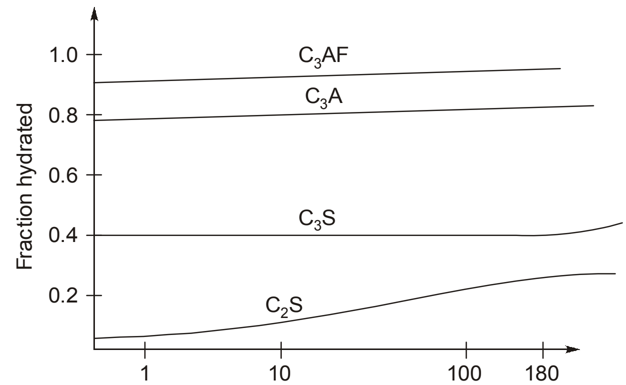
When C_3S and C_2S reacts with water, calcium silicate hydrate (C-S-H) and calcium hydroxide are formed. Calcium silicate hydrates are the most important products. It is the essence that determines the good properties of concrete.

$$C_3S + H_2O \rightarrow C_3S_2H_3 + 3Ca(OH)_2$$

$$C_2S + H_2O \rightarrow C_3S_2H_3 + Ca(OH)_2$$

It can be seen that C_3S produces a comparatively lesser quantity of calcium silicate hydrates and more quantity of $Ca(OH)_2$ than that formed in the hydration of C_2S. C_2S hydrates rather slowly than C_3S.

The relative rates of hydration of Bogue's compounds can be shown with the help of a graph as shown.



Rate of hydration of Bogue's Compounds

The approximate proportions of Bogue's compound are:

Compound	Percentage
C_3S	30-50
C_2S	20-45
C_3A	8-12
C_4AF	6-10

Q.3 (b) Solution:

Influence line for BM at *F*

When load is on AB, at x distance from A

$$R_B = \frac{x}{5}$$

This reaction transformed to beam CE at B′

Taking moments about C

$$R_D \times 6 = 2 \times R_B$$

$$R_D = \frac{R_B}{3} = \frac{1}{3} \times \frac{x}{5}$$

$$R_D = \frac{x}{15}$$

$$M_F = R_D \times 2 = \frac{x}{15} \times 2$$

At $x = 0$; $M_F = 0$

$x = 5$; $M_F = \frac{2}{3}$

When load is on B′F at a distance y from C

Taking moments about C

$$R_D \times 6 = y$$

$$R_D = \frac{y}{6}$$

$$M_F = R_D \times 2 = \frac{y}{6} \times 2 = \frac{y}{3}$$

At $y = 4$; i.e. load is at F

$$M_F = \frac{4}{3}$$

When load is in FE at 'y' distance from C

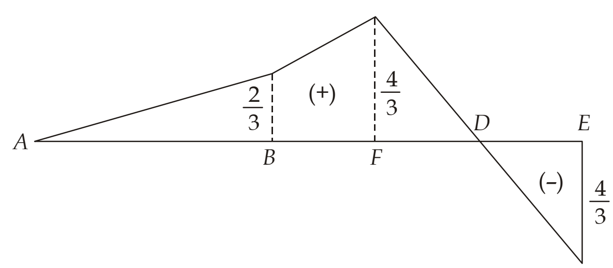
$$M_F = R_D \times 2 - 1 \times (y - 4)$$

$$= \frac{y}{6} \times 2 - (y - 4)$$

At $y = 6$, $M_F = 0$

At $y = 8$, $M_F = \frac{8}{3} - 4 = \frac{-4}{3}$

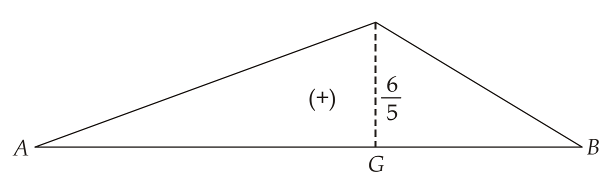
$\therefore$ ILD for BM at F is as shown in figure below:



ILD for BM at G

Point G is on simply supported beam AB. Influence lines for BM at G will be same as that for a point load on a simply supported beam.

$\therefore$ ILD for BM at G is as shown below.



The maximum BM at F occurs when portion AB and BE is loaded with uniformly distributed load and point load is at F.

$\therefore$ Maximum positive BM

$$= 2000\left[\frac{1}{2}\times\frac{2}{3}\times 5+\frac{1}{2}\left(\frac{2}{3}+\frac{4}{3}\right)\times 2+\frac{1}{2}\times\frac{4}{3}\times 2\right]+\frac{4}{3}\times 6000$$

$$= 10000 + 8000 = 18000 \text{ Nm} \quad \text{i.e. } 18 \text{ kNm}$$

Maximum –ve BM occurs when portion DE is loaded and point load is at E

$$\therefore \quad BM_{-ve} = \frac{1}{2}\times\frac{4}{3}\times 2\times 2000+\frac{4}{3}\times 6000$$

$$= 2666.6 + 8000$$

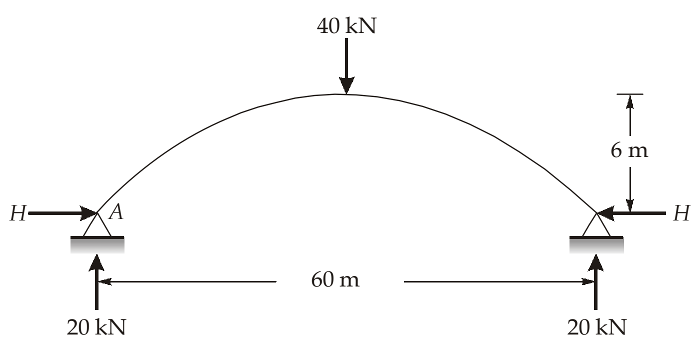
$$= 10666.6 \text{ Nm}$$

$$= 10.67 \text{ kNm}$$

Q.3 (c) Solution:

Horizontal thrust at each support in a two hinged arch is given by,

$$H = \frac{\int_0^l \frac{M_s y dx}{EI_C} + \alpha t L}{\int_0^l \frac{y^2 dx}{EI_C} + \frac{l}{EA_C} + k}$$



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

$$= \frac{4\times 6\times x(60-x)}{60\times 60} = \frac{x}{150}(60-x)$$

$$\int_0^l \frac{M_s y ds}{EI_C} = \frac{1}{EI_C}\times 2\int_0^{30} 20x\frac{x}{150}(60-x)\,dx$$

$$= \frac{4}{15EI_C}\int_0^{30}\left(60x^2-x^3\right)dx$$

$$= \frac{4}{15}\left[60\frac{x^3}{3} - \frac{x^4}{4}\right]_0^{30}$$

$$= \frac{90000}{EI_C}$$

$$\int_0^l \frac{y^2 dx}{EI_C} = \frac{2\int_0^{30}\left(\frac{x}{150}(60-x)\right)^2 dx}{EI_C} = \frac{32 \times 36}{EI_C}$$

$$EI_C = \frac{10 \times (1000)^2 \times 600000}{(100)^4} = 60000 \text{ kNm}^2$$

$$EA_C = 10 \times 10^2 \times 1000 = 10^6 \text{ kN}$$

$$k = 2 \times 0.006 \times \frac{1}{100} \times \frac{1}{10} = 0.12 \times 10^{-4} \text{ m/kN}$$

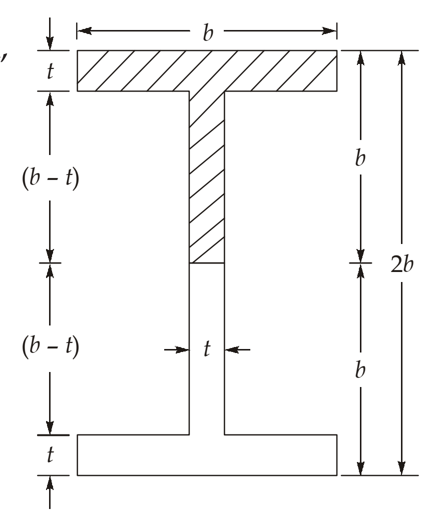
$$\therefore \quad H = \frac{\frac{90000}{60000} + 11 \times 10^{-6} \times 20 \times 60}{\frac{32 \times 36}{60000} + \frac{60}{10^6} + 0.12 \times 10^{-4}}$$

$$= \frac{90000 + 792}{1152 + 3.6 + 0.72} = 78.52 \text{ kN}$$

Q.4 (a) Solution:

(i)

Let the S.F. on the section be S. Area of the beam section,



$$A = 2bt + t\,(2b - 2t)$$

$$A = 2bt + 2bt - 2t^2$$

$$A = 4bt - 2t^2 \simeq 4bt$$

(Since $2t^2$ is a small quantity).

Moment of inertia of the section about the neutral axis

$$I = \frac{b(2b)^3}{12} - \frac{(b-t)(2b-2t)^3}{12}$$

$$= \frac{2}{3}b^4 - \frac{2}{3}(b-t)^4 = \frac{2}{3}\left[b^4 - (b-t)^4\right]$$

$$= \frac{2}{3}\left[b^4 - (b^4 - 4b^3 t)\right]$$

(Omitting terms involving t^2, t^3 and t^4 as these are very small)

$$= \frac{2}{3} \times 4b^3 t = \frac{8}{3} b^3 t$$

Shear stress is maximum at the neutral axis.

$$\therefore \quad q_{max} = \frac{Sa\bar{y}}{It}$$

Where $a\bar{y}$ = moment of the shaded area about the neutral axis

$$= bt\left(b - \frac{t}{2}\right) + \frac{(b-t)^2 t}{2} = b^2 t - \frac{bt^2}{2} + \frac{b^2 t}{2} - bt^2 + \frac{t^3}{2}$$

$= \frac{3}{2} b^2 t$ (neglecting terms involving t^2 and t^3)

$$q_{max} = \frac{S \times \frac{3}{2} b^2 t}{\frac{8}{3} b^3 t \times t} = \frac{9}{16} \frac{S}{bt}$$

Average shear stress,

$$q_{average} = \frac{S}{A} = \frac{S}{4bt}$$

$$\therefore \quad \frac{q_{max}}{q_{average}} = \frac{9}{16} \cdot \frac{S}{bt} \cdot \frac{4bt}{S} = \frac{9}{4} = 2.25$$

(ii)

For the upper cantilever,

Deflection at B $\quad \delta_b = \frac{Pl^3}{3EI}$

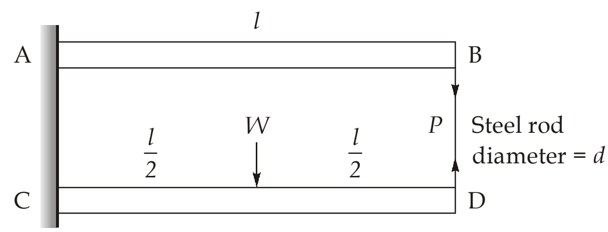
For the lower cantilever,

Deflection at D $\quad \delta_d = \frac{W\left(\frac{l}{2}\right)^3}{3EI} + \frac{W\left(\frac{l}{2}\right)^2}{2EI} \cdot \frac{l}{2} - \frac{Pl^3}{3EI} = \frac{5}{48} \frac{Wl^3}{EI} - \frac{Pl^3}{3EI}$

$\therefore$ Extension of the vertical rod $= \delta_d - \delta_b$

$$\Rightarrow \quad \frac{4Pa}{\pi d^2 E} = \frac{5}{48}\frac{Wl^3}{EI} - \frac{Pl^3}{3EI} - \frac{Pl^3}{3EI}$$

$$\Rightarrow \quad \frac{4Pa}{\pi d^2 E} = \frac{5}{48}\frac{Wl^3}{EI} - \frac{2}{3}\frac{Pl^3}{EI}$$



$$\Rightarrow \quad P\left[\frac{4a}{\pi d^2} + \frac{2l^3}{3I}\right] = \frac{5}{48}\frac{Wl^3}{I}$$

$$\Rightarrow \quad P = \frac{\frac{5}{48}\frac{Wl^3}{I}}{\frac{4a}{\pi d^2} + \frac{2l^3}{3I}}$$

$$\Rightarrow \quad P = \frac{5Wl^3}{\frac{192\,aI}{\pi d^2} + 32l^3}$$

$$\Rightarrow \quad P = \frac{5Wl^3}{32\left(l^3 + \frac{6\,aI}{\pi d^2}\right)}$$

Q.4 (b) Solution:

(i)

Pulley A is subjected to a torque of (3500 – 1050) 100 Nmm = 245000 Nmm

Pulley B is subjected to a torque of (2100 – 1160) 125 Nmm = 117500 Nmm

Pulley C is subjected to a torque of (2000 – 980) 125 Nmm = 127500 Nmm

Studying the above torques, we conclude that power is supplied to the shaft at A and power is taken off the shaft from B and C.

$\therefore$ Maximum torque on the shaft = 245000 Nmm

$$\because \quad T = f_s \frac{\pi d^3}{16}$$

$$\Rightarrow \quad 245000 = 60\frac{\pi d^3}{16}$$

$$\Rightarrow \quad d^3 = \frac{245000 \times 16}{60\pi}$$

$$\Rightarrow \quad d = 27.5 \text{ mm}$$

$\therefore$ Minimum diameter of shaft required is 27.5 mm.

(ii)

Safe axial tensile stress $f = \frac{350}{2.5} = 140\text{ N/mm}^2$

Let the area of the square section be a mm^2

Direct stress $p = \frac{40000}{a}\text{N/mm}^2$

Shear stress $q = \frac{15000}{a}\text{N/mm}^2$

$\therefore$ The principal stresses are,

$$p_{1,\,2} = \frac{p}{2} \pm \sqrt{\left(\frac{p}{2}\right)^2 + q^2}$$

$$= \frac{20000}{a} \pm \sqrt{\left(\frac{20000}{a}\right)^2 + \left(\frac{15000}{a}\right)^2} = \frac{20000}{a} \pm \frac{25000}{a}$$

$\therefore$ Major principal stress $p_1 = \frac{20000}{a} + \frac{25000}{a} = \frac{45000}{a}\text{N/mm}^2$

Major principal stress $p_2 = \frac{20000}{a} - \frac{25000}{a} = -\frac{5000}{a}\text{N/mm}^2$

Strain energy stored per unit volume,

$$u = \frac{1}{2E}\left[p_1^2 + p_2^2 - 2\mu p_1 p_2\right]$$

$$= \frac{1}{2E}\left[\left(\frac{45000}{a}\right)^2 + \left(-\frac{5000}{a}\right)^2 - 2\times 0.3\left(\frac{45000}{a}\right)\left(-\frac{5000}{a}\right)\right]$$

$$= \frac{1}{2E}\cdot\frac{21.85\times 10^8}{a^2}$$

$$\therefore \quad \frac{1}{2E}\cdot\frac{21.85\times 10^8}{a^2} = \frac{f^2}{2E} = \frac{140^2}{2E}$$

$$\Rightarrow \quad a^2 = \frac{21.85\times 10^8}{140^2}$$

$$\Rightarrow \quad a = 333.8856\text{ mm}^2$$

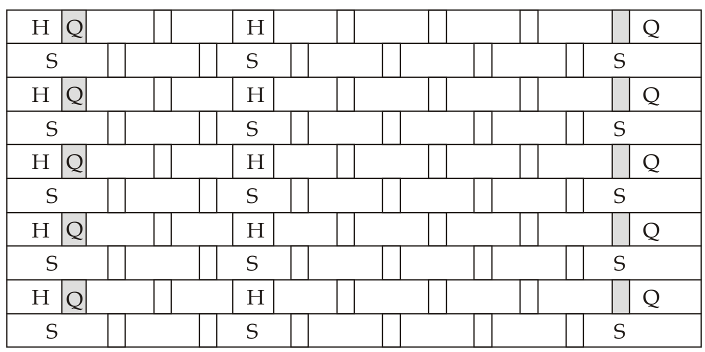
$\therefore$ Side of the square section $= \sqrt{333.8856} = 18.273$ mm

Q.4 (c) Solution:

Importance of bonding in brickwork: The bond is the interlacement as interlocking of bricks formed when they lay (or project beyond) those immediately below or above them.

The art of bonding brickwork consists of the orderly arrangement of the bricks in such a way that continuous or through joints at right angles to the face of the wall are eliminated and longitudinal through joints along the wall are also reduced to a minimum. Bonding helps in distribution of loads. Bonding is carried by use of closures (in the header course) or three quarters in the stretcher courses.

Single flemish bond : Single flemish bond is comprised of double flemish bond facing and English bond backing and hearing in each course. This bond thus uses the strength of the English bond and appearance of flemish bond.



Flemish bond

This bond can be used for those walls having thickness at least equal to $1\frac{1}{2}$ brick. Double flemish bond facing is done with good quality expensive bricks.

Double flemish bond : In double flemish bond, each course presents the same appearance both in front face as well as in the back face. Alternate headers and stretchers are laid in each course.

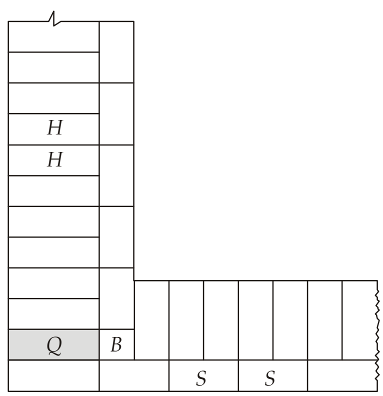
Because of this, double flemish bond presents better appearance than English bond.

Comparison of English bond and Flemish bond:

1. English bond in stronger than flemish bond for walls thicker than $1\frac{1}{2}$ brick.
2. Flemish bond give more pleasing appearance than the English bond.
3. Broken bricks can be used in the form of bats in Flemish bond. However, more mortar is required.

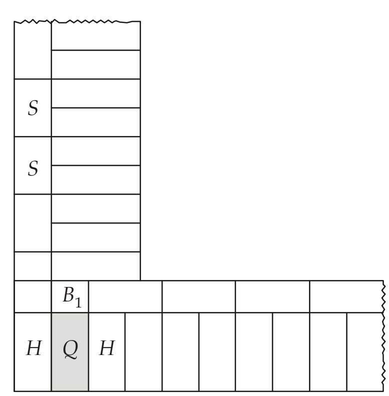
English bond : This bond consists of alternate courses of header and stretchers. In this arrangement, vertical joints in the header courses came over each other and the vertical joints in the stretcher course are also in the same line. For breaking of vertical joints in the successive course, it is essential to place queen closer after the first header in each heading course. The following additional points should be noted:

1. A heading course should never start with queen closer as it is liable to get displaced in this position.
2. In the stretcher course, the stretcher should have a minimum lap of 1/4th their length over the headers.
3. In walls having their thickness equal to odd number of half brick, i.e., $1\frac{1}{2}$ brick thick wall or $2\frac{1}{2}$ brick thick wall and so on, the same course will show stretchers on one face and headers on the other.



1, 3, 5 ..... courses

Plane for 1½ brick thick wall

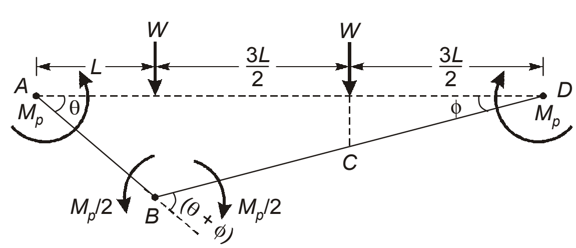


2, 4, 6 courses

Section B

Q.5 (a) Solution:

Collapse mechanism-I



$\because$ $\quad L\theta = 3L \times \phi$

$\Rightarrow$ $\quad \theta = 3\phi$ ...(i)

Internal work

$$M_p\theta + \frac{M_p}{2}(\theta+\phi) + M_p\phi = 6\,M_p\phi$$

By eq. (i) and eq. (ii)

External work

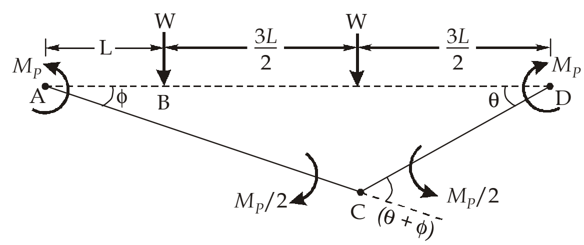
$$W\theta \times L + W \times \frac{3L}{2}\phi = 4.5\, WL\phi$$

External work = Internal work

$$4.5\, WL\phi = 6\, M_p\phi$$

$$W_{u_1} = \frac{4}{3}\left(\frac{M_p}{l}\right)$$

Collapse mechanism-II



$\because$
$$\frac{5L}{2}\phi = \frac{3L}{2}\theta$$

$$5\phi = 3\theta \qquad \text{...(i)}$$

External work

$$WL\phi + W \times \frac{3L}{2}\theta = 3.5\, WL\phi \qquad \text{...(ii)}$$

By eq. (i) and (ii)

Internal work

$$M_P\phi + \frac{M_P}{2}(\theta + \phi) + M_P\theta = 4\, M_p\phi$$

External work = Internal work

$$3.5\, WL\phi = 4M_p\phi$$

$$W_{u_2} = \frac{4}{3.5} = \frac{8}{7}\frac{M_P}{L}$$

Collapse load is $\frac{8}{7}\frac{M_P}{L}$ [Minimum of W_{u_1} and W_{u_2}]

Q.5 (b) Solution:

(i) Zone factor : It is the factor to obtain the design spectrum depending on the perceived maximum seismic risk characterised by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimates of effective peak ground acceleration.

(ii) **Importance factor :** It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterised by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.

(iii) **Response reduction factor :** It is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE), shaking shall be reduced to obtain the design lateral force.

(iv) **Soft storey :** It is the one whose lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

Q.5 (c) Solution:

$$\text{Tension} = \text{Compression}$$

$$\Rightarrow \quad 0.87 \times f_y A_{st} = 0.36 \times f_{ck} x_u B$$

$$\Rightarrow \quad 0.87 \times 415 \times \frac{\pi}{4} \times 25^2 \times 4 = 0.36 \times 20 \times x_u \times 300$$

$$\Rightarrow \quad x_u = 328.20 \text{ mm}$$

For Fe415

$$x_{u\,\lim} = 0.48\, d$$

$$= 0.48 \times 655 = 314.4 \text{ mm} < 328.20 \text{ mm}$$

So section is over reinforced so MOR is limited to $M_{u\,\lim}$

$$M_{u\,\lim} = 0.36 \times f_{ck}\, x_u\, B\, [d - 0.42\, x_{u\,\lim}]$$

$$= 0.36 \times 20 \times 314.4 \times 300\, [655 - 0.42 \times 314.4]$$

$$= 355.14 \times 10^6 \text{ Nmm} = 355.14 \text{ kNm}$$

$$W_u = \frac{8M_{u\lim}}{l^2} = \frac{8 \times 355.14}{7^2} = 57.98 \text{ kN/m}$$

$$\text{So load factor} = \frac{57.98}{38} = 1.53$$

Q.5 (d) Solution:

Span of the beam AB, $L = 20$ m

Eccentricity at centre of span, $e = 400$ mm

Coefficient of friction, $\mu = 0.35$

Friction coefficient for wave effect,

$$K = 0.15 \text{ for } 100 \text{ m or } 0.0015/\text{m}$$

The cable is parabolic between the supports A and B with an eccentricity of 40 mm at the centre of span AB i.e., at C. (say).

$$\text{Slope of cable at end support, } A = \left[\frac{4e}{L}\right] = \left[\frac{4 \times 400}{20 \times 1000}\right] = 0.08$$

Cumulative angle between tangents at A and B = α = (2 × 0.08) = 0.16 radians.

$$\text{Loss of stress due to friction} = P_o[\mu\alpha + Kx]$$

where x = distance from the jacking end to the point under consideration.

(a) Loss of stress between A and B ($x = L = 20$ m)

$$= 1000\,[(0.35 \times 0.16) + (0.0015 \times 20)] = 86 \text{ N/mm}^2$$

$$\text{Effective stress at B} = [1000 - 86] = 914 \text{ N/mm}^2$$

(b) If the cable is tensioned simultaneously from both ends A and B, the minimum stress will occur at the centre of span C.

Cumulative angle between A and C = α = 0.08 radians

Loss of stress between A and C ($x = 0.5$, $L = 10$ m)

$$= 1000\,[(0.35 \times 0.08) + (0.0015 \times 10)] = 43 \text{ N/mm}^2$$

Effective stress at the centre of span C = [1000 – 43] = 957 N/mm^2

Q.5 (e) Solution:

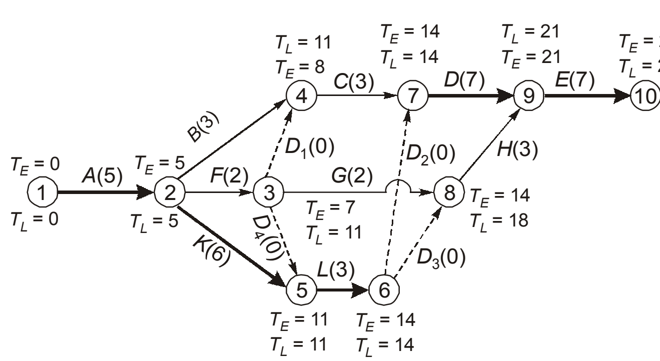
Network diagram is drawn and computation of T_E and T_L is done on network itself.

We know,

For activity, i - j

$$T_E^j = \left(T_E^i + t_{ij}\right)_{\max}$$

$$T_L^i = \left(T_L^j - t_{ij}\right)_{\min}$$



Project duration = 28 days, Critical path is 1 - 2 - 5 - 6 - 7 - 9 - 10

Activity	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*K*	*L*
t_{ij}	5	3	3	7	7	2	2	3	6	3
EST	0	5	8	14	21	5	7	14	5	11
LST	0	8	11	14	21	9	16	18	5	11
F_T	0	3	3	0	0	4	9	4	0	0

where, EST = Earliest start time = T_E^i

LST = Latest start time = $T_L^j - t_{ij}$

F_T = Total float

= LST – EST

Q.6 (a) Solution:

Area of the connected leg = (125 – 5) 10 = 1200 mm^2

Area of the outstanding leg = (95 – 5) 10 = 900 mm^2

Factored tension in the angle member = 450 kN

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

Force in the connected leg = 450 – 192.9 = 257.1 kN

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

= 1.2 × 192.9 = 231.48 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.

Size of weld be 6 mm

Strength of the weld per mm length

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

$\because$ Length of weld required for connecting the 100 mm leg of the lug angle to the gusset plate

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

$\because$ Force in the connected leg of the main angle = 257.1 kN

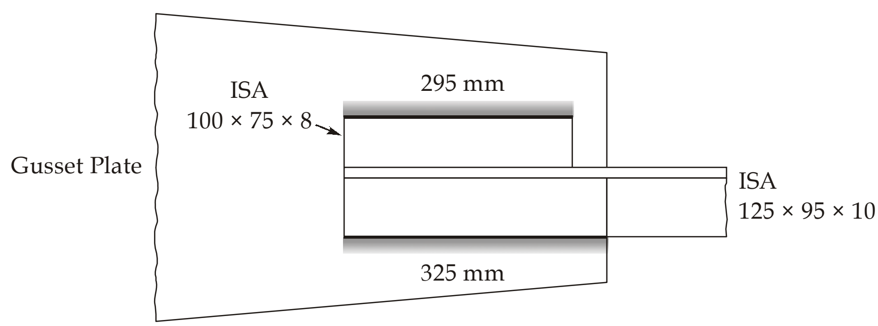
$\therefore$ Length of weld required to connect the main angle to the gusset plate

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

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.6 (b) Solution:

(i)

Item rate contract: Item rate contract is also known as unit price contract or schedule contract. In this, the contractor undertakes the execution of the work on an item rate basis. The contractor is required to quote rate for individual items of work on the basis of schedule of quantities (i.e. bill of quantities) furnished by the department. The amount to be received by the contractor, depends upon the quantities of work actually performed.

Suitability: The item rate contract is most commonly used for all types of engineering works of the government and its undertakings (PSUs). It is suitable for works which can be distinctly split into various items and quantities under each item can be estimated accurately.

Merits:

(a) This method ensures a very detailed analysis of cost and payment to the contractor and is also based upon detailed measurements of each item actually done and so this method is more scientific.

(b) Changes in drawings and quantities of individual items can be made as per the requirements within agreed limits.

(c) There is no urgency of providing detailed drawings at the time of awarding the contract. It can be prepared later on.

(d) The contractor is asked to write down the rate of individual items in figures and words both so it does not become difficult during submission of tender.

(e) An engineer can compare the rates quoted by the contractor with that of schedule of rates prepared by the department to find out whether the tender is unbalanced.

Demerits:

(a) A contractor may quote high rate for items whose rates are likely to be increased and low rate for items whose rate are likely to be decreased, making an unbalanced tender and consequently the department may stand to loose substantially.

(b) Comparative statement of item rate tenders are more elaborate and comprehensive and an intelligent scrutiny is required.

(c) A contractor may quote some item rates in words excluding paise intentionally in order to tamper in rates.

(d) The total cost of work can only be known after completion. As such the owner may face financial difficulty if the final cost is substantially high.

(e) Additional staff is required to take detailed measurements.

(f) The scope of savings with use of inferior quality materials may prompt the contractor to perform sub-standard work.

(ii)

Soil compaction is achieved by rolling, ramming or by vibration.

The following equipments are used for compacting soils:

1. **Smooth wheeled rollers:** The smooth wheeled rollers are suitable to roll a wide range of soils, preferably granular soils and pavement materials for the various layers. These are particularly found to be useful in compacting soils and other materials where a crushing action is advantageous.
2. **Pneumatic tyre roller:** In Pneumatic tyre rollers, in addition to the direct pressure due to rolling, also a slight kneading action on the soil. Hence these are considered to be most suitable to compacting nonplastic silts and fine sands.
3. **Sheepfoot rollers:** Sheepfoot rollers are considered more suitable to compact clayey soils. During rolling operation the soil under the projecting feet is compacted and also there is a considerable kneading action on the soil.

4. **Rammers:** Rammers are useful to compact relatively small areas and where the rollers cannot operate such as compaction of trenches, foundation and slopes.
5. **Vibrators:** Vibrators are most suited for compacting dry cohesionless granular materials. There are also vibrator mounted rollers to give the combined effects of rolling and vibration. Vibratory rollers are advantageously used in compacting a wide range of materials.

Q.6 (c) Solution:

For the given member, $f_y = 250$ MPa and $f_u = 410$ MPa

For 20 mm diameter bolts, diameter of bolt hole,

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

$$A_{nb} = 245 \text{ mm}^2 \text{ (Given)}$$

Design one-way shear strength of bolt,

$$V_{d,sb} = \frac{A_{nb} f_{ub}}{\sqrt{3}\gamma_{mb}} = \frac{245 \times 400}{\sqrt{3} \times 1.25} \times 10^{-3} \text{kN} = 45.26 \text{ kN}$$

$$\text{Design strength of bolt} = 45.26 \text{ kN}$$

$$\text{Number of bolts required} = \frac{900}{45.26} = 19.88 \simeq 20$$

So provide 5 rows of bolts with 4 bolt in one line.

The minimum required area from the consideration of yielding of cross-section,

$$A_g = \frac{T_u \gamma_{m0}}{f_y} = \frac{(900 \times 10^3) \times 1.10}{250} = 3960 \text{ mm}^2$$

Minimum required area from consideration of rupture at net section:

With lines of four or more bolts, $\alpha = 0.8$ (given) and thus

$$A_n = \frac{T_u \gamma_{m1}}{\alpha f_u} = \frac{(900 \times 10^3) \times 1.25}{(0.8 \times 410)} = 3430 \text{ mm}^2$$

With four bolts located at any cross-section;

$$A_g = A_n + \text{bolt hole area}$$
$$= 3430 + 4 \times 22 t_f$$

Minimum required value of radius of gyration, $r_{min} = \dfrac{l}{\lambda}$

$$= \left(\frac{8 \times 10^3}{300}\right) = 26.67 \text{ mm}$$

Selection of section: The lightest ISWB section satisfying above criteria is selected from steel tables given above.

Consider ISWB 225 rolled steel section with

A_g = 4324 mm^2; h = 225 mm, b_f = 150 mm, t_f = 9.9 and r_{min} = 32.2 mm

Gross area of the required cross-section,

Thus, A_g = 3430 + 4 × 22 × 9.9 = 4301 mm^2

Check for bearing of bolt

$$\text{Bearing strength} = 2.5 \times k_b \times \frac{f_u}{1.25} \times d \times t$$

$$= 2.5 \times 0.488 \times \frac{410}{1.25} \times 20 \times 9.9$$

$$= 79.23 \text{ kN} > 45.26 \text{ kN}$$ So OK

Q.7 (a) Solution:

For Fe410 (E250) grade of steel

f_u = 410 N/mm^2, f_y = 250 N/mm^2

$$M_{u,\max} = 150 \text{ kNm}$$

$$V_{u,\max} = 210 \text{ kN}$$

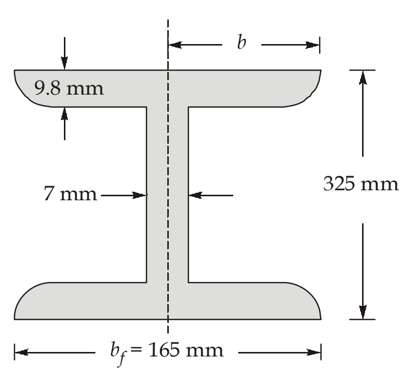
$$z_{p,\text{req}} = \frac{M}{(f_y / \gamma_{m_0})} = \frac{150 \times 10^6}{(250/1.1)} = 660 \times 10^3 \text{ mm}^3$$

Let us adopt ISLB 325

$$z_{pz} = 687.76 \times 10^3 \text{ mm}^3$$

Classification of section:

R_1 = 16 mm



Outstand of flange, $\frac{b}{t_f} = \frac{165/2}{9.8} = 8.42 < 9.4\varepsilon$ where $\in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

Flange is plastic

Web, $d = h - 2(t_f + R_1) = 325 - 2(9.8 + 16)$

$= 273.4$ mm

$$\therefore \qquad \frac{d}{t_w} = \frac{273.4}{7} = 39.06 < 84\varepsilon$$

Web is plastic

Also $\frac{d}{t_w} < 67\varepsilon$ and thus web is stocky and so shear buckling check is not required.

$\therefore$ Whole cross-section is plastic.

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} \times ht_w = \frac{250}{\sqrt{3} \times 1.1} \times 325 \times 7 \times 10^{-3} \text{ kN} = 298.52 \text{ kN}$$

Given, $V_u = 210$ kN

$\therefore \quad V_u < V_d$

$\therefore$ Section is safe in shear

Also, $0.6V_d = 0.6 \times 298.52 = 179.112$ kN

$\therefore \quad V > 0.6V_d$

$\therefore$ It is a high shear case:

Check for bending:

For high shear case

$$M_{dv} = M_d - \beta(M_d - M_{fd})$$

$$M_d = \beta_b Z_{pz} \times \frac{f_y}{\gamma_{m0}} \leq 1.2 z_e \times \frac{f_y}{\gamma_{m0}}$$

where $\beta_b = 1$ For plastic section

$$= \left(1.0 \times 687.76 \times 10^3 \times \frac{250}{1.1}\right) \times 10^{-6} \leq \left(1.2 \times 607.7 \times 10^3 \times \frac{250}{1.1}\right) \times 10^{-6}$$

$$= 156.309 \text{ kNm} < 165.74 \text{ kNm} \qquad \text{(OK)}$$

$$\therefore \qquad M_d = 156.309 \text{ kNm}$$

Plastic design strength of the area of cross-section excluding the shear area.

$$M_{fd} = \frac{f_y}{\gamma_{m0}}\left(z_{pz} - \frac{t_w h^2}{4}\right) = \frac{250}{1.1}\left(687.76 \times 10^3 - \frac{7 \times 325^2}{4}\right) \times 10^{-6}$$

$$= 114.3 \text{ kNm}$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 210}{298.516} - 1\right)^2 = 0.166$$

$$M_{dv} = 156.309 - 0.166\,(156.309 - 114.3)$$

$$= 149.34 \text{ kNm} < (M_{u,\max} = 150 \text{ kNm})$$

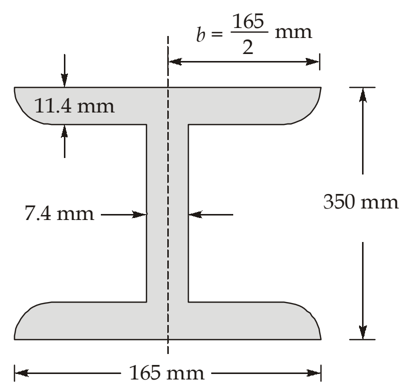
(Not safe in bending);

Now, let us adopt ISLB 350

$$z_{pz} = 851.11 \times 10^3 \text{ mm}^3 > 660 \times 10^3 \text{ mm}^3$$

Classification the section:

Outstand of flange: $$\frac{b}{t_f} = \frac{165/2}{11.4} = 7.23 < 9.4\varepsilon$$



Flange is plastic

Web, $$d = h - 2(t_f + R_1)$$
$$= 350 - 2(11.4 + 16) = 295.2 \text{ mm}$$
$$\frac{d}{t_w} = \frac{2952.2}{7.4} = 39.89 < 84\varepsilon$$

∴ Web is plastic.

Hence, overall section is plastic

Also $\frac{d}{t_w} < 67\varepsilon$, web is stockey and shear buckling check is not required.

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} \times ht_w = \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 7.4 \times 10^{-3} = 339.85 \text{ kN}$$

But $V_u = 210 \text{ kN} < V_d$ (Safe in shear) (OK)

Also, $0.6V_d = 0.6 \times 339.85 = 203.91 \text{ kN}$

∴ $V > 0.6V_d$ ∴ It is high shear case.

Check for bending:

For high shear case

$$M_{dv} = M_d - \beta(M_d - M_{fd})$$
$$M_d = \beta_b z_{pz} \times \frac{f_y}{\gamma_{m0}} \leq 1.2 z_{ez} \times \frac{f_y}{\gamma_{m0}}$$
$$= \left(1.0 \times 851.11 \times 10^3 \times \frac{250}{1.1}\right) \times 10^{-6}$$

$$\leq \left(1.2 \times 751.9 \times 10^3 \times \frac{250}{1.1}\right) \times 10^{-6}$$

$$= 193.43 \text{ kNm} < 205.06 \text{ kNm (OK)}$$

$$M_d = 193.43 \text{ kNm}$$

$$M_{fd} = \frac{f_y}{\gamma_{m0}}\left(z_{pz} - \frac{t_w h^2}{4}\right) = \frac{250}{1.1}\left(851.11 \times 10^3 - \frac{7.4 \times 350^2}{4}\right) \times 10^{-6}$$

$$= 141.93 \text{ kNm}$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2 \times 210}{339.85} - 1\right)^2 = 0.056$$

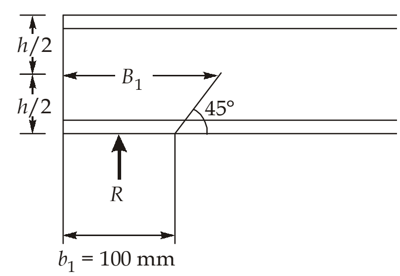
$$M_{dv} = 193.43 - 0.056(193.43 - 141.93)$$

$$= 190.55 \text{ kNm} > 150 \text{ kNm (Safe in bending);}$$

Check for web buckling:

Since $V > 0.6V_d$ so check for web buckling is required.

Stiff bearing length = 100 mm



$$B_1 = b_1 + \frac{h}{2} = 100 + \frac{350}{2} = 275 \text{ mm}$$

$$f_{wb} = \text{Web buckling strength}$$

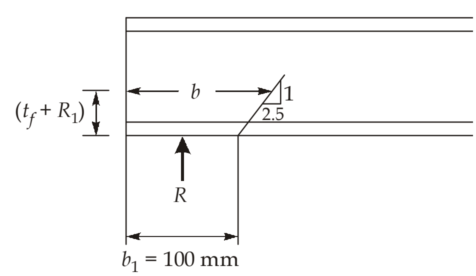
$$= B_1 t_w f_{cd}$$

$$\lambda = \frac{2.5 \times 295.2}{7.4} = 99.729$$

$$\therefore \quad f_{cd} = 121 + \frac{107 - 121}{100 - 90}(99.729 - 90) = 107.38 \text{ N/mm}^2$$

Now, $f_{wb} = (275 \times 7.4 \times 107.38) \times 10^{-3} = 218.52 \text{ kN} > V_{max}$ (OK)

Check for web crippling:



Bearing length, $b = b_1 + 2.5(t_f + R_1)$

$$= 100 + 2.5(11.4 + 16)$$

$$= 168.5 \text{ mm}$$

$$f_w = \frac{f_y}{\gamma_{m0}} \times b t_w$$

$$= \frac{250}{1.1} \times 168.5 \times 7.4 \times 10^{-3} = 283.386 \text{ kN} > V_{max} \text{ (OK)}$$

Safe in web crippling;

Q.7 (b) Solution:

(i) Total load = 20 + 30 = 50 kN/m

$\therefore$ Factored load, W_u = 50 × 1.5 = 75 kN/m

$\therefore$ Factored SF, $\quad V_u = \dfrac{75\times6}{2} = 225$ kN

Shear force at critical section (at distance d from face of support)

$$V_1 = \frac{225}{3}\times(3-0.6-0.115) = 171.375 \text{ kN}$$

Now, $\quad p_t = \dfrac{4\times\frac{\pi}{4}\times25^2}{600\times300}\times100 = 1.09\%$

So, $\quad V_c$ = Shear strength of concrete

$= 0.4144 \times 600 \times 300 \times 10^{-3}$ kN = 74.6 kN

So shear reinforcement is designed for

$$V_s = V_1 - V_c = 171.375 - 74.6 = 96.775 \text{ kN}$$

So, $\quad S_v = \dfrac{0.87\times f_y d_1\times A_{sv}}{V_s}$

$\Rightarrow \quad S_v = \dfrac{0.87\times415\times600\times\frac{\pi}{4}\times10^2\times2}{96.775\times1000} = 351.44$ mm c/c

Since, $\quad S_v$ = 351.44 mm > 280 mm (provided)

So shear reinforcement provided is adequate.

(ii) Since 2 bars are curtailed so thus

$$0.36\times25\times x_u\times300 = 0.87\times f_y\times2\times\frac{\pi}{4}\times25^2$$

$$x_u = 131.28 \text{ mm}$$

$x_{u\,\text{lim}} = 0.48\times600 = 288$ mm $> x_u$ so under-reinforced section.

$$\text{MOR} = 0.36\times f_{ck}\times x_u B\,[d - 0.42\,x_u]$$

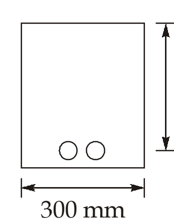
$$= 0.36\times25\times131.28\times300\,[600 - 0.42\times131.28]$$

$$= 193.13\times10^6 \text{ Nmm}$$

$$M_u = 193.13 \text{ kNm}$$

For adequate bond strength

$$\frac{M_u}{V_1} + l_o \ge l_d$$



$$V_1 = 225 \text{ kN}$$

$$l_d = \frac{0.87\,f_y\phi}{4\,\tau_{bd}}$$

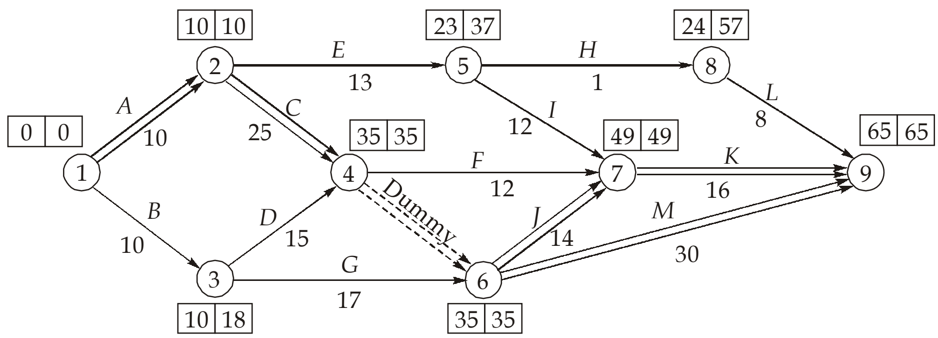
$$= \frac{0.87 \times 415 \times 25}{4 \times 1.4 \times 1.6} = 1007.4 \text{ mm}$$

So, $$\frac{193.13 \times 10^6}{225 \times 10^3} + l_o \geq 1007.4 \text{ mm}$$

$$\Rightarrow \quad l_o \geq 1007.4 - 858.36$$

$$\Rightarrow \quad l_o \geq 149.04 \text{ m} \simeq 150 \text{ mm (say)}$$

Q.7 (c) Solution:



Examining EST and LST marked in time box at each event, there are two critical paths viz.:

Critical path I

1 – 2 – 4 – 6 – 9 or A – C – Dummy – M

Critical path II

1 – 2 – 4 – 6 – 7 – 9 or A – C – Dummy –J – K

Project duration = 65 days.

The calculation of all activity times (EST, EFT, LST and LFT) and all floats (TF, FF, IF, INTF) of each activity is shown in the table below:

Activity	i-Node	j-Node	Duration (days)	Activity times (days)				Floats (days)				Remarks
				EST	EFT	LST	LFT	TF	FF	IF	INTF	
A	1	2	10	0	10	0	10	0	0	0	0	Critical
B	1	3	10	0	10	8	18	8	0	0	8	
C	2	4	25	10	35	10	35	0	0	0	0	Critical
D	3	4	15	10	25	20	35	10	10	2	0	
E	2	5	13	10	23	24	37	14	0	0	14	
F	4	7	12	35	47	37	49	2	2	2	0	
Dummy	4	6	0	35	35	35	35	0	0	0	0	Critical
G	3	6	17	10	27	18	35	8	8	0	0	
H	5	8	1	23	24	56	57	33	0	–14	33	
I	5	7	12	23	35	37	49	14	14	0	0	
J	6	7	14	35	49	35	49	0	0	0	0	Critical
K	7	9	16	49	65	49	65	0	0	0	0	Critical
L	8	9	8	24	32	57	65	33	33	0	0	
M	6	9	30	35	65	35	65	0	0	0	0	Critical

Q.8 (a) Solution:

Clear dimensions of slab = 4 m × 9 m

$$\frac{\text{Span}}{\text{Effective depth}} \not> 20 \text{ for spans upto 10 m}$$

Effective depth is decided according to short span.

$$\frac{4000}{20} = d$$

$$\therefore \quad d \not< \frac{\text{Span}}{20} = \frac{4000}{20} = 200 \text{ mm}$$

$$\text{Assuming width of support} \leq \frac{\text{Shorter span}}{12} = \frac{4000}{12} = 333.33 \text{ mm}$$

Let, w = Width of support = 300 mm

$$\therefore \quad \text{Effective span} = \text{Min.}\begin{cases}\text{Clear span + Effective depth}\\ \text{c/c distnace between the supports}\end{cases}$$

Effective in x-direction

$$l_{\text{eff }x} = \text{Min.}\begin{Bmatrix}4 + 0.2 = 4.2 \text{ m}\\ 4 + 0.3 = 4.3 \text{ m}\end{Bmatrix}$$

$$= 4.2 \text{ m}$$

$$l_{\text{eff }y} = \text{Min.}\begin{Bmatrix}9 + 0.2 = 9.2 \text{ m}\\ 9 + 0.3 = 9.3 \text{ m}\end{Bmatrix}$$

$$= 9.2 \text{ m}$$

$$\therefore \quad \frac{l_{eff\,y}}{l_{eff\,x}} = \frac{9.2}{4.2} = 2.19 > 2$$

$\therefore$ It is a one way slab.

Thus, slab will bend and curvature will occur along short direction.

Let nominal cover = 15 mm

Take 10 mm dia. bars.

$\therefore$ Oveall depth of slab, $D = 200 + 15 + \frac{\phi}{2} = 200 + 15 + 5 = 220$ mm

Load calculation

Dead load

Self weight of slab = 25 × 1 × 0.22 = 5.5 kN/m

DL due to floor finishes = 1 × 1 = 1 kN/m

Live load, LL = 3 × 1 = 3 kN/m

$\therefore$ Total dead load, w_d = 5.5 + 1 = 6.5 kN/m

Total live load, w_l = 3 kN/m

Total load, $w_{total} = w_d + w_l$ = 9.5 kN/m

Total factored load, w_u = 1.5 w = 1.5 × 9.5 = 14.25 kN/m

$$\therefore \text{ Factored/design, } \quad M_u = \frac{w_u\, l_{effx}^2}{8} = \frac{14.25 \times 4.2^2}{8} = 31.42125 \text{ kNm}$$

For Fe415, $M_{u\,lim} = 0.138 f_{ck}\, bd^2$

$\Rightarrow$ $31.4\,2125 \times 10^6 = 0.138\,(20)\,(1000)\,d^2$

$\Rightarrow$ d = 106.7 mm < 200 mm (OK)

Design of flexural reinforcement

$$\frac{A_{st\,x}}{Bd} = \frac{0.5\, f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, M_u}{f_{ck} B d^2}}\right]$$

$$\Rightarrow \quad \frac{A_{st\,x}}{Bd} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 31.42125 \times 10^6}{20 \times 1000 \times 200^2}}\right]$$

$\Rightarrow$ $A_{st\,x}$ = 457.025 mm²

$\because$ 10 mm ϕ bars are usd

$$\therefore \quad A_\phi = \frac{\pi}{4} \times 10^2 = 78.54 \text{ mm}^2$$

$$\therefore \quad S = \frac{1000 \times A_\phi}{A_{st}} = \frac{1000 \times 78.54}{457.025} = 171.85 \text{ mm} \qquad \text{(OK)}$$

Provide 10 mm ϕ dia bars @ 150 mm c/c.

$$\text{Max. spacing} = \text{Min.}\begin{cases} 3d = 600\text{mm} \\ 300 \text{ mm} \end{cases} = 300 \text{ mm}$$

$$A_{st\text{ provided}} = \frac{1000 \times A_\phi}{S} = \frac{1000 \times 78.54}{150} = 523.6 \text{ mm}^2$$

Check for shear

$$\tau_v = \frac{V_u}{Bd}$$

$$V_u = \frac{w_u l_0}{2} = \frac{14.25 \times 4.0}{2} = 28.5 \text{ kN}$$

$$\tau_v = \frac{28.5 \times 10^3}{1000 \times 200} = 0.1425 \text{ MPa}$$

$$p_{t\text{ prov.}} = \frac{A_{st\text{ provided}}}{Bd} \times 100 = \frac{523.6}{1000 \times 200} \times 100 = 0.2618\%$$

For $p_t = 0.2618\%,\ \tau_c = 0.37$ MPa

Thus, $\tau_v < \tau_c$

∴ Slab is safe in shear.

Distribution Reinforcement

Provbide transverse reinforcement @ 0.12%.

$$A_{st\,y} = \frac{0.12}{100} \times 1000 \times 200 = 240 \text{ mm}^2$$

Using 8 mm ϕ bars spacing, $S = \dfrac{1000 \times A_\phi}{A_{st\,y}}$

Where $A_\phi = \dfrac{\pi}{4} \times 8^2 = 50.3 \text{ mm}^2$

∴ $S = \dfrac{1000 \times 50.3}{240} = 209.583$ mm c/c

∴ Provide 8 mm ϕ bars @ 200 mm c/c

$$\text{Max. spacing} = \text{Min.}\begin{cases} 5d \\ 450 \text{ mm} \end{cases}$$

$$= \text{Min.}\begin{cases} 5 \times 200 = 1000 \text{ mm} \\ 450 \text{ mm} \end{cases}$$

$$= 450 \text{ mm c/c} > 200 \text{ mm} \quad \text{(OK)}$$

Check for Bond

For MOR,

$$0.36 f_{ck} x_u b = 0.87 f_y \times A_{st\text{ provided}}$$

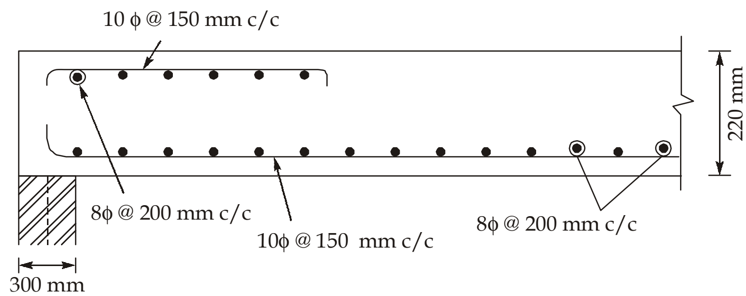
$$x_u = \frac{0.87 \times 415 \times 523.6}{0.36 \times 20 \times 1000} = 26.26 \text{ mm}$$

$$\text{MOR} = M_u = 0.36 f_{ck} x_u b [d - 0.42 x_u] = 35.73 \text{ kNm}$$

$$\frac{M_u}{V} = \frac{35.73\times10^3}{28.5} = 1253.68\,\text{mm}$$

$$L_d = \frac{0.87 f_y \times \phi}{4\tau_{bd}}$$

$$= \frac{10\times0.87\times415}{4\times1.2\times1.6} = 470\text{ mm} < 1253.68\text{ mm} \qquad \text{So OK}$$



Q.8 (b) Solution:

(i)

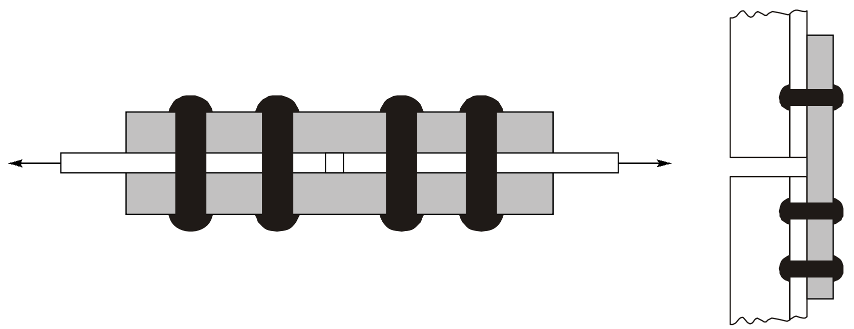
Limit state are the states beyond which the structure no longer satisfies the specified performance requirement. **As per IS 800: 2007, the limit states are generally grouped under.**

1. **Limit State of Strength:** Limit state of strength are associated with failure of structure under the worst combination of loading including appropriate partial factor of safety. The limit state of strength include
   (a) Loss of stability/equilibrium of structure (including the effect of sway) or any of parts including supports and foundation.
   (b) Strength limit (general yielding, formation of mechanism, rupture of structure or any of its parts of components)
   (c) Fatigue and brittle failure.

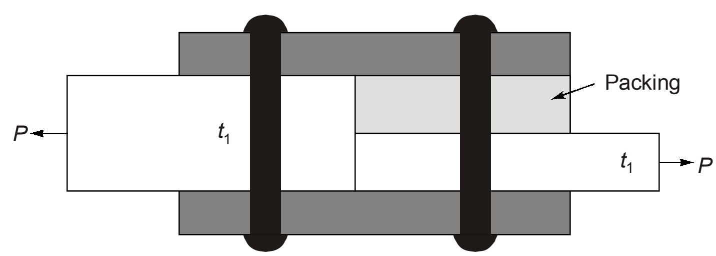
2. **Limit state of serviceability:** There are limit states beyond which specified service criteria are no longer met. These include
   (a) Deformation and deflections
   (b) Vibrations in the structure or any of its component causing discomfort to people or damages to the structure
   (c) Ponding of structures
   (d) Corrosion and durability
   (e) repairable damage due to fatigue

(ii)

A tension member is spliced when the length of section available is less than that of tension member required. Tension splices are also used when size of section change over different segments. The force is then carried through these covers across the joint and is transferred to the other portion of the member through the connection.

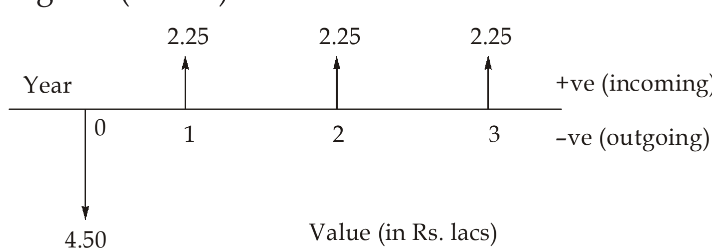


Design: Splices connection for axial tensile force is usually the riveted or bolted connections. As per IS specification, the splice cover and its connection should be designed to develop the net tensile strength of the main member. When member of different thickness are spliced, packing is required to fill the gap. If the difference of thickness between two plates being joined ($t_1 - t_2$) is greater than 6 mm, then additional rivets shall be provided on packing extension. The no. of additional rivets shall be 2.5% of actual no. of rivets obtained from normal calculation per 2 mm thickness of packing.



Q.8 (c) Solution:

The cash flow diagram (in lacs) associated with brand '*A*' is shown below:



The net present worth of brand '*A*' is computed as below:

$$\text{Net present worth} = \text{Present worth of savings} - \text{Present worth of cost}$$

$$= 225000\left(\frac{P}{A}, 8\%, 3\right) - 450000$$

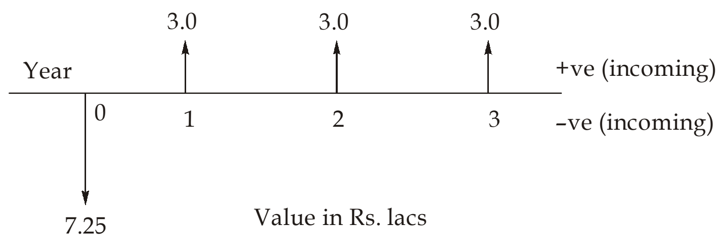
$$\because \quad \left(\frac{P}{A}, r, n\right) = \left[\frac{(1+r)^n - 1}{r(1+r)^n}\right]$$

$$= 225000\left[\frac{(1+0.08)^3 - 1}{0.08(1+0.08)^3}\right] - 450000$$

$$= 225000 \times 2.5771 - 450000$$

$$= 579847.50 - 450000 = \text{Rs } 129847.50$$

The cash flow diagram associated with brand B is shown below:



$$\text{Net present worth} = \text{Present worth of savings} - \text{Present worth of cost}$$

$$= 300000\left(\frac{P}{A}, 8\%, 3\right) - 725000$$

$$= 300000\,(2.5771) - 725000$$

$$= \text{Rs. } 48130.00$$

The net present worth associated with brand A is greater than the net present worth associated with brand B.

So, select brand A.

❍❍❍❍