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ESE 2022

Main Exam Detailed Solutions

Civil Engineering

PAPER-I

EXAM DATE : 26-06-2022 | 9:00 AM to 12:00 PM

MADE EASY has taken due care in making solutions. If you find any discrepency/error/typo or want to contest the solution given by us, kindly send your suggested answer(s) with detailed explanation(s) at: info@madeeasy.in

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ANALYSIS

Civil Engineering Paper-I
ESE 2022 Main Examination

Sl.	Subjects	Marks
1.	Building Materials and Construction	96
2.	Strength of Materials	64
3.	Structural Analysis	92
4.	Steel Structures	84
5.	RCC	84
6.	CTPM and Equipments	60
	Total	**480**

Scroll down for detailed solutions

Section-A

Q.1 (a) (i) Name any six tools for cutting and dressing stones.

(ii) What are the impurities in lime and how do they affect the cementing properties?

[4 + 8 = 12 marks]

Solution:

(i) Stone cutting and dressing tools: The dressing/cutting tools for stones are (i) wedge, (ii) pitching tool (iii) boaster (iv) scabbing hammer (v) mash hammer (vi) separate pick, (vii) punch, (viii) scabbing pick, (ix) crow bar (x) axe punch (xii) dressing knife and (xii) splitting chisel.

(ii) Impurities in lime

(a) Magnesium carbonate: Lime contain Magnesium Carbonate in varying proportions. Presence of this constituent allows the lime to slake and set slowly. The excess of $MgCO_3$ impart hydraulicity even in absence of clay.

The makes the magnesium oxide "hard burned" and therefore slow slaking and less active. This will ultimately affect the binding properties of lime.

(b) Clay: It is responsible for hydraulic properties of lime. It also makes lime insoluble in water. Clay in excess of 10% to 30%, arrests slaking. If it is in small quantity, the slaking is retarded and do not display any hydraulic properties which ultimately do not set and harden under water.

(c) Silica: In its free from (sand). It has considerable effect on the cementitious properties of lime. Lime having high silica content shows poor cementing and hydraulic properties.

(d) Iron compounds: Iron oxides, carbonates or sulphides at lower temperature of calcination converted in Fe_2O_3. But at higher temperature iron combines with lime and silicates and forms complex silicate compounds. Pyrite or iron sulphide is regarded to be highly undesirable.

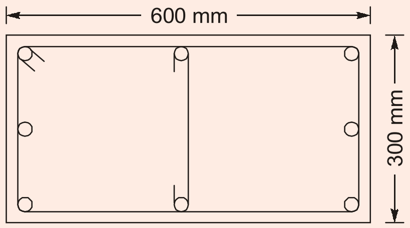
(e) Carbonaceous matter: Its presence in the lime is an indication of poor quality of lime.

(f) Sulphates: They slow down the slaking action and increase the setting rate of lime.

(g) Alkalis: When pure lime is required, the alkalis are undesirable.

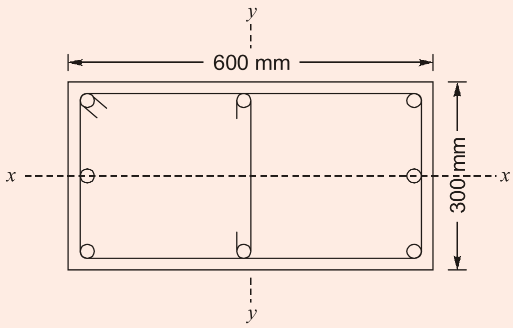
End of Solution

Q.1 (b) A short column of 3 m effective height is subjected to an ultimate load of 1400 kN. Cross section of the column is 300 × 600 mm. The reinforced concrete column consists of 8 longitudinal reinforcing bars of 20 mm diameter as shown. M25 grade concrete and Fe500 grade are used. Obtain its ultimate moment carrying capacity only about its major axis. Effective cover to the longitudinal reinforcing bars is 60 mm.



[12 marks]

Solution:



$$A_{sc} = 8 \times \frac{\pi}{4} \times 20^2 = 2513.27 \text{ mm}^2$$

∵ Moment of inertia of column section is maximum about y-y axis and thus y-y axis is the major axis of column section.

$$\therefore \quad I_{yy} = \frac{300 \times 600^3}{12} = 54 \times 10^8 \text{ mm}^4$$

Effective cover, $d' = 60$ mm

Ultimate load, $P_u = 1400$ kN

$$\therefore \quad \frac{P_u}{f_{ck} bD} = \frac{1400 \times 10^3}{25 \times 300 \times 600} = 0.311$$

$$\frac{d'}{D} = \frac{60}{600} = 0.1$$

Percentage reinforcement, $p = \frac{A_{sc}}{bD} \times 100 = \frac{2513.27}{300 \times 600} \times 100 = 1.396\%$

$$\therefore \quad \frac{p}{f_{ck}} = \frac{1.396}{25} = 0.05584 \simeq 0.056 \text{ (say)}$$

$\therefore$ Using chart 48, for $\dfrac{P_u}{f_{ck}bD} = 0.311$ and $\dfrac{p}{f_{ck}} = 0.056$

$$\frac{M_u}{f_{ck}bD^2} = 0.10$$

$\Rightarrow$ $M_u = 0.10 \times 25 \times 300 \times 600^2$ Nmm $= 270$ kNm

$\therefore$ Ultimate moment carrying capacity about major axis is 270 kNm.

End of Solution

Q.1 (c) A simply supported post-tensioned concrete beam of 10 m span, 230 mm wide and 400 mm deep is prestressed with a straight cable, having cross sectional area 385 mm^2, located at 60 mm from the soffit of the beam subjected to an initial stress of 1200 N/mm^2. Using M50 grade concrete and the following data, estimate the percentage loss of prestress.

$E_c = 5000\sqrt{f_{ck}}$

Modulus of Elasticity of steel = 2.1×10^5 N/mm^2

Relaxation of stress in steel = 4.5%

Shrinkage strain of concrete = 0.0003

Creep coefficient of concrete = 1.6

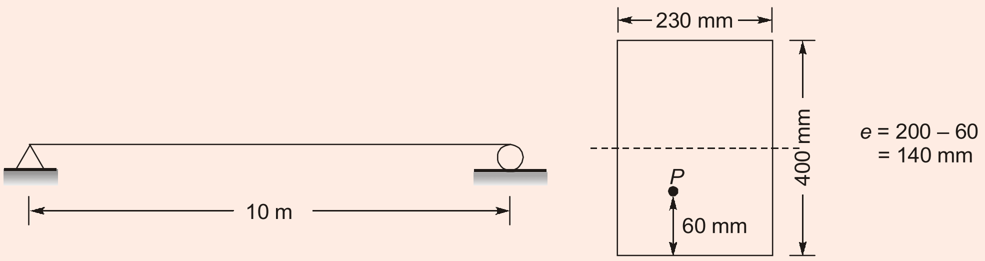
Friction coefficient for wave effect = 0.0025 per metre

Slip at anchorage = 2 mm

Loss of prestress due to friction = $p_0(\mu\alpha + kx)$

[12 marks]

Solution:



Cross-sectional area of cable = 385 mm^2

Initial stresses = 1200 N/mm^2

Tension in cable [P] = $1200 \times 385 \times 10^{-3}$ kN = 462 kN

$$E_c = 5000\sqrt{50} = 35355.34 \text{ MPa}$$

Stress in concrete at the level of steel

$$f_c = \frac{P}{A} + \frac{P.e}{I} \times e$$

$$= \frac{462 \times 10^3}{230 \times 400} + \frac{462 \times 10^3 \times 140}{\frac{230 \times 400^3}{12}} \times 140$$

$$= 5.02 + 7.38 = 12.4 \text{ N/mm}^2$$

Loss of stress in post-tensioned beam,

(a) Loss of stress due to elastic shortening of concrete will be zero because there is only one cable.

In case of postensioned member if there is only one tendon, there is no losss because the applied prestress is recorded after the elastic shortening of member.

(b) Loss of stress due to creep of concrete:

$$= \theta . m . f_c$$

$$= 1.6 \times \frac{E_s}{E_c} \times f_c$$

$$= 1.6 \times \frac{2.1 \times 10^5}{35355.34} \times 12.4 = 117.878 \text{ N/mm}^2$$

(c) Loss due to shrinkage of concrete: $\in_c \times E_s$

$$= 0.0003 \times 2.1 \times 10^5 = 63 \text{ N/mm}^2$$

(d) Loss of stress due to anchorage slip:

$$= \frac{\Delta L}{L} E_s$$

$$= \frac{2}{10{,}000} \times 2.1 \times 10^5 = 42 \text{ N/mm}^2$$

(e) Loss of stress due to Relaxation in steel:

$$= \frac{4.5}{100} \times 1200 = 54 \text{ N/mm}^2$$

(f) Loss due to friction:

$$= p_0[\mu\alpha + k.x]$$

For straight cable, $\alpha = 0$

Let us assume that the cable anchored from one end only,

$\therefore$ $\Delta f = 1200[\mu \times 0 + 0.0025 \times 10] = 30 \text{ N/mm}^2$

Let us assume that the cable anchored on both side,

$\therefore$ $\Delta f = 1200[\mu \times 0 + 0.0025 \times 10] = 15 \text{ N/mm}^2$

$\Rightarrow$ Total loss of stress when cable in anchored on one side:

$117.878 + 63 + 42 + 54 + 30 = 306.878 \text{ N/mm}^2$

$$\%\text{age loss} = \frac{306.878}{1200} \times 100 = 25.5\%$$

Total loss of stress when cable is anchored one both side

$$= 306.878 - 30 + 15 = 291.878 \text{ N/mm}^2$$

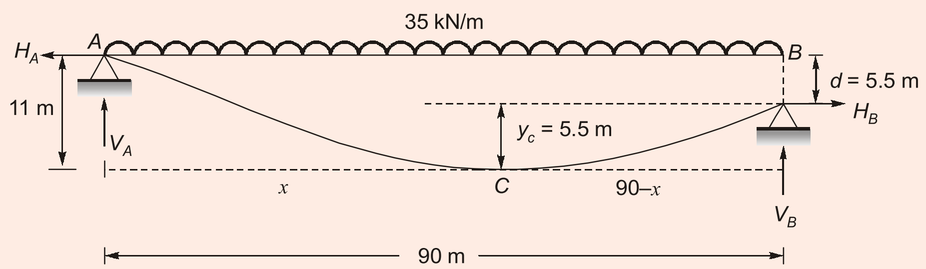
$$\text{Percentage loss} = \frac{291.878}{1200} \times 100 = 24.3\%$$

End of Solution

Q.1 (d) A cable is suspended from the points (1) and (2) which are 90 m apart horizontally and are at different levels, the point (1) being 5.50 m vertically higher than the point (2) and the lowest point in the cable is 11 m below point (1). The cable is subjected to a UDL of 35 kN/m over the horizontal span. Determine the horizontal and vertical reactions at each end and also the maximum tension in the cable.

[12 marks]

Solution:



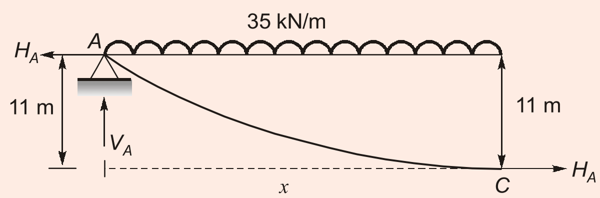
$$\Sigma M_B = 0$$

$$V_A \times 90 - H_A \times 5.5 - 35 \times 90 \times \frac{90}{2} = 0$$

$$90\, V_A - 5.5\, H_A = 35 \times 90 \times 45 \quad \text{...(i)}$$

BM at C = 0

Location of lowest point (C)



$$\Sigma F_y = 0$$

$$V_A - 35x = 0$$

$$V_A = 35x \quad \text{...(ii)}$$

$$\Sigma M_A = 0$$

$$H_A \times 11 + 35\frac{x^2}{2} = 0$$

$$H_A = 1.591\, x^2 \quad \text{...(iii)}$$

Put values from (ii) and (iii) into (i), we get

$90 \times 35x - 5.5 \times 1.591\, x^2 = 35 \times 90 \times 45$

$8.751\,x^2 - 3150x + 141750 = 0$

$x = 52.72$ m

$$\therefore \quad V_A = 35 \times 52.72 = 1845.2 \text{ kN}$$

$$H_A = 1.591 \times 52.72^2 = 4422 \text{ kN}$$

Now,

$$V_B = 35 \times 90 - V_A = 35 \times 90 - 1845.2 = 1304.8 \text{ kN}$$

$$H_A = H_B = 4422 \text{ kN}$$

Maximum tension will be at support A

$$T_{max} = \sqrt{V_A^2 + H_A^2} = \sqrt{(1845.2)^2 + (4422)^2} = 4791.54 \text{ kN}$$

End of Solution

Q.1 (e) It is quite common to replace/partially substitute, cement with flyash. However, all the concretes made by cement substituted by flyash may not be used for all the applications. Specify some applications where using flyash concrete is useful and the applications where we should avoid using flyash products and provide reasons.

[12 marks]

Solution:

- Flyash is pozzolanic material obtained by burning the pulverized coal.
- Replacement of cement with flyash modifies property of cement and concrete and entire cementaneous material is combination of cement clinker and pozzolanic material like flyash.
- Addition of flyash by reduction in amount of basic cement clinker results in reduction in rate and amount of heat of hydration.
- Lesser rate of hydration maintain workability of concrete for longer time, i.e., concrete will remain with it's internal energy to achieve a higher degree of compaction.

  So, best use of replacement of cement with flyash found for self compacted concrete, where water powder ratio kept to be 0.3 with high powder content.
- Flyash does not leach out portlandite while formation of C-S-H gel, so finished structure will not become porous. Moreover, less amount and rate of heat evolution does not cause shrinkage. Hence, overall durability of structure is improved e.g. Mass concreting.
- Due to less amount of C_3A clinker, sulphate resistance can also be provided. Hence, cement with flyash can be used against sulphur attack as well.
- Replacement with flyash reduces the cement demand, which helps in reducing emission of CO_2 (GHG gases) in atmosphere. Hence, concrete prepared by flyash is a green concrete as well.
- But as amount of C_3A clinker is less, the rate of hydration is less. Hence, it can not be use against quick setting like for underwater concreting, grounting, shortcrete etc.

- Slow rate of hydration will not impart strength gaining construction like precast construction, pavement repair, cold weather construction etc.
- Still replacement of flyash is common as it gives better workability to mix, high durability to finished structure economical and environment friendly.

End of Solution

Q.2 (a) The floor of a hall is 9 m × 12 m to the centre of supports. The beams are spaced at 3 m centre to centre. Thickness of the floor slab is 150 mm. Live load on the floor is 3 kN/m^2. Load due to floor finish and plaster being 1.5 kN/m^2, design one intermediate T-beam for flexure and shear as per Limit State Design. Sketch the reinforcement details. Density of concrete is 25 kN/m^3. Assume an effective cover to the main reinforcement is 50 mm. Width of rib = 300 mm.

Effective width of Flange $b_f = \dfrac{l_0}{6} + b_w + 6D_f$

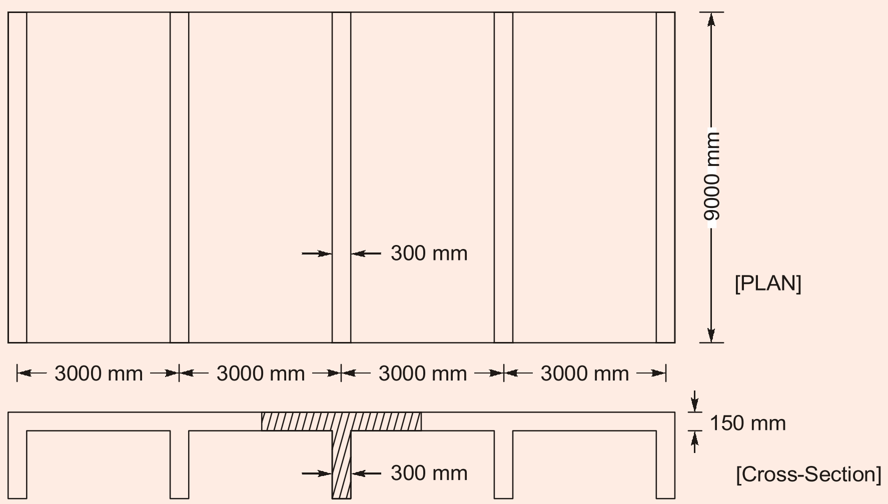
For $Fe500, \left(\dfrac{x_{u,max}}{d}\right) = 0.46$

Strength of shear reinforcement $V_{us} = \dfrac{0.87 f_y A_{sv} d}{S_v}$

$\dfrac{100A_s}{bd}$	Design shear strength of concrete τ_c N/mm^2
0.75	0.57
1.00	0.64
1.25	0.70
1.50	0.74

[20 marks]

Solution:



Dead load of slab $= 0.15 \times 25 = 3.75$ kN/m^2

Live load on floor $= 3$ kN/m^2

Load due to floor finish and plaster $= 1.5$ kN/m^2

Total load $= 8.25$ kN/m^2

Load per m run of beam $=$ Load on slab per unit area $\times$ c/c distance between beam

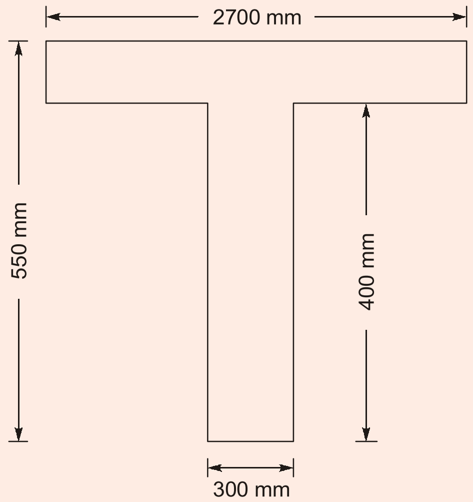
$= 8.25 \times 3 = 24.75$ kN/m

Effective width of flange, $b_f = \dfrac{l_0}{6} + b_w + 6D_f = \dfrac{9000}{6} + 300 + 6 \times 150 = 2700$ mm

Let us adopt overall depth D of beam equal to 550 mm

Effective cover $= 50$ mm

$\therefore$ Effective depth, $d = 550 - 50 = 500$ mm



Dead load of web of beam $=$ Width of web $\times$ Depth of web $\times$ Concrete density

$= 0.3 \times 0.40 \times 25 = 3$ kN/m

Total load on beam per meter run $= 24.75 + 3 = 27.75$ kN/m

$$\text{Factored (BM)}_u = \frac{w_u \times l^2}{8} = \frac{1.5 \times 27.75 \times 9^2}{8} = 421.45 \text{ kNm}$$

Assume grade of concrete is to be M25, as the table given for design shear strength of concrete is of M25.

Let us assume that N.A. lies in flange, that is

$$x = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times b_f} = \frac{0.87 \times 500 \times A_{st}}{0.36 \times 25 \times 2700}$$

$$x = 0.0179\, A_{st} \qquad \text{...(i)}$$

Now, $(\text{BM})_u = 0.87 \times f_y \times A_{st}\, [d - 0.472\, x_u]$

$421.45 \times 10^6 = 0.87 \times 500 \times A_{st}\, [500 - 0.42 \times 0.0179\, A_{st}]$

$$421.45 \times 10^6 = 217500\, A_{st} - 3.27\, A_{st}^2$$

$$A_{st} = 1997.7 \text{ mm}^2$$

Now, from eq. (i), $x_u = 35.75$ mm

$\therefore$ N.A. lies in the flange.

Let's provide 2 bars of 28 mmϕ and 2 bars of 25 mmϕ.

$$\text{Minimum area of steel} = \frac{0.85 \times b_w d}{100} = \frac{0.85}{100} \times 300 \times 500 = 1275 \text{ mm}^2 \qquad \text{(OK)}$$

Check for shear:

Factored shear force, $$V_u = \frac{w_u \times L}{2} = \frac{1.5 \times 27.75 \times 9}{2} = 187.31 \text{ kN}$$

Factor 1 shear stress, $$\tau_v = \frac{V_u}{b_w d} = \frac{187.31 \times 10^3}{300 \times 500} = 1.248 \text{ N/mm}^2$$

Maximum shear stress for M25:

$$\tau_{max} = 0.625\sqrt{f_{ck}} = 0.625\sqrt{25} = 3.125 \text{ N/mm}^2$$

$$\tau_u < \tau_{max} \qquad \text{(OK)}$$

Now, percentage tensile reinforcement

$$p_t = \frac{A_{st}}{b_w d} \times 100 = \frac{1997.7}{300 \times 500} \times 100 = 1.3318\%$$

From the table given in the question,

For, $$\frac{100 A_s}{bd} = 1.3318$$

$$\tau_c = 0.7 + \left(\frac{0.74 - 0.7}{1.5 - 1.25}\right)(1.3318 - 1.25) = 0.713 \text{ N/mm}^2$$

$\because$ $$\tau_v > \tau_c$$

Now, shear reinforcement is designed for shear force of

$$V_{us} = V_u - \tau_c\, bd$$

$$= 187.31 - 0.713 \times 300 \times 500 \times 10^{-3}$$

$$= 80.36 \text{ kN}$$

Using 2-legged 8 mm diameter stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.48 \text{ mm}^2$$

Spacing of shear reinforcement,

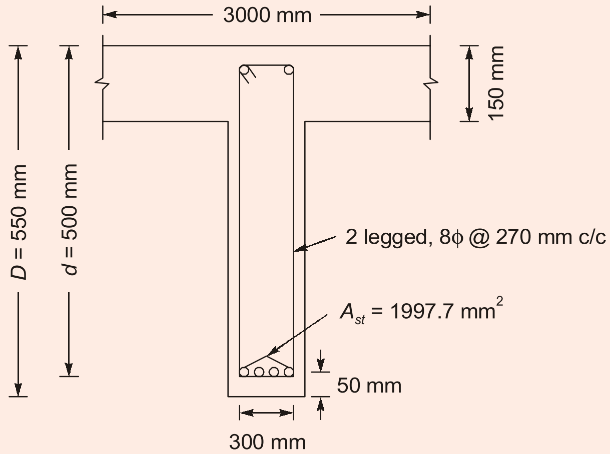
$$S_v = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$$

$$S_v = \frac{0.87 \times 500 \times 100.48 \times 500}{80.36 \times 10^3} = 271.95 \text{ mm}$$

Thus, provide 2-legged 8 mm diameter stirrups @ 270 mm c/c.

Check for spacing:

$$S_v \leq \begin{cases} 0.75d = 0.75 \times 500 = 375 \text{ mm} \\ 300 \text{ mm} \end{cases} \qquad \text{(OK)}$$



End of Solution

Q.2 (b) Determine the proportion of aggregates required to give a suitable grading based on their fineness moduli. Assume the following gradings of aggregates:

IS sieve designation (mm)	Aggregate size associated with percentage passing IS sieve (mm)		
	40	20	10
40	90	—	—
20	10	90	—
10	0.2	10	90
4.75	—	0.3	20

How, fineness modulus affect the characteristics of concrete? What is the optimum value of FM for fine aggregates?

[20 marks]

Solution:

IS sieve designation (mm)	Aggregate size associated with percentage passing IS sieve (mm)		
	40	20	10
40	90	—	—
20	10	90	—
10	0.2	10	90
4.75	—	0.3	20

The data provided for the percentage passing from 20 mm size sieve is inconsistent.
In accordance with IS 383-2016 of 40 mm maximum size aggregate:

IS sieve designation (mm)	Desired grading for 40 mm maximum size aggregate as IS 383 : 2016
40	90 to 100
20	30 to 70
10	10 to 35
4.75	0 to 5

The fractions coarse aggregates are combined to get a combined grading.
Start with trial fraction of 4:87:9, check whether the combined grading is obtained.

IS sieve designation (mm)	Fraction I 40 mm 4%	Fraction II 20 mm 87%	Fraction III 10 mm 9%	Combined grading
40	90	100	—	90.60
20	10	90	—	78.70
10	2	10	90	16.88
4.75	—	3	20	4.41

From the table it is observed that the with the of 4:87:9, the desired combined grading as per IS 838:2016 is obtained.

(i) The significance of fineness modulus [FM] is in specifying the proportions of fine and coarse aggregates when designing the concrete mixes.

(ii) higher value of FM indicates that the aggregates are coarser.

(iii) Lower value of FM results in more paste which helps in making the concrete easier to finish.

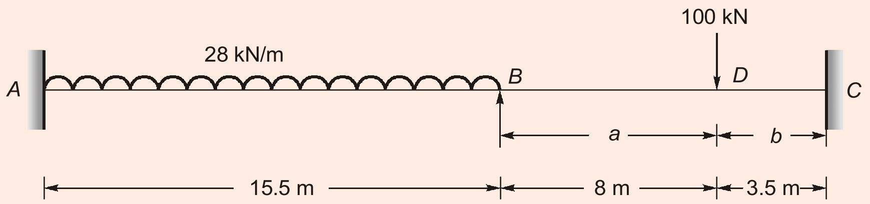
(iv) It helps in proper gradation of aggregates.

End of Solution

Q.2 (c) A continuous beam *ABC* consists of two spans *AB* and *BC* of length 15.50 and 11.50 m respectively. The span *AB* carries a UDL of 28 kN/m and span *BC* carries a point load of 100 kN at 8 m from support *B*. EI is constant for both the span. The end supports *A* and *C* are fixed, while support *B* is simply supported. Determine the support moments and reactions. And also draw the shear force and bending moment diagrams.

[20 marks]

Solution:



First calculating fixed end moment

$$M_{FAB} = -\frac{wl^2}{12} = -\frac{28\times15.5^2}{12} = -560.58 \text{ kNm}$$

$$M_{FBA} = \frac{wl^2}{12} = \frac{28\times15.5^2}{12} = 560.58 \text{ kNm}$$

$$M_{FBC} = -\frac{wab^2}{l^2} = -\frac{100\times8\times3.5^2}{11.5^2} = -74.102 \text{ kNm}$$

$$M_{FCB} = \frac{wab^2}{l^2} = \frac{100\times3.5\times8^2}{11.5^2} = 169.376 \text{ kNm}$$

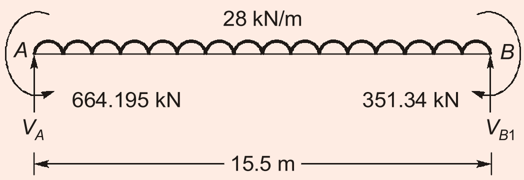
Now distribution factor for point *B*.

Joint	Member	Stiffness	Total Stiffness	D.F.
B	*BA*	$\frac{4EI}{15.5}$	$\frac{432EI}{713}$	0.4260
	BC	$\frac{4EI}{11.5}$		0.57

By using moment distribution method.

	A	B		C
D.F.		0.4260	0.5740	
F.E.M.	–560.58	560.58	–74.102	169.376
B.M.		–207.23	–279.24	
C.O.M	–103.615			–139.62
Final Moment	–664.195	353.35	351.34	29.756

Taking individual spans *AB* and *BC*.



$\Rightarrow$ $\quad V_A + V_B = 434 \text{ kN}$ ...(i)

Taking $\quad \Sigma M_B = 0$

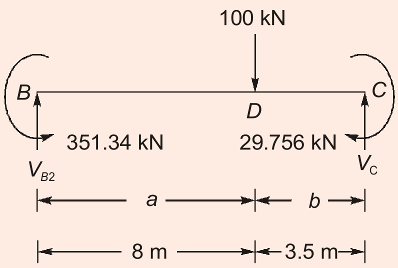
$$V_A \times 15.5 - 664.195 - 28 \times 15.5 \times \frac{15.5}{2} + 351.34 = 0$$

$$V_A = 237.184 \text{ kN}$$

Putting value in eq. (i)

$$V_{B_1} = 196.81 \text{ kN}$$

Similarly for span *BC*.



$\Rightarrow$ $\quad V_{B_2} + V_C = 100 \text{ kN}$ ...(ii)

Taking $\quad \Sigma M_C = 0$

$\Rightarrow$ $\quad V_{B2} \times 11.5 - 100 \times 3.5 - 351.34 + 29.756 = 0$

$\Rightarrow$ $\quad V_{B2} = 58.40 \text{ kN}$

Putting value in eq. (ii)

$$V_C = 41.60 \text{ kN}$$

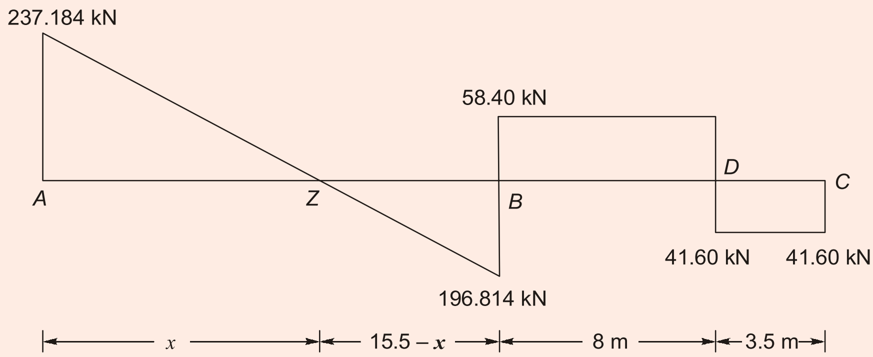
Hence, $\quad V_A = 237.184 \text{ kN}$

$$V_B = V_{B_1} + V_{B_2}$$

$$V_B = 196.81 + 58.40 = 255.21 \text{ kN}$$

$$V_C = 41.60 \text{ kN}$$

SFD

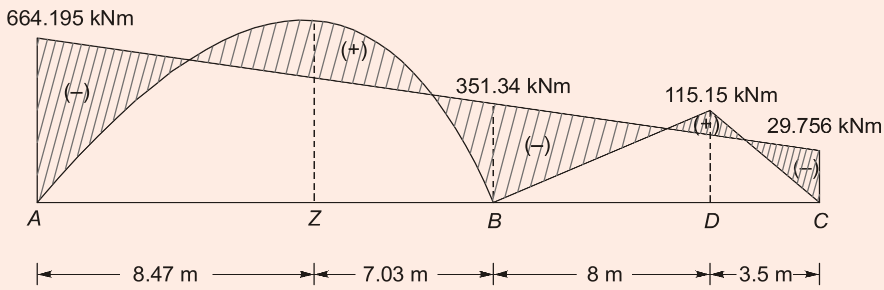


Point Z can be found out by similar triangle property

Then $$\frac{x}{15.5-x} = \frac{237.184}{196.80}$$

$$196.80x = 3676.35 - 237.184\,m$$

$\Rightarrow$ $$x = 8.47$$



End of Solution

Q.3 (a) (i) Classify the bricks based on the physical and mechanical properties. Indicate the properties of first class bricks. Which are the applications where first class bricks are highly recommended and other classes of bricks should not be used?

(ii) Discuss the scale of sampling for physical characteristics of bricks. Indicate its importance.

[10 + 10 = 20 marks]

Solution:

(i) Clay bricks are classified as first class, second class, third class and fourth class based on their physical and mechanical properties:

First class bricks:

1. These are thoroughly burnt and are of deep reed, cherry or copper colour.
2. The surface should be smooth and rectangular , with parallel, sharp and straight edges and square corners.
3. These should be free from flaws, cracks and stones.
4. These should have uniform texture.
5. No impression should be left on the brick when a scratch is made by a finger nail.
6. The fractured surface of the brick should not show lumps of lime.
7. A metallic or ringing should come when two bricks are struck against each other.
8. Water absorption should not be more than 15% by its dry weight of compressive strength of brick is ≥ 12.5 N/m^2, it should not be more than 20% by its dry weight if compressive strength is < 12.5 N/mm^2.

Use first class brick recommended for pointing, exposed facework in masonry structure, flooring and reinforced brick work.

(ii) About fifty piece of bricks are taken at random from different parts of the stack to perform various tests. For the purpose of sampling , a lot should contain maximum of 50,000 bricks. The number of bricks selected for forming a sample (IS : 5454). The scale of sampling for physical characteristics is

Scale of sampling and permissible number of defectives for visual and dimensional characteristics

No. of bricks in the lot	For characteristics specified for individual brick		For dimensional characteristics specified for group of 20 bricks-No. of bricks to be selected
	No. of bricks to be selected	Permissible No. of defectives in the sample	
2001 to 10000	20	1	40
10001 to 35000	32	2	60
35001 to 50000	50	3	80

Scale of sampling for physical characteristics

Lot size	Sampling size for compressive strength, breaking load, transverse strength, bulk density, water absorption and efflorescence	Permissible No. of defectives for eflorescence	Warpage	
			Sample size	Permissible No. of defectives
2001 to 10000	5	0	10	0
10001 to 35000	10	0	20	1
35001 to 50000	15	1	30	2

- If lot contains 2000 or less bricks, the sampling shall be subjected to agreement between purchaser and supplier.

End of Solution

Q.3 (b) A prestressed concrete beam, 300 × 800 mm, is simply supported over an effective span of 12 m. It is prestressed with a cable whose eccentricity varies linearly from 50 mm at the supports to 250 mm at $\left(\frac{1}{3}\right)^{rd}$ span points from either support. Effective prestress in the cable is 900 kN. Determine,

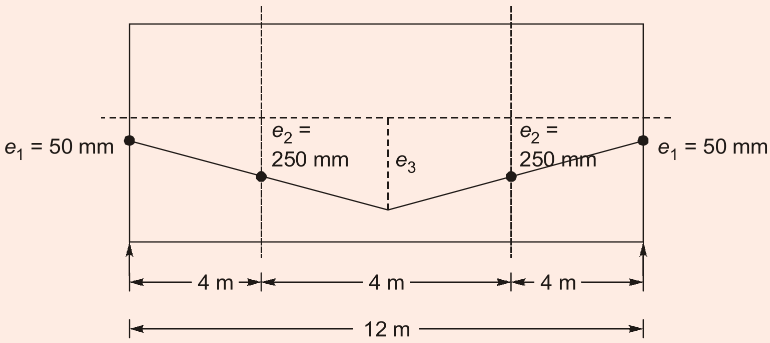
(i) Net deflection due to prestress and self weight.

(ii) Central concentrated load required to caused a maximum downward deflection of $\left(\frac{\text{Span}}{400}\right)$.

Use M40 grade concrete. Derive only the expressions for deflections due to prestress. $E_c = 5000\sqrt{f_{ck}}$. Density of concrete = 25 kN/m^3.

[20 marks]

Solution:



Given:

$$b = 300 \text{ mm}$$
$$d = 800 \text{ mm}$$
$$l_{eff} = 12 \text{ m}$$
$$e_1 = 50 \text{ mm (at support)}$$
e_2 = 250 mm (at middle third, varying linearly, so cable is straight cable)

$$E_c = 5000\times\sqrt{f_{ck}} = 5000\times\sqrt{40}$$

$$E_cI_c = 5000\times\sqrt{40}\times\frac{300\times800^3}{12} = 4.0477\times10^{14}$$

1. Net deflection due to prestress and self weight.

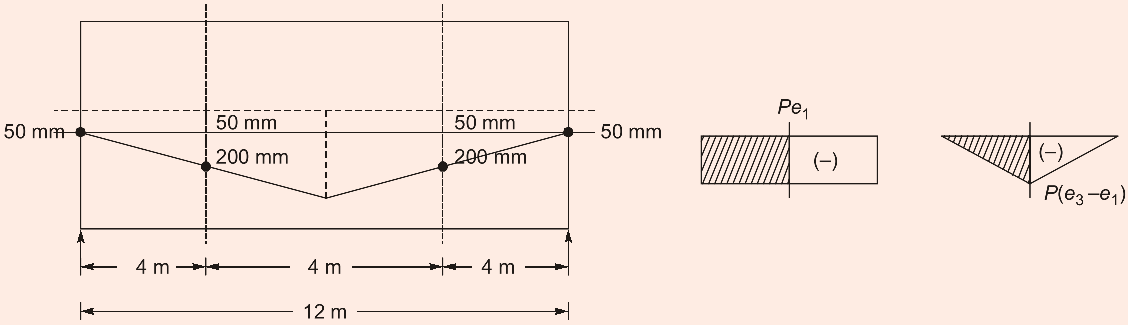
(a) Deflection due to self weight

$$DL = 0.3\times0.8\times1.0\times25 = 6.0 \text{ kN/m} = 6.0 \text{ N/mm}$$

$$\delta_1 = \frac{5}{384} \times \frac{wL^4}{E_c I_c} = \frac{5}{384} \times \frac{6.0 \times 12000^4}{4.0477 \times 10^{14}}$$

$$= 4.002 \text{ mm} \downarrow \text{downward}$$

(b) Deflection due to prestressing force



$$\text{Eccentricity at 4 m} = 250 \text{ mm}$$

$$\text{Eccentricity at end} = 50 \text{ mm}$$

$$\text{Eccentricity at mid span} = 50 + \frac{250-50}{4} \times 6 = 350 \text{ mm}$$

Deflection at centre due to prestressing cable (upward)

$$= (-)\left(\frac{Pe_1L^2}{8E_cI_c} + \frac{P(e_3 - e_1)L^2}{12E_cI_c}\right)$$

$$= (-)\left(\frac{900 \times 10^3 \times 50 \times 12000^2}{8 \times 4.0477 \times 10^{14}} + \frac{900 \times 10^3 \times (350-50) \times 12000^2}{12 \times 4.0477 \times 10^{14}}\right)$$

$$= (-)\ (2.001 + 8.004) = (-)\ 10.005 \text{ mm}$$

Net deflection at centre

$$= 4.002 - 10.005 = -6.003 \text{ mm}$$

$$\text{say} = -6.0 \text{ mm}$$

(ii) Central concentrated load required for maximum deflection of $\left(\frac{\text{span}}{400}\right)$

$$\text{Deflection required} = \frac{\text{span}}{400} = \frac{12000}{400} = 30 \text{ mm}$$

$$\text{Deflection due to point load} = \frac{WL^3}{48EI}$$

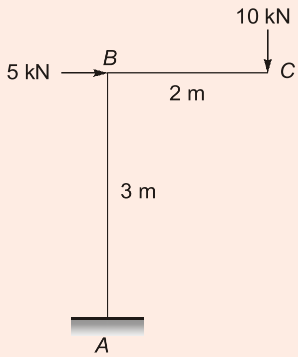
$$\frac{WL^3}{48EI} - 6.003 = 30 \text{ mm}$$

$$\frac{W \times 12000^3}{48 \times 4.0477 \times 10^{14}} = 36.003$$

$$W = 404803 \text{ N} = 404.803 \text{ kN}$$

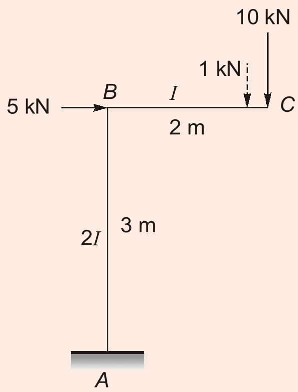
End of Solution

Q.3 (c) Determine the vertical and horizontal deflections at the free end of the frame shown in figure. Support A is fixed, B is rigid joint and point C is free. Moment of inertia of BC = I and BA = 2I, E = 210 GPa and I = 300 × 10^{-6} m^4. Use Load Method.



[20 marks]

Solution:



Vertical deflection at free end C.

$$E = 2.1 \times 10^5 \text{ N/mm}^2$$

$$= 2.1 \times 10^5 \times \frac{10^{-3} \text{ kN}}{10^{-6} \text{ m}^2} = 2.1 \times 10^8 \text{ kN/m}^2$$

$$I = 300 \times 10^{-6} \text{ m}^4$$

So, $EI = 630 \times 10^2$ kNm^2

By unit load method,

Portion	CB	BA
Origin	C	B
Limit	0 to 2	0 to 3
M	$-10x$	$-(20+5x)$
m_1	$-x$	-2
MOI	I	$2I$

Here, m_1 is moment when unit load is applied in direction of deflection required as shown in figure.

$$\text{Deflection }(\delta_v) = \Sigma\int\frac{Mm_1dx}{EI} = \int_0^2\frac{-10x(-x)dx}{EI} + \int_0^3\frac{(20+5x)2dx}{2EI}$$

$$= \frac{26.67+82.5}{630\times10^2} = \frac{109.17}{630\times10^2}$$

$$= 1.732\times10^{-3}\text{ m} = 1.732\text{ mm}$$

Horizontal deflection at point C,

Portion	CB	BA
Origin	C	B
Limit	0–2	0–3
M	$-10x$	$-(20+5x)$
m_1	0	$-x$
MOI	I	$2I$

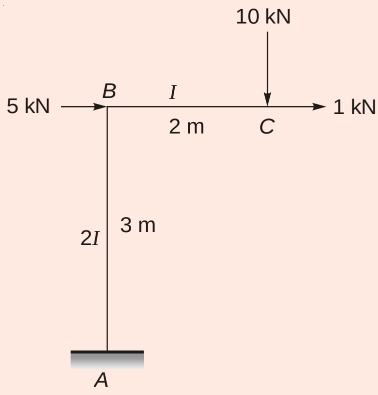
$$(\delta_H) = \Sigma\int\frac{Mm_2dx}{EI} + \int_0^3\frac{-(20+5x)x - xdx}{2EI}$$

$$= \frac{1}{2EI}\int_0^3(20+5x)xdx$$

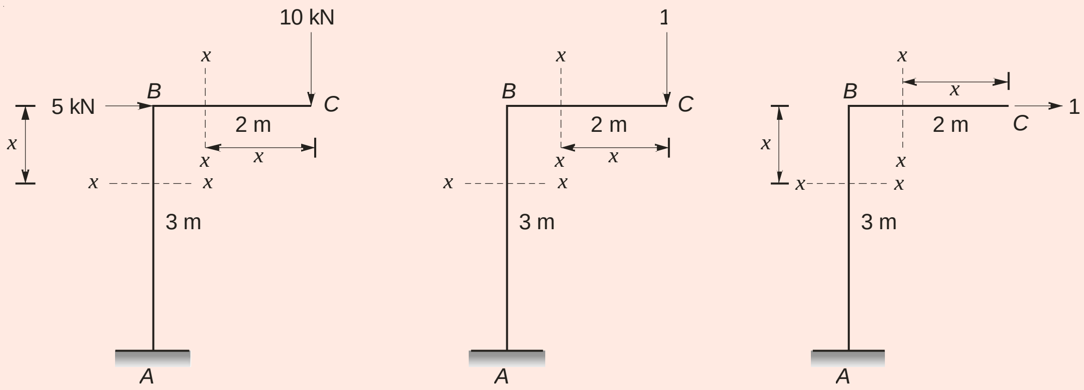
$$= \frac{135}{2EI} = \frac{67.5}{630\times10^2}$$

$$= 1.07\times10^{-3}\text{ m} = 1.07\text{ mm}$$

So, horizontal deflection = 1.07 mm



Alternate Solution:



Member	M_x	m_1	m_2	$\delta_{V_c} = \int \frac{M_x m_1 dx}{EI}$	$\delta_{H_c} = \int \frac{M_x m_2 dx}{EI}$
CB	$-10x$	$-1x$	0	$\int_0^2 \frac{-10x(-x)dx}{EI} = \frac{10\times 2^3}{3EI} = \frac{80}{3EI}$	0
BA	$-10\times 2 - 5x$ $= -20 - 5x$	-1×2	$-1\times x$	$\int_0^3 \frac{(-20-5x)(-2)dx}{E(2I)}$ $= \frac{+120 + \frac{10\times 3^2}{2}}{2EI} = \frac{165}{2EI}$	$\int_0^3 \frac{(-20-5x)(-x)dx}{E(2I)}$ $= \frac{\frac{20\times 3^2}{2} + \frac{5\times 3^3}{3}}{2EI} = \frac{135}{2EI}$
				$= \frac{109.16}{EI}$	$= \frac{67.5}{EI}$

$$E = 210 \times 10^6 \text{ kN/m}^2$$

$$I = 300 \times 10^{-6}\ \text{m}^4$$

$$EI = 630 \times 10^2\ \text{kNm}^2$$

$$\delta_{Vc} = \frac{109.16}{630 \times 10^2} = 1.73\ \text{mm}$$

$$\delta_{Hc} = \frac{67.5}{630 \times 10^2} = 1.07\ \text{mm}$$

End of Solution

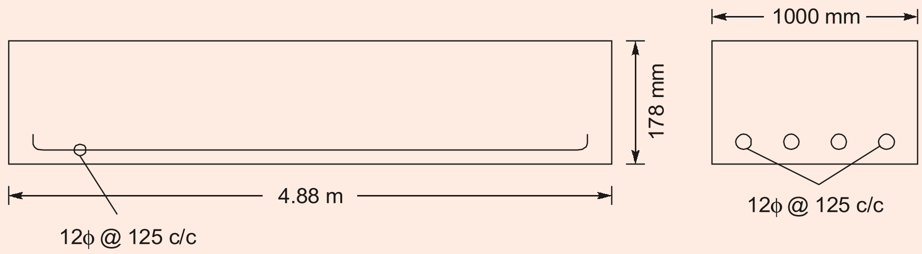
Q.4 (a) A simply supported one way slab 178 mm thick having an effective span of 4.88 m is reinforced with 12 mm diameter rebars at 125 mm centre to centre. The nominal concrete cover to the main reinforcement is 20 mm. The slab is subjected to a live load of 4 kN/m^2 and surface finish of 1.2 kN/m^2. Use M25 concrete and Fe500 grade steel. Compute only the short-term deflection and deflection due to shrinkage. Shrinkage strain is 0.0003. Density of concrete is 25 kN/m^3.

$$E_c = 5000\sqrt{f_{ck}}\ ,\ E_s = 2 \times 10^5\ \text{MPa}$$

$$P_t = \frac{100A_{st}}{bd}\ ;\ P_c = \frac{100A_{sc}}{bd}$$

[20 marks]

Solution:



$$\text{Nominal cover} = 20\ \text{mm}$$

$$\therefore \quad \text{Effective cover} = 20 + \frac{12}{2} = 26\ \text{mm}$$

$$\text{Live load} = 4\ \text{kN/m}^2$$

$$\text{Surface finish} = 1.2\ \text{kN/m}^2$$

$$\text{Dead load of slab} = 25 \times 0.178 = 4.45\ \text{kN/m}^2$$

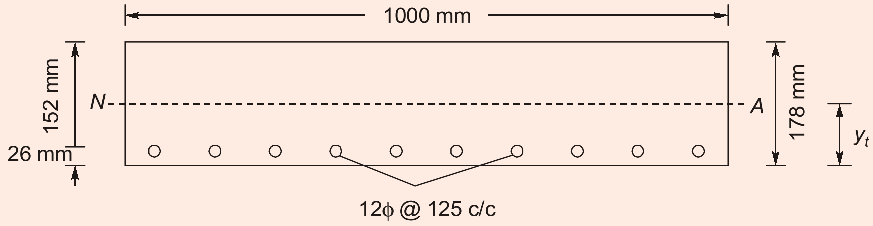
$\therefore$ Total load, $w = 9.65\ \text{kN/m}^2$

Short term deflection is given by,

$$\delta = \frac{5}{384}\frac{wl^4}{EI}$$

where $E = E_c = 5000\sqrt{f_{ck}} = 5000\sqrt{25} = 25000\ \text{N/mm}^2$

$$I = I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \times \frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$



Uncracked section

$$y_t = \frac{178}{2} = 89 \text{ mm}$$

$$I_{gr} = \frac{100 \times 178^3}{12} = 4.7 \times 10^8 \text{ mm}^4$$

For cracked section, $f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{25} = 3.5 \text{ N/mm}^2$

$$M_r = \frac{f_{cr} I_{gr}}{y_t} = \frac{3.5 \times 4.7 \times 10^8}{89} \text{ Nm} = 18.48 \text{ kNm}$$

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

$$m = \frac{E_s}{E_c} = \frac{2 \times 10^5}{25000} = 8$$

Taking moments about NA,

$$\frac{bx^2}{2} = mA_{st}(d - x)$$

$$\Rightarrow \quad 1000\frac{x^2}{2} = 8 \times \left(\frac{1000 \times \frac{\pi}{4} \times 12^2}{125}\right)(152 - x)$$

$$\Rightarrow \quad 500x^2 = 7238.23\,(152 - x)$$

$$\Rightarrow \quad x^2 + 14.476x - 2200.42 = 0$$

$$\therefore \quad x = 40.225 \text{ mm}$$

$\therefore$ Lever arm, $z = d - \frac{x}{3} = 152 - \frac{40.225}{3} = 138.59 \text{ mm}$

$$\therefore \quad I_{cr} = \frac{1000 \times 40.25^3}{3} + 8 \times \left(\frac{1000 \times \frac{\pi}{4} \times 12^2}{125} \right) (152 - 40.225)^2$$

$$= 112167748.7 \text{ mm}^4$$

$$M = \frac{wl^2}{8} = \frac{9.65 \times 4.88^2}{8} = 28.726 \text{ kNm}$$

$$\therefore \quad I_{\text{eff}} = \frac{I_{cr}}{1.2 - \frac{M_r}{M} \times \frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$

$$= \frac{112167748.7}{1.2 - \frac{18.48}{28.726} \times \left(\frac{138.59}{152}\right)\left(1 - \frac{40.225}{152}\right)(1)}$$

$$= 145925685.9 \text{ mm}^4$$

$\therefore$ Short term deflection, $\delta = \frac{5}{384} \times \frac{9.65 \times (4880)^4}{25000 \times 145925685.9} \text{ mm} = 19.533 \text{ mm}$

Deflection due to shrinkage

$$k_4 = \frac{0.72(p_t - p_c)}{\sqrt{p_t}}$$

$$p_t = \frac{1000 \times \frac{\pi}{4} \times \frac{12^2}{125}}{1000 \times 152} \times 100 = 0.595\%$$

$$p_c = 0$$

$$\therefore \quad k_4 = \frac{0.72(0.595)}{\sqrt{0.595}} = 0.5554$$

$$\psi_{cs} = \frac{k_4 E_{cs}}{D} = \frac{0.5554 \times 0.0003}{178} = 9.3606 \times 10^{-7}$$

$$k_3 = 0.125 \text{ for simply supported members}$$

$$\therefore \quad \alpha_{cs} = \psi_{cs}\, k_3\, l^2 = (9.3606 \times 10^{-7})\, 0.125 \times (4880)^2$$

$$= 2.786 \text{ mm}$$

End of Solution

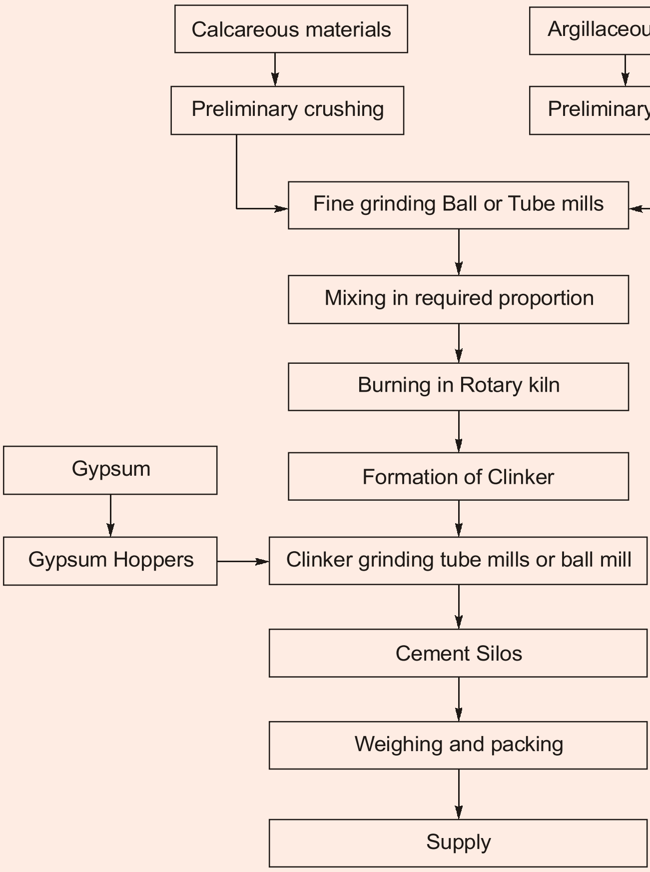
Q.4 (b) (i) Draw a neat sketch of complete operational sequence of Dry process in making cement. Also indicate the place/equipment/part of plant where the process is taking place.

(ii) Indicate the advantages of Dry process over the Wet process.

[10 + 10 = 20 marks]

Solution:

(i)



A rotary kiln is formed of steel tubes. Its diameter varies from 2.50 m to 3 m. Its length varies from 90 m to 120 m. It is laid at a gradient of about 1 in 25 to 1 in 30. The kiln is supported at intervals by columns of masonry or concrete. The kiln rotates at about one to three revolutions per minute about its longitudinal axis. The corrected slurry is injected at the upper end kiln. The hot gases or flames are forced through the lower end of kiln.

Exhaust gases from kiln in dry process can be used for preheating, hence helps to make manufacturing economical.

The portion of the kiln near its upper end the water of slurry is evaporated. As the slurry gradually descend, there is rise in temperature and in the next section of kiln, the carbon dioxide from slurry is evaporated. The small lumps, known as the nodules, are formed at this stage. These nodules then gradually roll down passing through zones of rising temperature and ultimately reach to the burning zone, where temperature is about 1400°C to 1500°C. In burning zone, the calcined product is formed and nodules are converted into small hard dark greenish blue balls which are known as the clinkers.

The size of clinkers varies from 3 mm to 20 mm and they are very hot when they come out of burning zone of kiln. The clinker temperature at the outlet of kiln is nearly 1000°C. The ball mills are used to have preliminary grinding and the tube mills are used to carry out final grinding. The ball mill is in the form of steel cylinder of diameter about 2 m to 2.50 m and length about 1.80 m to 2 m.

The cylinder is placed in horizontal position and it rotates around a steel shaft. On the inside of cylinder, the perforated curved plates are fixed. The ends of these plates overlap each other. The cylinder is filled partly with steel balls of size varying from 50 to 120 mm.

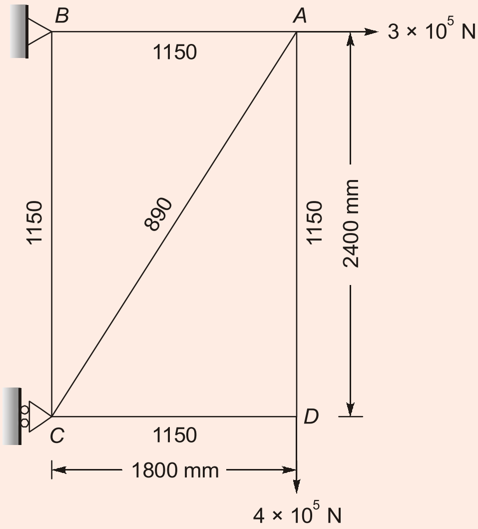
When the mill is rotated about its horizontal axis, the steel balls strike against the perforated curved plates and in doing so, they crush the material. This crushed material passes through an inner sieve plate and then through an outer sieve plate. It is collected from an outlet at the bottom of outer casing of mill. Tube mill is in the form of a long horizontal steel cylinder of diameter about 1.50 m and of length about 7 m to 10 m. The cylinder is filled partly with steel balls of size varying from 20 mm to 25 mm.

The action of tube mill is similar to that of ball mill. But fine grinding is achieved due to steel balls of smaller size. To combine preliminary and final grinding, the compartment mill or multiple chamber mill may be adopted. Such a mill has different chambers or sections in which steel balls of different sizes are placed. The material to be ground is allowed to pass through chambers in succession. The chambers with steel balls of bigger size are placed first and they followed by chamber having steel balls of smaller size.

(ii)
- Dry process require less fuel cost burning.
- Product of dry process can satisfy variable clinker demand
- For dry process short kilns are required.

End of Solution

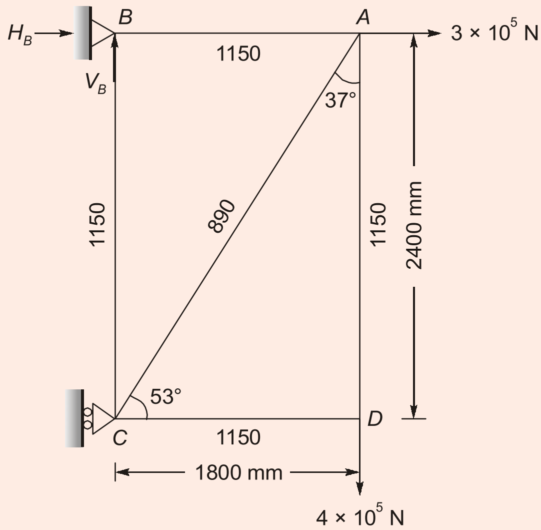
Q.4 (c) Determine the vertical displacement of joint A of the metal truss shown in figure, due to heating from the wall member *BC* is subjected to an increase in temperature of ΔT = 70°C. Take coefficient of thermal expansion of member α = 1.05 × (10^{-5})/°C and E = 210 × 10^3 N/mm². The cross-sectional area of each member is indicated in the figure in mm².



[20 marks]

Solution:

Deflection at *A* only due to external load



Reaction at *B* are H_B and V_B

Reaction at *C* is H_C

Taking, $\qquad M_C = 0$

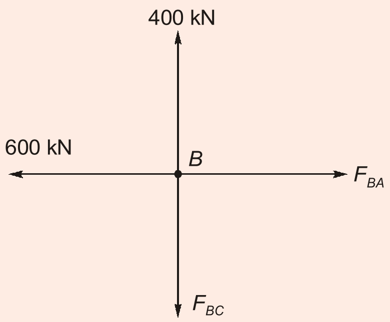
$$H_B \times 2.4 + 300 \times 2.4 + 400 \times 1.8 = 0$$

$$\Rightarrow \qquad H_B = -600 \text{ kN i.e., } 600 \text{ kN } (\leftarrow)$$

$$\therefore \qquad H_C = 300 \text{ kN}$$

$$V_B = 400 \text{ kN}$$

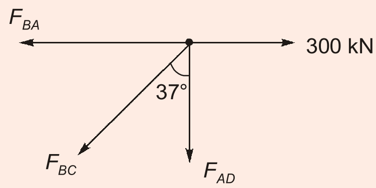
Considering joint B,



So, $$F_{BA} = 600 \text{ kN (Tension)}$$

$$F_{BC} = 400 \text{ kN (Tension)}$$

Considering joint A,



$$\Rightarrow \qquad F_{BA} + F_{AC}\cos 53° = 300 \text{ kN}$$

$$\Rightarrow \qquad 600 + F_{AC} \times 0.6 = 300 \text{ kN}$$

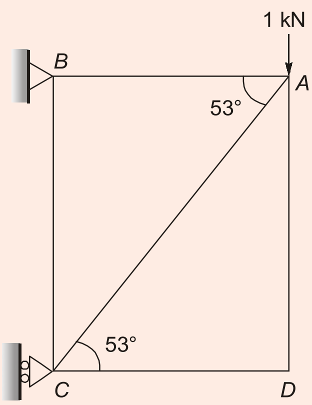
$$\Rightarrow \qquad F_{AC} = -500 \text{ kN (compression)}$$

$$F_{AC}\cos 37° + F_{AD} = 0$$

$$\Rightarrow \qquad F_{AD} = 400 \text{ kN (Tension)}$$

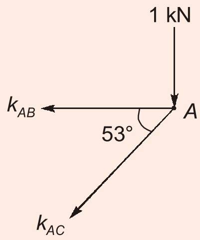
$$F_{CD} = 0$$

Now remove external load and apply unit load in direction of A,



$$k_{DC} = k_{AD} = 0$$

Considering joint A,



$$k_{AC} \cos 53° + k_{AB} = 0$$

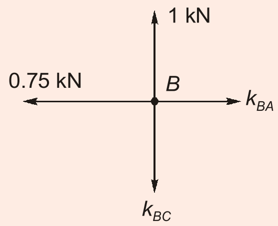
$$\Rightarrow \quad k_{AC} \sin 53° + 1 = 0$$

$$\Rightarrow \quad k_{AC} = -1.25$$

$$\Rightarrow \quad k_{AB} = 0.75$$

$$\Rightarrow \quad k_{BC} = 1$$

Considering joint B,



Now,

Member	P (kN)	k (kN)	L (m)	PkL	$\frac{PkL}{AE}$
AB	600	0.75	1.8	810	3.354×10^{-3}
BC	400	1	2.4	960	3.975×10^{-3}
CD	0	0	1.8	0	0
DA	400	0	2.4	0	0
AC	–500	–1.25	3	1875	0.01

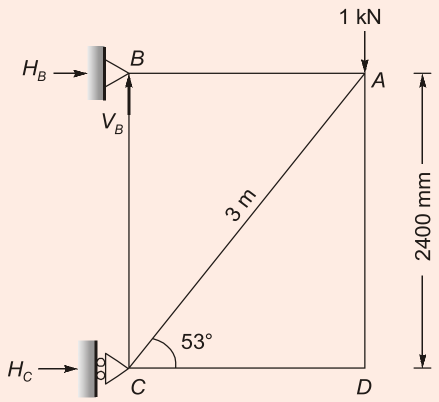
$$\Delta = \sum \frac{PkL}{AE} \text{ (Due to external load only)}$$

$$\Rightarrow \quad \Delta = (3.354 + 3.975 + 10.03) \text{ mm}$$

$$= 17.36 \text{ mm}$$

Now, deflection at A due to temperature change.

We need to calculate force in members BC only because no temperature change in given in other member when unit load is applied in direction of deflection at A.



$$V_B = 1\text{ kN}$$

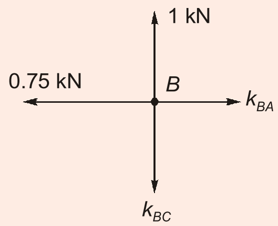
Taking $M_C = 0$

$\Rightarrow$ $H_B \times 2.4 + 1 \times 1.8 = 0$

$\Rightarrow$ $H_B = -0.75\text{ kN}$

$\Rightarrow$ $H_B = 0.75\text{ kN} (\leftarrow)$

Considering joint B,



$$k_{BC} = 1$$

Therefore, $\Delta_A = k_i(L\alpha\Delta T)$

$= 1\times(2400 \times 1.05 \times 10^{-5} \times 70)$

$= 1.764\text{ mm}$

So, Total deflection at A $= 17.36 + 1.764 = 19.124\text{ mm}$

End of Solution

Section-B

Q.5 (a) (i) To put large quantity of concrete in short duration with accuracy, which pump should be used and why? Compare it with other available pumps which are used for concreting.

(ii) What safety measures should be taken in case of placing the concrete using pumps?

[6 + 6 = 12 marks]

Solution:

(i) Pumping of concrete through steel pipelines is one of the successful methods of transporting concrete. Pumped concrete has largely been used in construction of multistory buildings tunnels and bridges. The pump capacity can range from 15 m^3/h to 150 m^3/h. The normal distance to which the concrete can be pumped is about 400 m, horizontally and 80 m vertically. Usually 1 m of vertical movement is equivalent to approximately 10 m horizontally. Bends in the pipeline reduce the effective pumping distance by approximately 10 m for each 90 degree bend, 5 m for 45 degree bend, and 3 m for 22.5 degree bend.

A modern concrete pump, basically consists of three parts: a concrete receiving hopper, a controlling valve system and concrete transmission system. In the commonly used pump called squeeze pump, the concrete placed in the receiving hopper is fed by rotating blades into the flexible pipe connected to the pumping chamber, which is under vacuum of about 600 mm of mercury. Two rotating rollers progressively squeeze the flexible pipes and force the concrete to move through the delivery pipe in a continuous flow. The diameter of the pipe depends on the pumping pressure and the size of aggregate. For long horizontal distance involving high pumping pressure, a larger diameter pipe would be suitable for reduce resistance to flow. On the other hand, for pumping concrete to heights, smallest possible diameter pipelines should be used from gravity considerations. The pipe diameter should be between 3 to 4 times the maximum size of aggregate. As a guide, a pump with an output of 30 m^3/h and with length of pipe line not exceeding 200 m may have a diameter of 100 mm, but for lengths in excess of 500 m a 150 mm diameter could be considered. Generally, 125 mm diameter pipes are used . The pipeline should be carefully laid and well anchored well anchored when bends are introduced for trouble free pumping operation. The pumps should not be kept very close to the vertical pipe. There must be a starting distance of about 10 to 15 percent of the vertical distance.

(ii) Although the method of transporting and placing concrete by pumps is fast and efficient, a small part of unpumpable mix in hopper can block the pump, leading to delay while the pump is stripped down. The blockage is indicated by an increase in the pressure shown on the pressure gauge. Most blockage occur at the tapered sections at the pump end. The reasons include unsuitability of concrete mix, pipeline and joint deficiencies, careless use of hose end, and operator's errors. High temperatures, may also cause blockage. Chances of blockage are least in continuous pumping. A pipeline not well cleaned after previous operation, uncleared and worn-out hoses, too may or too sharp bend and use of worn-out joints add to the problem of blockage.

Great attention is required in the design of mix, for a minor variation in the concrete mix is sufficient to make an otherwise pumpable mix completely unpumpable. At the end of the run, the pipeline must be cleared of concrete by inserting a plunger at the pipe end and forcing it through under pressure. After the concrete is cleared, the pipeline is washed out to leave a smooth clean surface ready for next day's work. The minor blockage may be cleared by forward and reverse pumping.

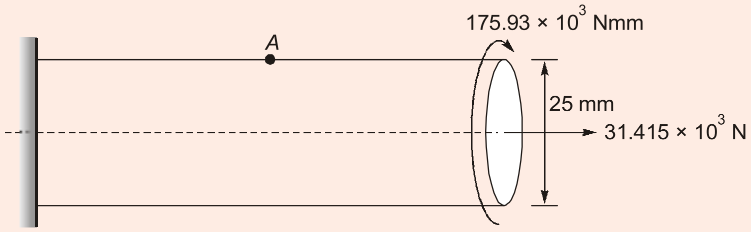
End of Solution

Q.5 (b) Determine the maximum principal stresses and maximum shearing stresses at a point A on the surface of a 25 mm diameter shaft. The shaft is subjected to a torque of 175.93 Nm and axial tension of 31.415 kN. Show the orientation of principal planes and planes of maximum shearing stress with respect to an axis parallel to the axis of the shaft.

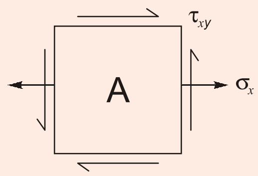
[12 marks]

Solution:

Consider the figure shown below.



Stress element for point *A* on surface of shaft:



$$\sigma_x = \frac{\text{Axial force}}{\text{Area}} = \frac{31.415\times10^3}{\frac{\pi}{4}\times25^2} = 64\,\text{N/mm}^2$$

$$\tau_{xy} = \frac{16T}{\pi d^3} = \frac{16\times175.93\times10^3}{\pi\times25^3} = 57.34\ \text{N/mm}^2$$

Principal stresses:

$$\sigma_{p1}/\sigma_{p2} = \left(\frac{\sigma_x+\sigma_y}{2}\right) \pm \sqrt{\left(\frac{\sigma_x-\sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

$\sigma_y = 0$

So,
$$\sigma_{p1}/\sigma_{p2} = \frac{64}{2} \pm \sqrt{\left(\frac{64}{2}\right)^2 + (57.34)^2}$$

∴
$$\sigma_{P1} = 97.66\ \text{N/mm}$$
$$\sigma_{P2} = -33.665\ \text{N/mm}^2$$

Maximum shear stress,

$$\tau_{xy} = \left|\frac{\sigma_{P1}-\sigma_{P2}}{2}\right|$$

$$= \frac{97.66-(-33.66)}{2} = 65.66\ \text{N/mm}^2$$

Orientation of planes w.r.t. vertical plane,

$$\tan 2\theta_p = \frac{2\tau_{xy}}{\sigma_x - \sigma_y} = \frac{2 \times 57.35}{64}$$

$$2\theta_p = 60.84°$$

$$\theta_p = 30.42°$$

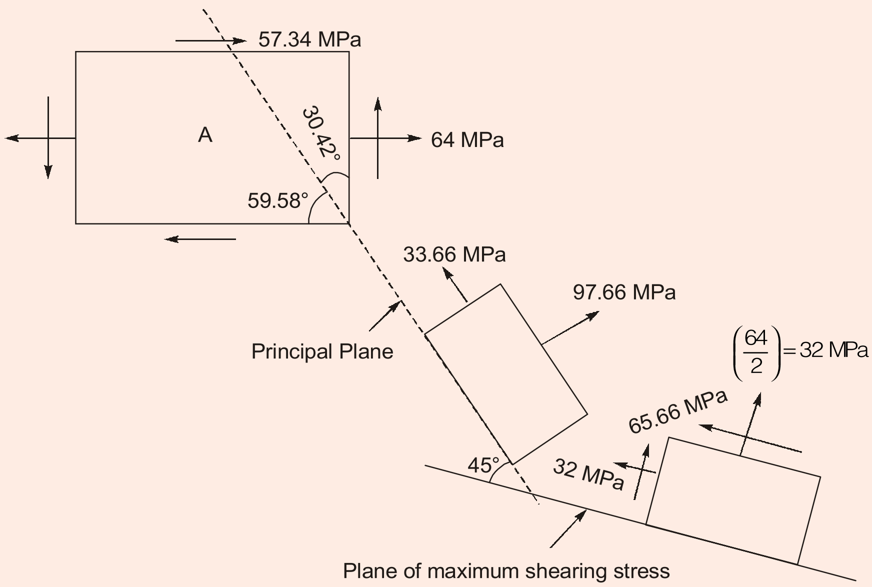
Check: σ_x' at $\theta = 30.42°$

$$\sigma_x' = \left(\frac{\sigma_x + \sigma_y}{2}\right) + \left(\frac{\sigma_x - \sigma_y}{2}\right)\cos 2\theta + \tau_{xy}\sin 2\theta$$

$$= \left(\frac{64}{2}\right) + \left(\frac{64}{2}\right)\cos 60.84° + 57.34 \sin 60.84°$$

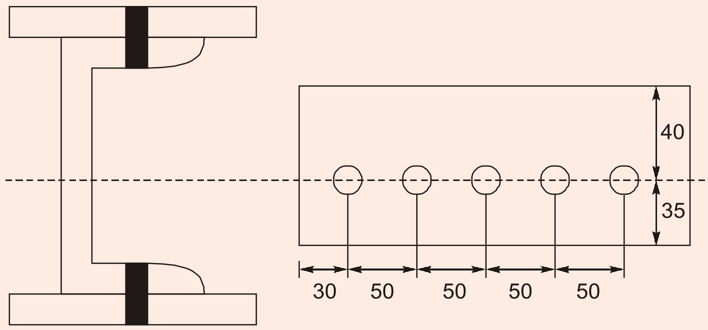
$$= 97.66 \text{ N/mm}^2$$

Hence 97.56 N/mm^2 will be at 30.42° from vertical plane and will be at 59.58° w.r.t. an axis parallel to the axis of shaft.



End of Solution

Q.5 (c) Determine the tensile strength of an ISMC 175 when it is connected to gusset plates through the two flanges by two rows of 16 mm bolts with a connection length 200 mm as shown in the figure. Use grade Fe410 steel (f_y = 250 MPa). Also given γ_{m0} = 1.1 and γ_{m1} = 1.25.



Properties of ISMC 175:

A = 2490 mm^2, h = 175 mm, b_f = 75 mm, t_f = 10.2 mm and t_w = 6 mm.

[12 marks]

Solution:

Gross strength of channel section,

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2490 \times 250}{1.1} \text{ N} = 565.91 \text{ kN}$$

Net strength of channel section,

Here the unconnected part is web

$$A_{go} = (175 - 10.2) \times 6 = 988.8 \text{ mm}^2$$

Connected part is flange $$A_{nc} = \left(75 - \frac{6}{2} - 18\right) \times 10.2 \times 2 = 1101.6 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{m0}}$$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t_w}\right)\left(\frac{b_s}{L_c}\right)\left(\frac{f_y}{f_u}\right)$$

$$0.7 \leq \beta \leq 0.9 \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$$

For the given section, $$w = \frac{175}{2} \text{ mm} = 87.5 \text{ mm (shear lag is considered in web)}$$

Shear lag distance, $$b_s = w + g - t$$

$$= \frac{175}{2} + 40 - \left(\frac{10.2+6}{2}\right) = 119.4 \text{ mm}$$

$$\beta = 1.4 - 0.076 \times \left(\frac{87.5}{6}\right)\left(\frac{119.4}{200}\right)\left(\frac{250}{410}\right)$$

$$\beta = 0.996$$

$$0.7 < \beta \; (= 0.996) < 0.9 \times \frac{410}{250} \times \frac{1.1}{1.25}$$

$$0.7 < 0.996 < 1.298$$

$$T_{dn} = \frac{0.9 \times 410 \times 1101.6}{1.25} + \frac{0.996 \times 250 \times 988.8}{1.1}$$

$$= 325.192 \text{ kN} + 223.828 \text{ kN}$$

$$= 549.02 \text{ kN}$$

Block shear strength of connection,

$$T_{db} = \text{Minimum of } T_{db1} \text{ and } T_{db2}$$

$$A_{vg} = 230 \times 10.2 = 2346 \text{ mm}^2$$

$$A_{tn} = \left(35 - \frac{18}{2}\right) \times 10.2 = 265.2 \text{ mm}^2$$

$$A_{vn} = \left(230 - 4 \times 18 - \frac{18}{2}\right) \times 10.2 = 1519.8 \text{ mm}^2$$

$$A_{tg} = 35 \times 10.2 = 357 \text{ mm}^2$$

$$T_{db1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right] \times 2$$

$$T_{db1} = \left(\frac{2346 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 265.2 \times 410}{1.25}\right) \times 2 \text{ N}$$

$$= 772.24 \text{ kN}$$

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \left(\frac{0.9 \times 1519.8 \times 410}{\sqrt{3} \times 1.25} + \frac{357 \times 250}{1.1}\right) \times 2 \text{ N}$$

$$= 680.32 \text{ kN}$$

$$T_{db} = \text{minimum } [772.24, 680.32]$$

$$= 680.32 \text{ kN}$$

$\therefore$ Tensile strength of given channel section is 549.02 kN.

End of Solution

Q.5 (d) How are the various types of loads estimated for the design of roof truss?
[12 marks]

Solution:

Various type of loads on Roof Truss:

1. **Dead load:** The dead loads on roof trusses consists of (i) weight of roof covering; (ii) weight of purlins; (iii) weight of bracings; and (iv) self-weight of the trusses.

   (i) Weight of roof covering

	Type of covering	Weight per m^2 of plan area
1.	Trafford Asbestos sheets	159 N/m^2
2.	20 gauge CGI sheets	112.7 N/m^2

   *(ii) **Weight of purlins:*** The load due to weight of purlins per square meter of plan area, may be assumed as 70 to 120 N for glazed roofing, 60 to 90 N for G.I. sheeting and 90 to 150 N for A.C. sheeting

   *(iii) **Weight of bracings:*** The load due to the weight of bracings may be assumed as 12 to 15 N/m^2 of plan area.

   Generally, the load due to self-weight of the truss is estimated from the following empirical expression applicable for pitch equal to 1 in 4 and spacing of 4 m, with corrugated G.I. sheets.

$$w = 10\left(\frac{L}{3} + 5\right)$$

   where, w = load per square meter of plan area, due to weight of the truss, in N/m^2.

   L = span of the truss, in meters.

2. **Imposed load**: IS 875 recommends that the roofs with slope upto and including 10°, live load measured on plan should be taken as 1500 N/m^2 if access to roof in provided, and as 750 N/m^2 if access to roof is not provided except for the maintenance. For sloping roofs with slope greater than 10°, the live load may be taken as 750 N/m^2 less 20 N/m^2 for every degree increase in slope over 10°, subject to a minimum of 400 N/m^2 of the plan area. For members supporting the roof members and roof purlins, such as trusses, beam, girders etc, for live load may be taken equal to 2/3rd of the above load.

3. **Snow load:** IS 875 recommends a snow load of 2.5 N/m^2 per mm depth of snow. No snow load may be considered if the slopes are greater than 50°.

4. **Wind load:** The load due to wind is one of the most important loads to be considered in the design of roof trusses and other types of pitched roofs. The design wind pressure is p_z as per IS code 875 Part 3 given by

$$p_z = 0.6\ V_z^2 = 0.6\ (k_1\ k_2\ k_3\ k_4 V_b)^2$$

   where,

   p_z = wind pressure at any height z above MSL

   V_b = basic wind speed in m/s at 10 m height

   k_1 = Probability factor (or risk coefficient)

   k_2 = Terrain, height and structure size factor

k_3 = Topography factor

k_4 = Important factor

Design wind pressure $p_d = k_d\, k_a\, k_c \,.\, p_z$

where, k_d = wind directionality factor

k_a = area averanging factor

k_c = combination factor

The wind force F acting in a direction normal to the individual structural element or cladding unit is

$$F = (C_{pe} - C_{pi})\, Ap_d$$

where C_{pe} = external pressure coefficient

C_{pi} = internal pressure coefficient

End of Solution

Q.5 (e) (i) An axial pull of 45 kN is applied on a steel bar of diameter 13 mm and length of 2500 mm. Determine the change in length, diameter and volume of the steel bar if the Poisson's ratio is 0.25. Use $E = 200 \times 10^3$ N/mm^2.

(ii) Determine the value of Poisson's ratio and Modulus of Elasticity, if the modulus of rigidity of a given material is 50×10^3 N/mm^2 and the bulk modulus is 80×10^3 N/mm^2.

[6 + 6 = 12 marks]

Solution:

(i) Axial pull, P = 45 kN

Diameter, d = 13 mm

Length, L = 2500 mm

Poisson's ratio, μ = 0.25

(a) Change in length
$$\Delta l = \frac{PL}{AE} = \frac{45 \times 10^3 \times 2500}{\frac{\pi}{4} \times 13^2 \times 2 \times 10^5}$$

$$\Delta l = 4.24 \text{ mm}$$

(b)
$$\mu = \frac{-\Delta d / d}{\Delta l / L}$$

$$\Rightarrow \quad -\Delta d = \left[\mu\left(\frac{\Delta l}{L}\right)\right] d$$

$$\Rightarrow \quad -\Delta d = \left[0.25 \times \frac{4.24}{2500}\right] \times 13$$

$$\Rightarrow \qquad \Delta d = -0.0055 \text{ mm}$$

(c) Change in volume,

$$\epsilon_v = \epsilon_l + 2\epsilon_d$$

$$\Delta V = \left[\frac{4.24}{2500} + 2\left(\frac{-0.0055}{13}\right)\right]\left[\frac{\pi}{4} \times 13^2 \times 2500\right]$$

$$\Delta V = 282.005 \text{ mm}^3$$

(ii) Modulus of rigidity, $G = 50 \times 10^3$ N/mm^2

Bulk modulus, $K = 80 \times 10^3$ N/mm^2

Modulus of elasticity 'E'

$$E = \frac{9KG}{3K+G}$$

$$E = \frac{9 \times 80 \times 10^3 \times 50 \times 10^3}{3 \times 80 \times 10^3 + 50 \times 10^3} = 124137.93 \text{ N/mm}^2$$

$$\Rightarrow \qquad E = 3K(1-2\mu)$$

$$\Rightarrow \qquad 124137.93 = 3 \times 80 \times 10^3[1-2\mu]$$

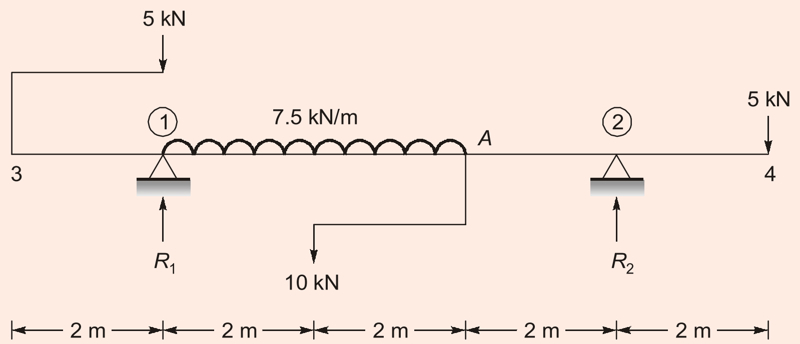
$$\Rightarrow \qquad \mu = 0.24$$

End of Solution

Q.6 (a) (i) Define determinate and indeterminate structures with suitable examples.

(ii) Draw the shear force diagram of the beam shown in figure. Supports (1) and (2) are simply supported.

(iii) Draw the bending moment diagram for the beam shown in figure.



[4 + 8 + 8 = 20 marks]

Solution:

(i) Determinate structures:

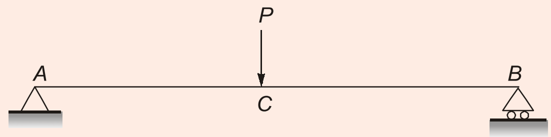
A structure is said to be determinate if conditions of static equilibrium are sufficient to analyse the structure.

(a) In determinate structures, bending moment and shear force are independent of properties of material and cross-sectional area.

(b) No stresses are induced due to temperature changes.

(c) No stresses are induced due to lack of fit and support settlement.

Examples:



Here a simply supported beam is shown. The total number of unknown reaction will be 3 (2 at *A* and 1 at *B)* and number of equilibrium equations is also 3.

Therefore, condition of static equilibrium can be used to calculate value of unknowns.

Indeterminate structures:

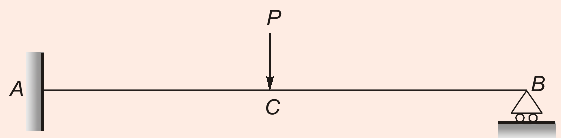
A structure is said to be indeterminate if conditions of static equilibrium are not sufficient to analyse the structure. To analyse these structures, compatibility conditions are required.

(a) In these structure, Bending Moment and Shear Force depends upon properties of material and cross-sectional area.

(b) Stresses are induced due to temperature variation.

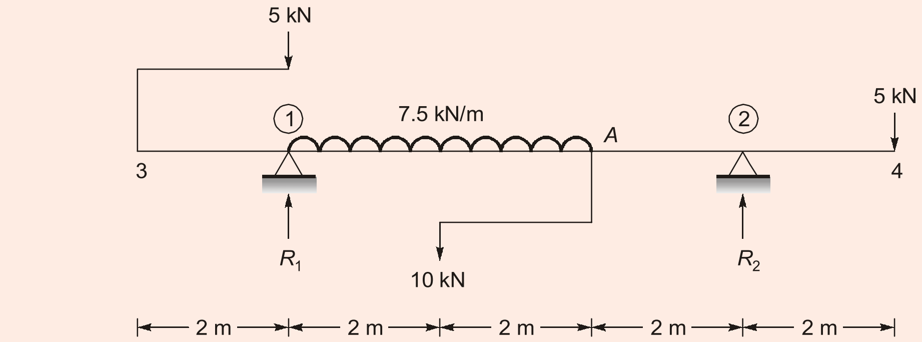
(c) Stresses are induced due to lack of fit and support settlement.

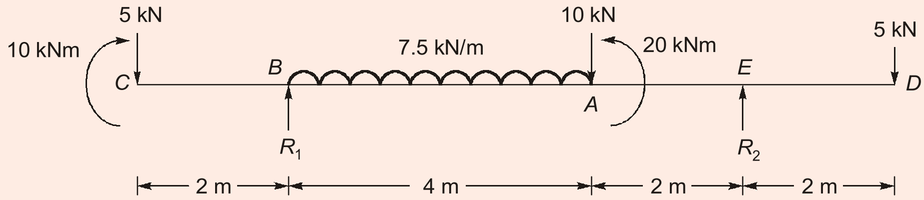
Example:



Here, a propped cantilever beam is shown. The total number of unknown will be 3 at A and 1 at *B*. So it can't be analysed using conditions of static equilibrium alone. A compatibility equation is required to calculate value of unknown.

(ii)





Reactions: $\Sigma F_y = 0$

$$R_1 + R_2 = 50 \quad \text{...(i)}$$

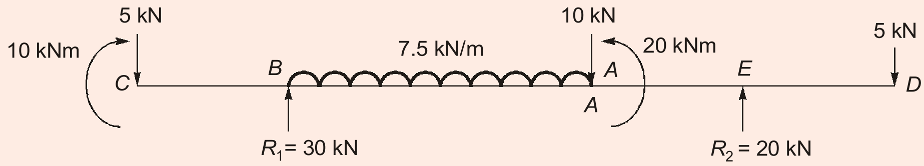
and $\Sigma M_B = 0$

$\Rightarrow \quad 10 - (5 \times 2) + (7.5 \times 4 \times 2) + (10 \times 4) - 20 - (R_2 \times 6) + 5 \times 8 = 0$

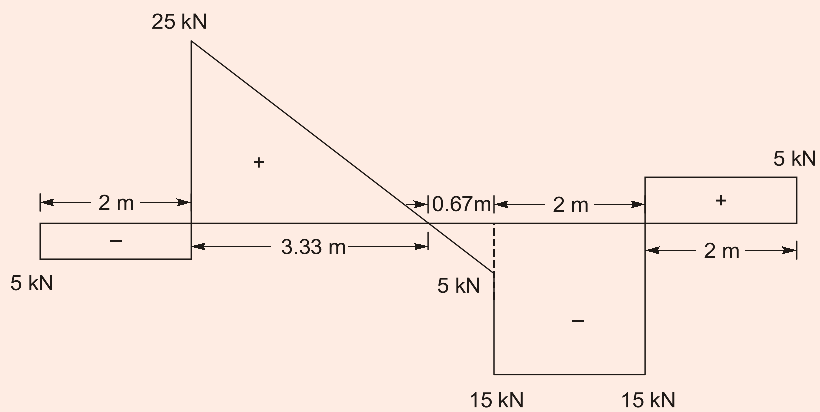
$\Rightarrow \quad R_2 = 20 \text{ kN}$

From eq. (i),

$$R_1 = 50 - 20 = 30 \text{ kN}$$



SFD



Portion *CB*:

$$S_x\ (x \text{ from C}) = -5 \text{ kN} \qquad [0 \le x < 2]$$

At $x = 0$, $\quad S_C = 5$ kN (Constant)

At $x = 2$ m,

$$S_B \text{ (just left of } B) = -5 \text{ kN}$$

$$S_B \text{ (just right of } B) = -5 + 30 = 25 \text{ kN}$$

Portion *BA*: (linear variation)

$$S_x\ (x \text{ from } B) = 25 - 7.5x \qquad [0 \le x < 4]$$

$$S_x = 0 = 25 - 7.5x$$

$$x = \frac{25}{7.5} = 3.33 \text{ m}$$

At $x = 0$,

$$S_B \text{ (just right of } B) = 25 \text{ kN}$$

At $x = 4$ m, $\quad S_A = 25 - 7.5 \times 4$

$$S_A = -5 \text{ kN (just left of } A)$$

Portion *AE*:

$$S_x = -5 + (-10) \qquad (0 \le x < 2)$$

$$S_x = -15 \text{ kN}$$

At $x = 0$, $\quad S_A = -15$ kN (just right of *A*)

At $x = 2$ m, $\quad S_E = -15$ kN (just left at *E*)

Portion *ED*:

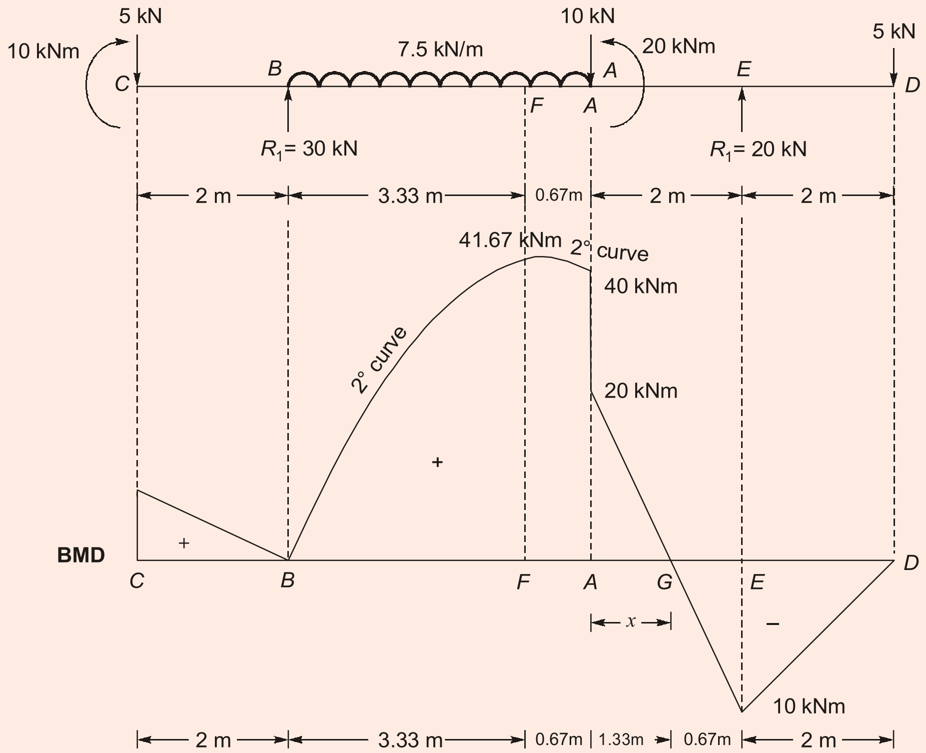
$$S_x = -15 + 20 = +5 \text{ kN} \qquad [0 \le x < 2]$$

At $x = 0$, $\quad S_E = 5$ kN (just right of *E*)

At $x = 2$ m, $\quad S_D = 5$ kN (just left of *D*)

$$S_D = 5 - 5 = 0 \text{ (just right of } D)$$

(iii) Bending moment diagram



Portion CB:

$$M_x\,(x \text{ from } C) = (10 - 5x) \qquad (0 \le x < 2)$$

At $x = 0$, $\quad M_C = 10$ kNm

At $x = 2$ m, $\quad M_B = 0$

Portion DE:

$$M_x\,(x \text{ from } D) = -5x \qquad (0 \le x < 2)$$

At $x = 0$, $\quad M_D = 0$ kNm

At $x = 2$ m, $\quad M_E = -10$ kNm

Portion EA:

$$M_x\,(x \text{ from } E) = -5(x + 2) + 20x \qquad (0 \le x < 2)$$

At $x = 0$, $\quad M_E = -10$ kNm

At $x = 2$ m, $\quad M_A = 20$ kNm (Just right of A)

Portion AF:

$$M_x\,(x \text{ from } A) = -5(4+x) + 20(x+2) + 20 - 10x - \left(\frac{7.5 \times x^2}{2}\right)$$

$$(0 \le x < 0.67)$$

At $x = 0$, $\quad M_A = 40$ kNm (just left of A)

At $x = 0.67$ m, $\quad M_F = 41.67$ kNm

Portion FB:

$$M_x\ (x \text{ from } F) = -5(4.67+x)+20(2.67+x)+20-10(0.67+x)-\frac{\left[7.5(x+0.67)^2\right]}{2}$$

$$(0 \le x < 3.33)$$

At $x = 0$, $\quad M_x = 41.67$ kNm

At $x = 3.33$ m, $\quad M_x = 0$

In portion AE, location of zero BM (point of contraflexure)

$$\frac{20}{x} = \frac{10}{2-x}$$

$\Rightarrow \quad x = 1.33$ m

End of Solution

Q.6 (b) An upper storey column ISHB 300@577 N/m (b_f = 250 mm, t_f = 10.6 mm) carries a factored load of 1200 kN and a factored moment of 12 kNm. It is to be spliced with lower storey column ISHB 400@806 N/m (b_f = 250 mm, t_f = 12.7 mm). Design a suitable splice. Given that f_y = 250 MPa, γ_{m0} = 1.1, γ_{m1} = 1.25 and β_{pk} = (1 – 0.0125t_{pk}). Also sketch connection details. Use bolts of Grade 5.8.

[20 marks]

Solution:

Given data: $\quad f_y = 250$ MPa

$\gamma_{m0} = 1.10$

$\gamma_{m1} = 1.25$

Bolt grade 5.8

$f_{ub} = 520$ MPa

$f_{yb} = 400$ MPa

Properties of ISHB 400 @ 806 N/m

$b_F = 250$ mm

$t_F = 12.7$ mm

Properties of ISHB 500 @ 577 N/m

$b_F = 250$ mm

$t_F = 10.6$ mm

1. Packing plate thickness,

   As the two sections to be spliced are of different depths, packing plate will be provided.

   $$\text{Thickness of packing plate } = \frac{400-300}{2} = 50 \text{ mm}$$

2. Bearing plate design,

A bearing plate is provided between two column sections.

$$\text{Length of plate} = 400 \text{ mm}$$

$$\text{Width of plate} = 250 \text{ mm}$$

$$\text{Direct load on each flange} = \frac{1200}{2} = 600 \text{ kN}$$

Distance between the CG of column flanges of ISHB 400

$$= 400 - 12.7 = 387.3 \text{ mm}$$

Distance between the CG of column flanges of ISHB 300

$$= 300 - 10.6 = 289.4 \text{ mm}$$

Distance between the line of action of forces on the flanges of the two column sections

$$= \frac{387.3 - 289.4}{2} = 48.95 \text{ mm}$$

$$\text{Moment due to the couple} = 600 \times 48.95 \times 10^{-3} = 29.37 \text{ kNm}$$

$$\text{Total moment} = 29.37 + 12 = 41.37 \text{ kNm}$$

Equating the bending moment to the moment of resistance of the plate section,

$$41.37 \times 10^{-6} = \frac{1}{6} \times 250 \times t^2 \times \frac{250}{1.1}$$

$$t = 66.09 \text{ mm} \simeq 70 \text{ mm}$$

Provide a bearing plate of size 400 mm × 250 mm × 70 mm

3. Splice plate design

Assuming the column ends are made flash. Splices will be designed for 50% of the load on one flange.

$$P = \frac{1}{2} \times 1200 \times 0.5 = 300 \text{ kN}$$

Assuming 6 mm thickness of splice plate.

$$\text{Load on splice due to moment} = \frac{12 \times 10^3}{(400 + 6)} = 29.55 \text{ kN}$$

$$\text{Total load} = 300 + 29.55 = 329.55 \text{ kN}$$

Cross-sectional area of the splice plate required

$$= \frac{329.55 \times 10^3}{250} = 1318.2 \text{ mm}^2$$

$$\text{Width of splice plate} = \text{Width of column} = 250$$

$$\text{Thickness} = \frac{1318.2}{250} = 5.27 \simeq 6 \text{ mm}$$

Provide splice plate 250 mm × 6 mm

Length of splice plate.

Let us provide 20 mm diameter bolts of grade 5.6.

Strength of bolt in single shear $= \dfrac{520}{\sqrt{3}\times 1.25}\times 0.78\times\dfrac{\pi}{4}\times 20^2 = 58.85 \text{ kN}$

Strength of bolt in bearing (Assume $k_b = 0.5$)

$$= \frac{2.5 k_b d t f_y}{\gamma_{mb}} = \frac{2.5\times 0.5\times 20\times 6\times 410}{1.25} = 49.2 \text{ kN}$$

∴ Strength of bolt, $V_{db} = 49.2$ kN

Number of bolts required for lower column,

$$n = \frac{329.55}{49.2} = 6.698 \simeq 8$$

For bolts in upper column shear strength of bolt will be reduced.

Shear strength of bolt $= b_{pkg} \times V_{dsb}$

$= (1 - 0.0125 \times 50) \times 58.85 = 22.07$ kN

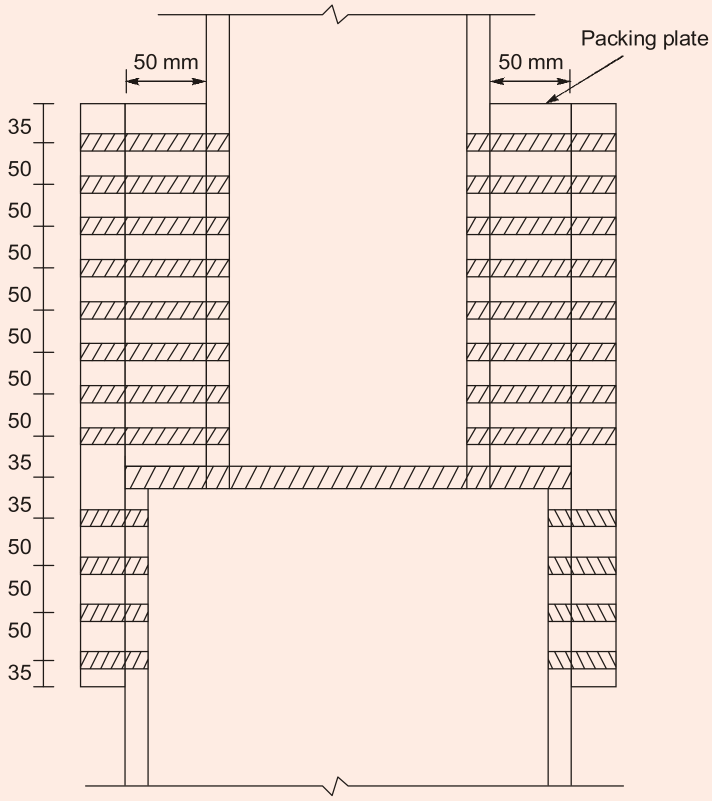
Bearing strength of bolt $= 49.2$ kN

No. of bolt required for upper column

$$= \frac{329.55}{22.07} = 14.93 \simeq 16$$

Provide 8, 20 mm diameter bolts to connect splice plate with the flange of lower storey column and 16, 20 mm diameter bolts to connect the splice plates to the flanges of upper storey column.

Provide the bolts in 2 vertical rows at pitch of 50 mm and edge distance of 35 mm as shown in figure.



Length of splice plate = $50 \times 10 + 35 \times 4 = 640$ mm

Provide splice plate of size = $640 \text{ mm} \times 250 \text{ mm} \times 6 \text{ mm}$

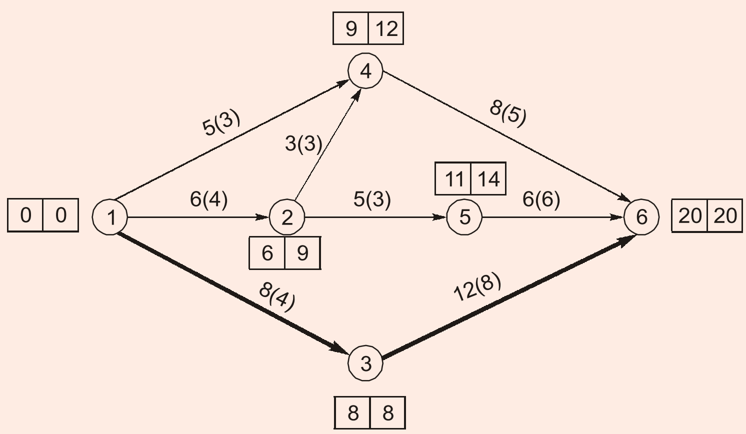
End of Solution

Q.6 (c) The following table shows the normal, shortest and longest durations for each activity to be completed for the completion of the project. The table also shows the slope (increase/decrease) in cost per unit duration of each activity. The contract includes penalty clause of ₹10000/per unit time and bonus of ₹5000/- per unit time for the completion of project with respect to the normal duration of the project. The overhead (indirect) cost per unit time is ₹16000/-. Calculate the optimum cost of project completion. The cost (direct) of completing all the eight activities in normal duration is ₹ 650000/-.

Activity	Normal duration	Minimum duration	Maximum duration	Slope (±)
1–2	6	4	6	8000/-
1–3	8	4	11	Rs. 4000/- for first 2 unit time and Rs. 9000/- subsequently
1–4	5	3	6	3000/-
2–4	3	3	3	—
2–5	5	3	9	8000/-
3–6	12	8	14	Rs. 8000/- for first 3 unit time and Rs. 20000/- subsequently
4–6	8	5	10	5000/-
5–6	6	6	6	—

[20 marks]

Solution:



Total project duration is 20 days and critical path is 1–3–6.

Total project cost for 20 days $= ₹(650000 + 16000 \times 20) = ₹\ 970000$

Crashing activity 1–3 by 2 days,

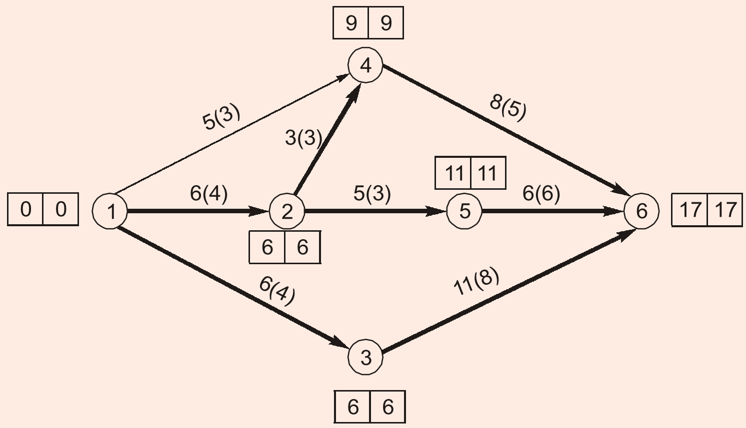
Total project cost for 18 days $=$ ₹ (650000 + 16000 × 18 + 4000 × 2 – 5000 × 2)

$=$ ₹ 936000

Crashing activity 3–6 by 1 day,

Total project for 17 days $=$ ₹ (650000 + 16000 × 17 + 8000 – 5000 × 3)

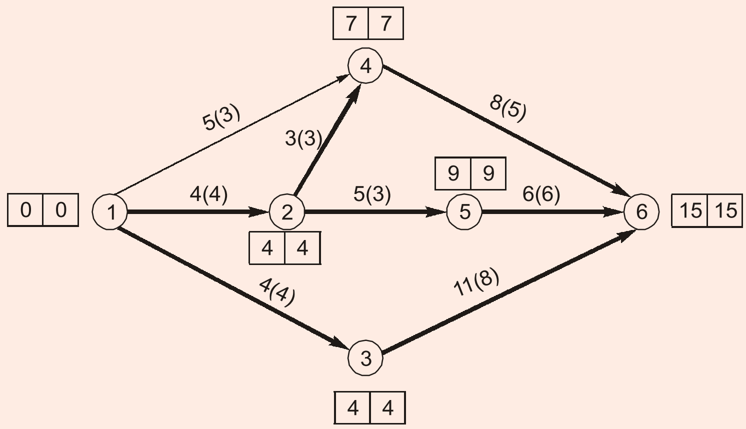
$=$ ₹ 915000



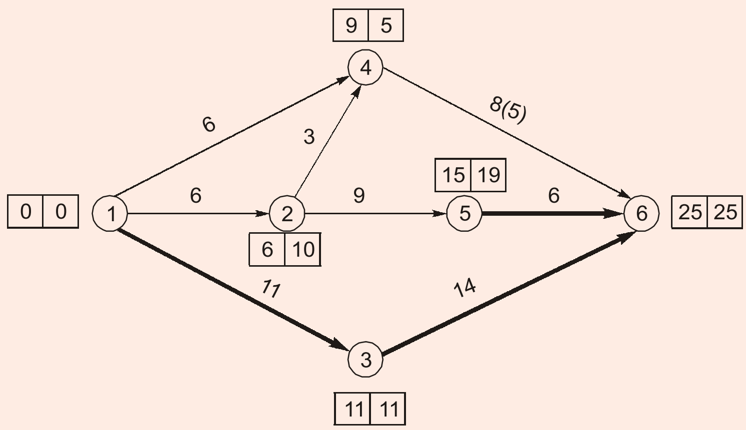
Crashing activity 1–2 by 2 days and activity 1–3 by 2 days

Total project cost for 15 days $=$ ₹ 650000 + 16000 × 15 + 2 × (8000 + 9000) – 5000 × 5

$=$ ₹ 897000



Maximum cost of project,



Maximum cost of project is corresponding to 25 days,

$$= ₹\ 650000 + 16000 \times 25 - (2 \times 4000 + 9000 \times 2 \times 8000)$$

$$+ 5 \times 1000$$

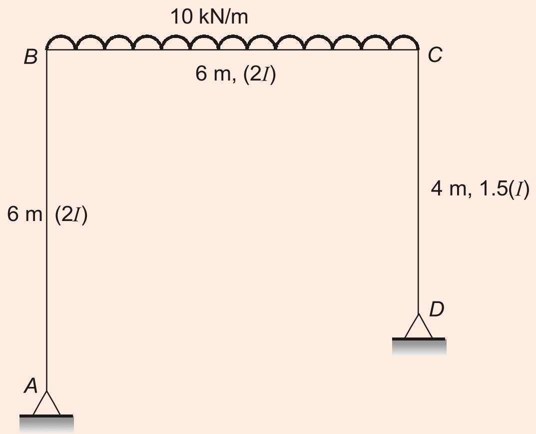
$$= ₹\ 1067000$$

End of Solution

Q.7 (a) A rigid jointed portal frame *ABCD* has the horizontal beam member *BC* of length 6 m and moment of inertia. 2I. The left vertical column *AB* of height 6 m and moment of inertia 2I and the right vertical column *DC* of length 4 m and moment of inertia is 1.5I. The supports *A* and *D* are hinged. A uniformly distributed load of 10 kN/m acts on the full length of the beam *BC*. Analyse the portal and draw the bending moment diagram. The analysis of the frame for a sway load of 1 kN at *B* in the direction of $\overrightarrow{BC}$ gives the moment M_{BC} = +2 kN-m and M_{CB} = +2.66 kN-m both clockwise. The *B* and *C* are at the same level.

[20 marks]

Solution:



$$M_{FBC} = -\frac{10\times 6^2}{12} - 30 \text{ kNm}$$

$$M_{FCB} = 30 \text{ kNm}$$

Due to 1 kN sway force,

$$M_{BC} = +2 \text{ kNm}$$

$$M_{CB} = 2.66 \text{ kNm}$$

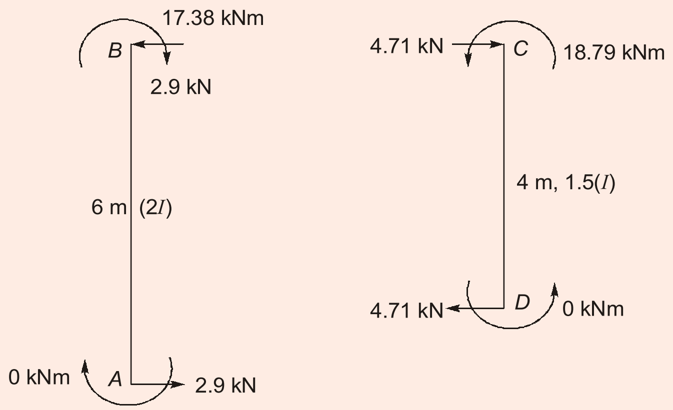
(i) Distribution factors

Joint	Member	k	Σk	DF = $\frac{k}{\Sigma k}$
B	BA	$\frac{3}{4}\left(\frac{2I}{6}\right)$	$\frac{14I}{24}$	$\frac{6}{14}=0.43$
	BC	$\frac{2I}{6}=\frac{8I}{24}$		$\frac{8}{14}=0.57$
C	CB	$\frac{2I}{6}=\frac{I}{3}=0.33I$	$0.61I$	0.54
	CD	$\frac{3}{4}\left(\frac{1.5I}{4}\right)=\frac{9I}{32}=0.28I$		0.46

(ii) BMD

Joint	A	B		C		D
D.F.	1	0.43	0.57	0.54	0.46	1
FEMs	0	0	–30	+30		
Balance		+12.9	+17.1	–16.2	–13.8	
COM	0		–8.1	8.55		0
Balance		+3.48	+4.62	–4.62	–3.93	
COM	0		–2.31	2.31		0
Balance		+1.0	+1.31	–1.25	–1.06	
Final end moment	0	17.38	–17.38	+18.79	–18.79	

To find sway force



$$\Sigma x \text{ at beam level} = -2.9 + 4.7 = 1.8 \text{ kN } (\rightarrow)$$

So, sway force at beam level = 1.8 kN (←)

For a sway force of 1 kN (→)

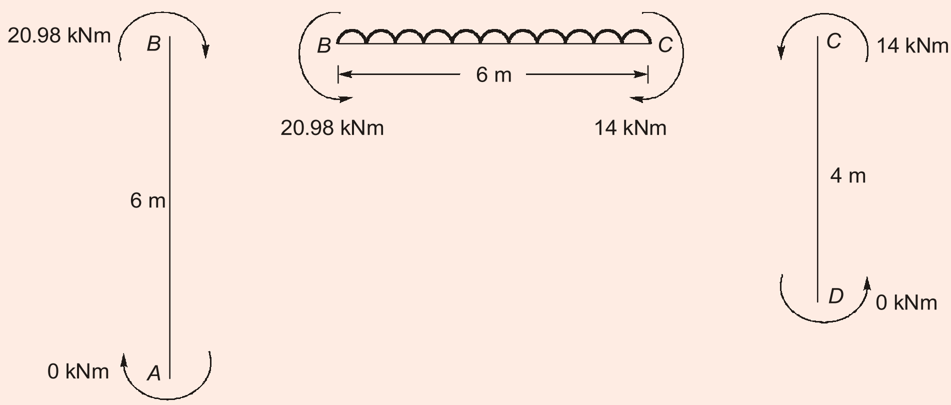
$$M_{BC} = +2 \text{ kNm}$$

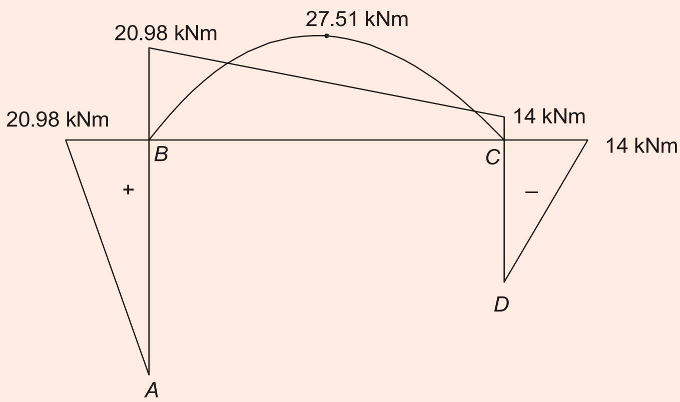
$$M_{CB} = 2.66 \text{ kNm}$$

Correction factor, $\dfrac{\overleftarrow{1.8}}{\overrightarrow{1}} = -1.8$

Final moment

Joint	A		B		C		D
1. Non-sway end moments	0	17.38	–17.38	+18.79	–18.79	0
2. Corrected sway moments	0	+1.8 × 2	–1.8 × 2	+1.8 × 2.66	1.8 × 2.66	0
		= 3.6	= –3.6	= +4.79	= +4.79	0
Final end moments	0	20.98	20.98	14	–14	0





End of Solution

Q.7 (b) Design a welded plate girder 24 m in span and laterally restrained throughout. It has to support a uniform load of 100 kN/m throughout the span, exclusive of self weight. Design the girder cross section without intermediate transverse stiffeners. The steel for the flange and web plates is of grade Fe410. (μ = 0.30 and $E = 2 \times 10^5$ MPa). Given that partial safety factor, $\gamma_{m\omega}$ = 1.5 (for site welding) and $\gamma_{m\omega}$ = 1.25 (for shop welding) and λ_{ω} = 1.2.

[20 marks]

Solution:

For Fe410 grade of steel:

$$f_u = 410 \text{ MPa}, f_y = f_{yp} = f_{yw} = 250 \text{ MPa}$$

$$\mu = 0.3$$

$$E = 2 \times 10^5 \text{ MPa}$$

Partial safety factor, $\gamma_{mw} = 1.50$ (for site welding)

$= 1.25$ (for shop welding)

$$\varepsilon = \varepsilon_w = \varepsilon_f \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

Design forces

Total superimposed load = 100 kN/m

Factored superimposed load = 1.5 × 100 = 150 kN/m

Let, self-weight of plate girder $= \frac{WL}{400} = \frac{(100 \times 24) \times 24}{400} = 144 \text{ kN}$

Self-weight of plate girder per meter length

$$= \frac{144}{24} = 6 \text{ kN/m}$$

Factored self-weight = 1.5 × 6 = 9 kN/m

Total uniform factored load = 150 + 9 = 159 kN/m

Maximum bending moment $= \frac{159 \times 24^2}{8} = 11448 \text{ kNm}$

Maximum shear force $= \frac{159 \times 24}{2} = 1908 \text{ kN}$

Design of web

Optimum depth of plate girder,

$$d = \left(\frac{M_z k}{f_y}\right)^{(0.33)}$$

When intermediate transverse stiffeners are not to be provided:

$$\frac{d}{t_w} \le 200\varepsilon \text{ i.e., } 200 \text{ (from serviceability criteria)}$$

and $\le 345\, \varepsilon_f^2$ i.e., 345 (from flange buckling criteria)

Let us assume $k = \frac{d}{t_w} = 180$

$$d = \left(\frac{11448\times10^6\times180}{250}\right)^{0.33} = 1871.9 \text{ mm} \simeq 1800 \text{ mm}$$

Optimum web thickness, $t_w = \left(\frac{M_z}{f_y k^2}\right)^{0.33}\left(\frac{11448\times10^6}{250\times180^2}\right)^{0.33} = 10.95 \text{ mm} \simeq 12 \text{ mm}$

(Thickness provided in more since intermediate transverse stiffeners are not to be provided)

Let us try web plate 1800 × 12 mm in size.

Design of flanges

Let us assume that bending moment will be resisted by the flanges and shear by the web.

Required area of flange, $A_f = \frac{M_z\gamma_{m0}}{f_y d} = \frac{11448\times10^6\times1.10}{250\times1800} = 27984 \text{ mm}^2$

Assuming width of flange equal to 0.3 times depth of girder.

$$b_f = 0.3\times1800 = 540 \text{ mm} \simeq 560 \text{ mm}$$

Thickness of flange, $t_f = \frac{27984}{560} = 49.97 \simeq 50 \text{ mm}$

Let us try 560 × 50 mm flange plate.

Classification of flanges

For the flanges to be classified as plastic $\frac{b}{t_f} \le 8.4\varepsilon$

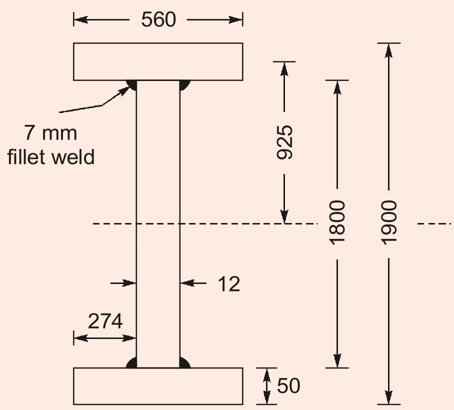
The outstand of flange, $b = \frac{b_f - t_w}{2} = \frac{560-12}{2} = 274 \text{ mm}$

$\frac{b}{t_f} = \frac{270}{50} = 5.48$ < 8.4 (84ε = 8.4 × 1 = 8.4)

Hence, the flanges are plastic. (β_b = 1.0)

Check for bending strength

The trial section of the plate girder is shown in figure. The plastic section modulus of the section.



(All dimensions in mm)

$$Z_{pz} = 2b_f t_f \frac{(D - t_f)}{2} = 2 \times 560 \times 50 \times \frac{1900 - 50}{2}$$

$$= 51.80 \times 10^6 \text{ mm}^3$$

Moment capacity:

$$M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_{m0}} = 1.0 \times 51.80 \times 10^6 \times \frac{250}{1.10} \times 10^6$$

$$= 11772.7 \text{ kNm} > 11448 \text{ kNm}$$

which is safe.

Shear capacity of web

Let us use simple post-critical method.

$$\frac{d}{t_w} = \frac{1800}{12} = 150 < 200 \qquad (200\varepsilon = 200 \times 1 = 200)$$

and also, < 345 $\qquad (345\varepsilon_f^2 = 345 \times 1 = 345)$

which is all right.

Elastic critical shear stress,

$$\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

Transverse stiffeners will be provided at supports only.

Hence, $k_v = 5.35$

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12 \times (1 - 0.3^2) \times 150^2} = 42.98 \text{ N/mm}^2$$

The non-dimensional web slenderness ratio for shear buckling stress.

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 42.98}} = 1.83 \simeq 1.80 > 1.20$$

Shear stress corresponding to buckling (for $\lambda_w > 1.20$)

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2} = \frac{250}{\sqrt{3} \times 1.80^2} = 44.55 \text{ N/mm}^2$$

Shear force corresponding to web buckling,

$$V_{cr} = d t_w \tau_b = 1800 \times 12 \times 44.55 \times 10^{-3} = 962.28 \text{ kN} < 1908 \text{ kN}$$

which is unsafe.

Let us revise the web thickness from 12 mm to 16 mm.

New value of $\tau_{cr,\,e}$, λ_w, τ_b, and V_{cr} will be as follows

$$\frac{d}{t_w} = \frac{1800}{16} = 112.5$$

$$\tau_{cr,\,e} = \frac{5.35\times\pi^2\times2\times10^5}{12\times\left(1-0.3^2\right)\times112.5} = 76.41\text{ N/mm}^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cre}}} = \sqrt{\frac{250}{\sqrt{3}\times76.41}} = 1.374 = 1.37 > 1.2$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3}\,\lambda_w^2} = \frac{250}{\sqrt{3}\times1.37^2} = 76.90\text{ N/mm}^2$$

$$V_{cr} = dt_w\,\tau_b = 1800\times16\times76.90\times10^{-3} = 2214.7\text{ N} > 1908\text{ kN}$$

which is safe.

Check for lateral-torsional buckling

Since the compression flange of the girder is laterally restrained throughout, the possibility of lateral-torsional buckling is not there and this check is not required.

Flange to web connection

There will be two weld length along the span for each flange to web connection.

$$q_w = \frac{VA_f\bar{y}}{2I_z}$$

$$I_z = \frac{b_fD^3}{12} - \frac{\left(b_f - t_w\right)d^3}{12}$$

$$= \frac{560\times1900^3}{12} - \frac{\left(560-16\right)1800^3}{12}$$

$$= 55702.6\times10^6\text{ mm}^4$$

$$q_w = \frac{1908\times560\times50\times\left(900+\frac{50}{2}\right)}{2\times55702.6\times10^6} = 0.4436\text{ kN/mm}$$

Let us provide weld of size, $S = 10$ mm

$$kS = 0.7\times10 = 7.0\text{ mm}$$

Strength of shop weld per unit length,

$$f_{wd} = \frac{4.2\times250\times10^{-3}}{\sqrt{3}\times1.25} = 0.808\text{ kN/mm} > 0.4436\text{ kN/mm}$$

which is all right.

End of Solution

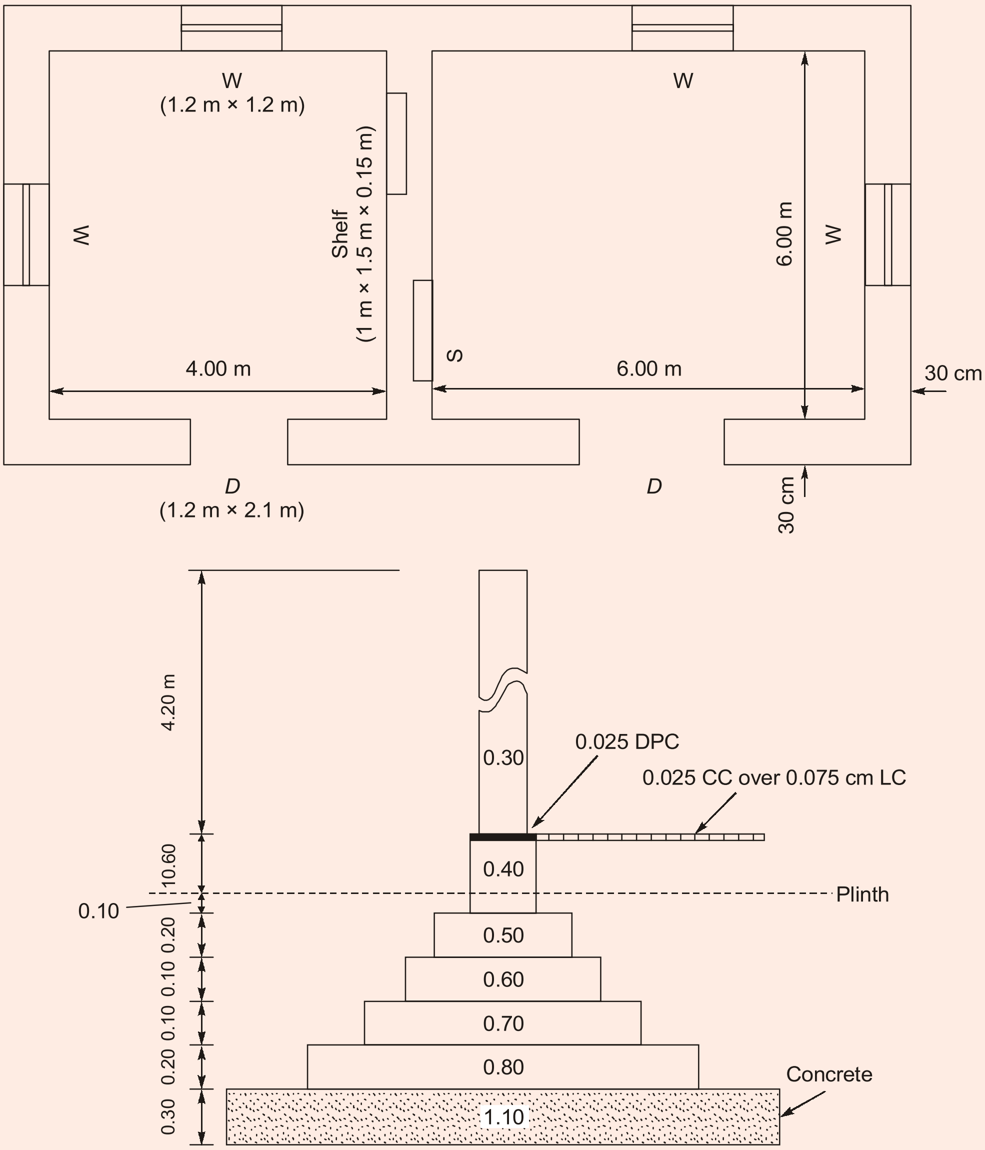
Q.7 (c) Estimate the following quantities for the given structure (plan and section):

(i) Earthwork in excavation in foundation.

(ii) Concrete in foundation.

(ii) Brick work in super structure

All windows, doors and shelves have the dimensions (1.2 m × 1.2 m), (1.2 m × 2.1 m) and (1 m × 1.5 m × 0.15 m) respectively. All dimensions are in m, if not mentioned.



[20 marks]

Solution:

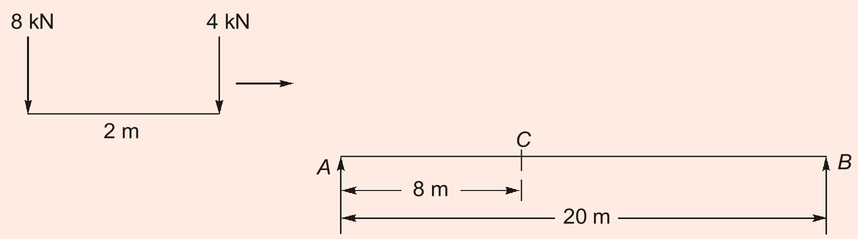
Item No.	Particulars of items	No.	Length	Breadth	Height of Depth	Quantity	Explanatory note
							Long wall, c/c. length = 4 + 6 + 0.30 + 2 × (0.30/2) = 10.60 m Short and inner wall, c/c. length = 6 + 2 × (0.30/2) = 6.30 m
1.	Earthwork in excavation in foundation						
	Long wall	2	11.70 m	1.10 m	1.00 m	25.74	L = 10.60 + 1.10 = 11.70 m
	Short wall	3	5.20 m	1.10 m	1.00 m	17.16	L = 6.30– 1.10 = 5.20 m
					Total	42.90 cu m	
2.	Concrete in foundation						
	Long wall	2	11.70 m	1.10 m	0.30 m	7.72	Length same for excavation
	Short wall	3	5.20 m	1.10 m	0.30 m	5.15	
					Total	12.87 cu m	
3.	1st class brick-work in lime mortar in superstructure						
	Long wall	2	10.90 m	0.30 m	4.20 m	27.47	L = 10.60 + 0.30 = 10.90 m
	Short wall	3	6.00 m	0.30 m	4.20 m	22.68	L = 6.30 – 0.30 = 6.00 m
					Total	50.15 cu m	

End of Solution

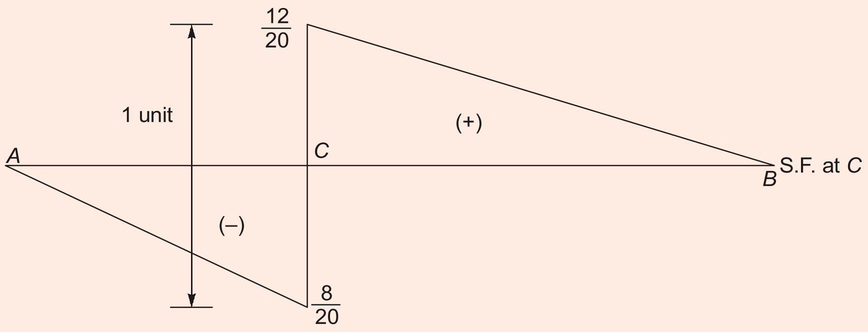
Q.8 (a) Two wheel loads of 8000 N and 4000 N at a fixed distance of 2000 mm, cross a beam of 20 m span. Draw the influence line diagram for bending moment and shear force for a point 8 m from the left side and also determine the maximum bending moment and shear force at that point. Also, evaluate the absolute maximum bending moment due to the given loading system. The loads cross the beam from left to right.

[20 marks]

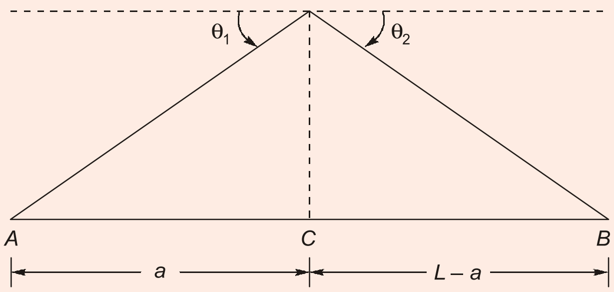
Solution:



Influence line diagram for $(S.F)_C$ using Muller Breslau's principle ILD of stress function be drawn by removing the restraint offered by that stress function and introducing a directly unit displacement. So, giving unit displacement at C and removing the restrain offered by S.F at C, we get



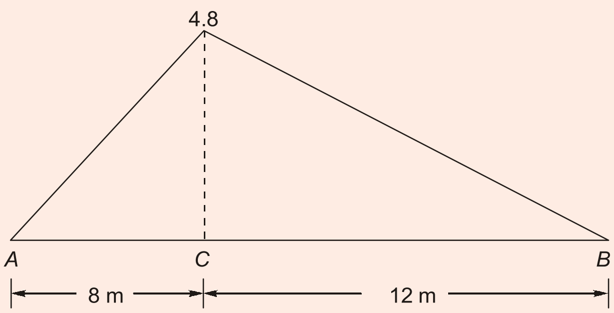
Similarly for Bending Moment at C, provide hinge at C and apply unit displacement in positive direction of moment.



$$\theta_1 + \theta_2 = 1$$

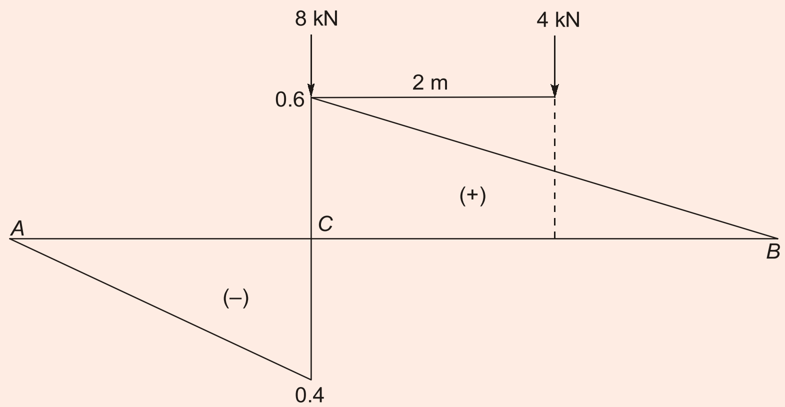
$$\theta_1 = \frac{L-a}{L} \qquad \theta_2 = \frac{a}{L}$$

Putting values of a and L, we get ILD for B.M. at C,



When given load moves on the span AB

(i) To calculate maximum shear force at C ILD for $(S.F.)_C$.



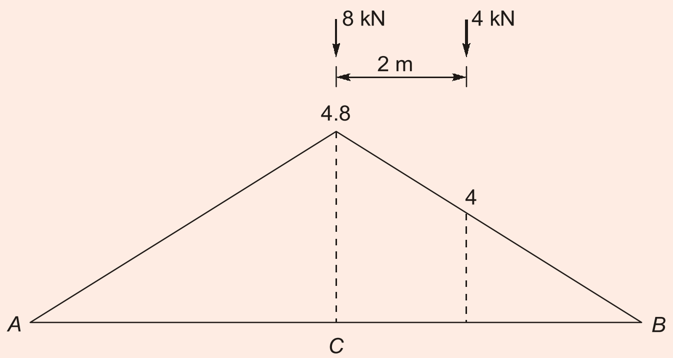
To have maximum shear force, place 8 kN load on just right of C as shown in figure.

The value of ordinates of ILD at 8 kN and 4 kN will be 0.6 and 0.5 respectively.

So,maximum shear force = $8 \times 0.6 + 4 \times 0.5 = 6.8$ kN

(ii) To calculate maximum Bending moment at C.

To have maximum shear force, place 8 kN load on just right of C as shown in figure.

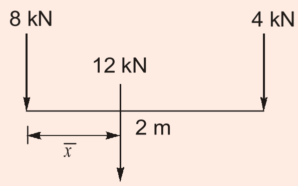


The value of ordinates of ILD at 8 kN and a 4 kN will be 4.8 and 4 respectively.

So, maximum B.M at C = $8 \times 4.8 + 4 \times 4 = 54.4$ kNm

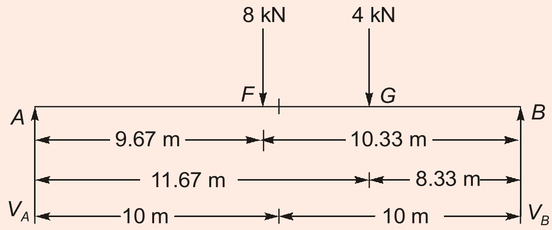
To calculate maximum bending moment any where in beam we have to place the loading on beam in such a way that centre of span is mid way between C.G of load system and load under consideration. Since 8 kN is nearer to C.G. Therefore maximum bending moment will occur below 8 kN.

C.G. of load system:



$$\bar{x} = \frac{8 \times 0 + 4 \times 2}{12} = 0.67\,\text{m}$$

So, place the load as shown in figure.



$$\Sigma M_B = 0$$

$$\Rightarrow V_A \times 20 - 8 \times 10.33 - 4 \times 8.33 = 0$$

$$V_A = 5.798\text{ kN}$$

$$V_B = 6.202\text{ kN}$$

Maximum bending moment will be at F.

$$(\text{B.M})_F = V_A \times 9.67$$

$$= 5.798 \times 9.67 = 56.067\text{ kNm}$$

End of Solution

Q.8 (b) A simply supported beam of 6 m span carries a udl of 70 kN/m including its self weight. The available section is ISMB 450. Cover plates of 8 mm thick may be provided, if required, only on the top flange. Design the steel beam and check for shear and deflection.

Take γ_f = 1.5; γ_{m0} = 1.1. Assume that the section is plastic,

f_y = 250 MPa.

Sectional area of ISMB 450; a = 9227 mm^2.

Width of flange = 150 mm.

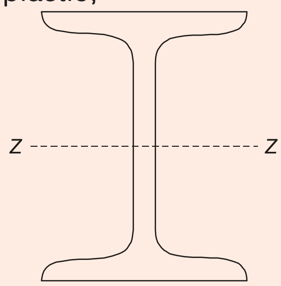
Thickness of flange = 17.4 mm.

Thickness of web = 9.4 mm.

Moment of inertia I_{zz} = 30390.8 cm^4

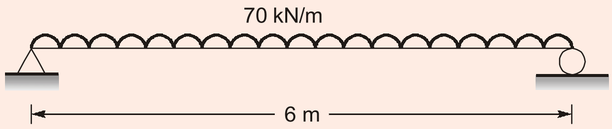
Radius at root = 15 mm

Plastic modulus Z_{pz} = 1533.4 cm^3

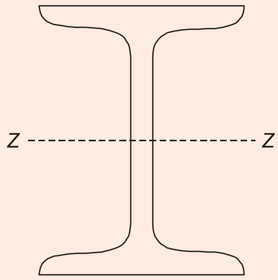


[20 marks]

Solution:



Given: Beam cross-section ISMB 450



$A = 4227\ \text{mm}^2$, $h = 450$ mm, $b = 150$ mm, $t_f = 17.4$ mm, $t_w = 9.4$ mm
$I_{zz} = 30390.8\ \text{cm}^4$, $R_1 = 15$ mm, $Z_{pz} = 1533.4\ \text{cm}^3$, Section is plastic
Factored Bending Moment

$$M_{uz} = \gamma_f \frac{wL^2}{8} = \frac{1.5 \times 70 \times 6^2}{8} = 472.5\ \text{kNm}$$

Maximum factored shear force,

$$V_u = \gamma_f \frac{wL}{2} = \frac{1.5 \times 70 \times 6}{2} = 315\ \text{kN}$$

Section Modulus required

$$(Z_{pz})_{\text{reqd.}} = \frac{M_{uz}}{\left(\frac{f_y}{\gamma_{m0}}\right)} = \frac{472.5 \times 10^6}{\left(\frac{250}{1.1}\right)} = 2079000\ \text{mm}^3 = 2079 \times 10^3\ \text{mm}^3$$

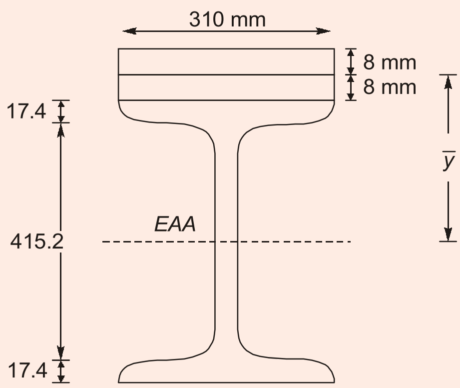
$$(Z_p)_{\text{rolled}} = 1533.4 \times 10^3\ \text{mm}^3$$

$\therefore$ $(Z_p)_{\text{rolled}} < (Z_p)_{\text{reqd.}}$

Let us provide two cover plates of size 310 mm × 8 mm at top flange.

Properties of built-up section

Thickness of cover plate provided = 8 mm



Equal area axis

$$A_1 = A_2$$

$$2\times(310\times 8) + 150\times\bar{y} = 150(17.4-\bar{y})+415.2\times 9.4+150\times 17.4$$

$$\bar{y} = 13.876\text{ mm}$$

$$(Z_{p2})_{\text{built-up}} = \frac{A}{2}(\bar{y}_1+\bar{y}_2)$$

$$\bar{y}_1 = \frac{(310\times 8\times 2)(8+13.876)+(150\times 13.876)(13.875/2)}{7041.4}$$

$$= \frac{108504.96+14440.75}{7041.4} = 17.46\text{ mm}$$

$$\bar{y}_2 = \frac{(150\times 3.524)\times\frac{3.524}{2}+(415.2\times 9.4)\left(\frac{415.2}{2}+3.524\right)+(150\times 17.4)\left(\frac{17.4}{2}+415.2+3.524\right)}{7039.6}$$

$$= \frac{931.4+823991.63+1115576.64}{7039.6} = 275.65\text{ mm}$$

$$Z_p = \frac{74187}{2}(17.46+275.65)$$

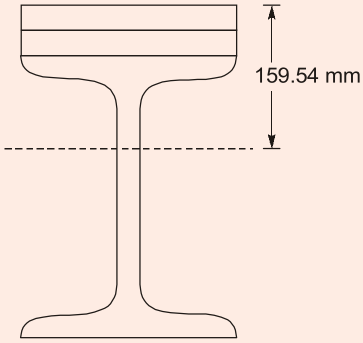
$$= 2079.17\times 10^3\text{ mm}^3$$

It is greater than $(Z_{p2})_{reqd}$ .Hence, safe

Centrodial axis from top

$$\text{C.G distance} = \frac{(310\times 8\times 2)\times 8+9227\times 241}{(310\times 8\times 2)+9227}$$

$$= \frac{2263387}{14187} = 159.54\text{ mm}$$



$$I_{CG} = \left[\frac{310\times16^3}{12} + 310\times16(159.54-8)^2\right]$$
$$+\left[30390.8\times10^4 + 9227(241-159.54)^2\right]$$
$$= (11400.91 + 36513.58)\times10^4\,\text{mm}^4$$
$$= 47914.5\ \text{cm}^4$$

Shear strength check for shear buckling

$$\frac{d}{t_w} = \frac{450-2(t_f+R)}{9.4} = \frac{450-2(17.4+15)}{9.4}$$
$$= 40.97 < 67\epsilon$$

Hence, web will not fail in shear buckling

$\therefore$ Shear strength $V_d = \dfrac{f_y}{\sqrt{3}\cdot\gamma_{m0}}\times A_v$

$$= \frac{250}{\sqrt{3}\times1.1}\times450\times9.4 = 555.04 > V_u$$

Hence safe in shear.

Check for deflection

Deflection are calculated at service load

$$\Delta_{cal} = \frac{5}{384}\frac{WL^4}{EI} = \frac{5\times70\times(6000)^4}{384\times47914.5\times10^4\times2\times10^5}$$
$$= 12.326\ \text{mm}$$

$$\Delta_{max} = \frac{\text{Span}}{300} = \frac{6000}{300} = 20\ \text{mm}$$

As $\Delta_{cal} < \Delta_{max}$

Hence safe in deflection.

End of Solution

Q.8 (c) Calculate the hourly tire cost that should be part of machine operating cost if a set of tires can be expected to last 5000 hr. Tires cost ₹40000/- (per set of four). Tire repair cost is estimated to average 16% of the straight-line tire depreciation. The machine has a service life of 4 yr and operates 2500 hr per year. The discount rate is 8%. Compare both the cases, i.e., without time value and with time value of money.

[20 marks]

Solution:

Hourly tire cost (by not considering time value of money).

The hourly tire cost is equal to the sum of hourly tire use (replacement) cost and hourly tire repair cost. The hourly tire use cost is obtained by dividing the cost of a set of tires by the life of tires in hours. The hourly tire repair cost is equal to 15% of the hourly depreciation (straight-line) cost of tires. The total depreciation cost of a set of tires over its estimated life is equal to its initial cost as its salvage value is assumed as zero.

$$\text{Hourly tire cost} = \text{(hourly tire use cost)} + \text{(hourly tire repair cost)}$$

$$= \frac{₹40000}{5000} + \frac{0.16 \times 40000}{5000} = ₹9.28/\text{hr}$$

Hourly tire cost (considering time value of money)

$$\text{Machine service life} = 4 \times 2500 \text{ hr} = 10000 \text{ hr}$$

$$\text{Tire life} = 5000 \text{ hr}$$

A second set of tires will be purchased at the end of $\frac{10000}{5000} = 2$ years

$$\text{Hourly tire repair cost} = \frac{0.16 \times 40000}{5000} = ₹1.28/\text{hr}$$

$$A_{\text{1st set}} = P\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] = 40000\left[\frac{0.08(1+0.08)^4}{(1+0.08)^4 - 1}\right]$$

$$= ₹12076.83/\text{years}$$

$$\text{First set cost per hour} = \frac{A_{1^{st}\text{ set}}}{2500} = \frac{12076.83}{2500} = ₹4.83/\text{hr}$$

$$A_{\text{2nd set}} = P'\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] = \frac{P}{(1+i)^n}\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right]$$

$$= \frac{40000}{(1+0.08)^2}\left[\frac{0.08(1+0.08)^4}{(1+0.08)^4 - 1}\right]$$

$= ₹\ 10353.94/\text{years}$

Second set cost per hour $= \dfrac{10353.94}{2500} = ₹\ 4.14/hr$

Hourly tire cost $=$ Repair cost + 1st set + 2nd set

$= ₹\ (1.28 + 4.83 + 4.14)\ /\ hr$

$= ₹\ 10.25/hr$

End of Solution

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