ESE 2025

Main Exam Detailed Solutions

Civil Engineering

PAPER-I

EXAM DATE : 10-08-2025 | 09:00 AM to 12:00 PM

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ANALYSIS

Civil Engineering Paper-I
ESE 2025 Main Examination

Sl.	Subjects	Marks
1	Building Materials and Construction	76
2	Strength of Materials	84
3	Structural Analysis	92
4	Steel Structures	52
5	RCC	104
6	Construction Planning and Management	72
		Total 480

Scroll down for detailed solutions

SECTION : A

Q.1 (a) (i) Write the qualities of good timber and the factors affecting the strength of timber.

[6 marks : 2025]

(ii) Write short notes on any three of the following:

(1) Brick buttresses
(2) Brick corbel
(3) Brick coping
(4) Thresholds
(5) Brick jambs
(6) Racking back

[2 × 3 = 6 marks : 2025]

Solution:

(i)

Qualities of good timber:

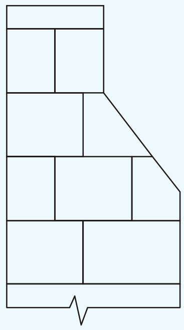
1. Narrow annual rings: This shows slow growth and better strength.
2. Compact medullary rays: They offer more resistance to shrinkage.
3. Dark colour: Heartwood is darker from sapwood and timber from heartwood is stronger.
4. Uniform texture: Even grain and texture give predictable strength and good finish.
5. Sonorous on striking: Indicates sound timber.
6. No woolliness at fresh cut surface.
7. Good timber is reasonably heavy and dense, indicating high strength.

Factors affecting the strength of timber:

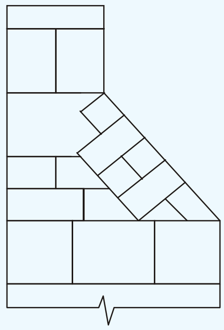
1. Density: Higher density generally means higher strength.
2. Moisture content: Wet timber is less stronger than a well seasoned timber.
3. Presence of defects: Knots, shakes, splits, rots, insects reduce strength of timber.
4. Grain direction and slope of grain: Strength is much greater parallel to the fibres than in perpendicular direction of fibres.
5. Temperature: Prolonged exposure to high temperature can cause significant strength loss.
6. Growth condition: Slower growth rate indicates better structure and density.

(ii)

1. Brick buttresses: A buttress is a masonry projection provided to main walls to strengthen them and to increase the capacity of the wall to resist lateral thrust. Buttresses are usually provided with cappings. It improves wall stability by transferring horizontal loads acting on wall to ground.

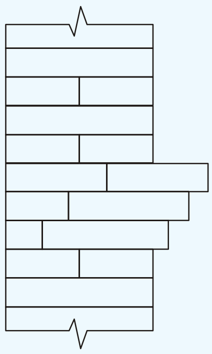


Buttress splayed capping



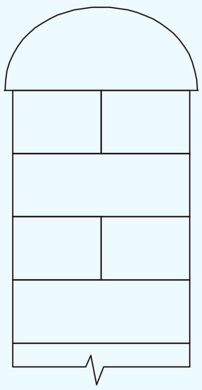
Buttress tumbled in tapping

2. **Brick corbel:** A brick corbel is a projection from the wall built by successively extending courses of brick out from the wall, forming a shelf or ledge that supports a weight or load above it. Besides providing structural support, brick corbels can also serve decorative purposes in architectural designs. Each course should not project more than 50 mm.

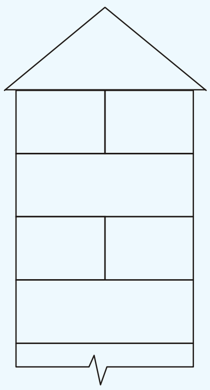


Corbel

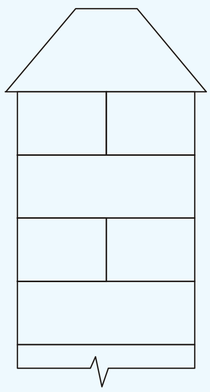
3. **Brick coping:** It is the protective cap or covering placed on the exposed top of a brick wall. It is designed to prevent water from penetrating the wall, thereby protecting it from weather damage or seepage. Brick coping projects slightly beyond the face of the wall and is installed with a slope to direct water away from the surface of wall.



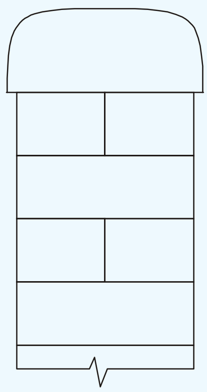
Half round coping



Saddleback coping

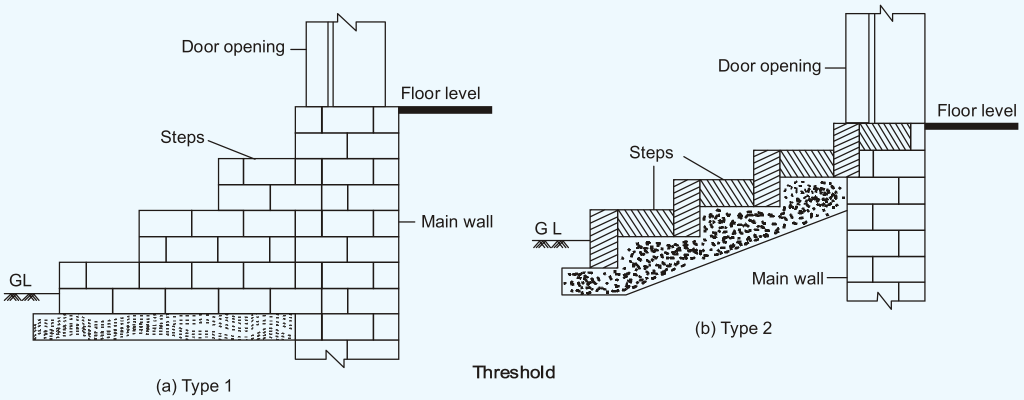


Chamfered coping



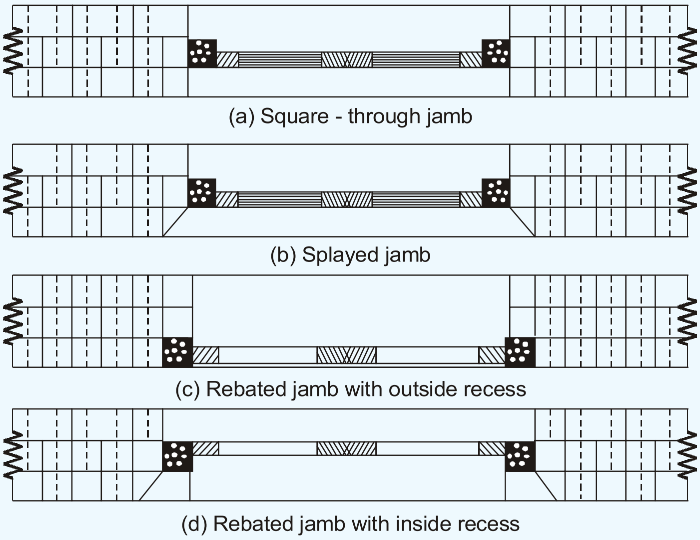
Bull nose coping

4. **Thresholds:** It is the arrangement of steps provided from ground level to reach plinth level on external doors and verandah. For this, brick steps may be provided with gentle slope outwards and with impervious top finish in order to drain off rainwater.



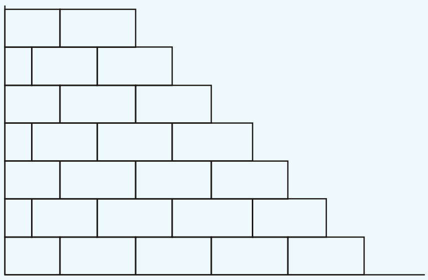
Threshold

5. **Brick jambs:** The vertical sides of a finished opening for door, window or fire place etc. are termed as jambs. Jambs may be plain, rebated or splayed. They provide structural support to the frame, hold hardware such as hinges and locks and ensure the proper fitting and operation of doors and windows.



Jambs

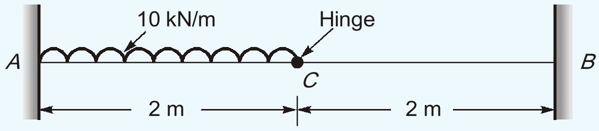
6. **Racking back:** It refers to the process of stopping the unfinished end of the wall in a stepped or staggered fashion by stepping back successive courses of bricks. It allows for proper bonding when the wall is extended later or continued, providing a mechanical interlock between old and new work.



Racking back

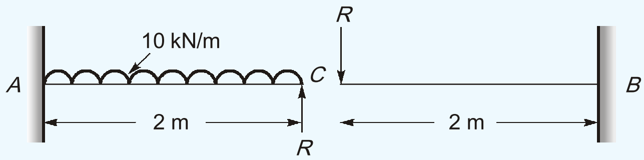
End of Solution

Q.1 (b) Draw the bending moment and shear force diagram for the beam shown below:



[12 marks : 2025]

Solution:



For AC Portion:

Deflection at point 'C'

$$(\Delta_c)_{AC} = \frac{wl^4}{8EI} - \frac{Rl^3}{3EI} \quad \text{...(i)}$$

For CB Portion:

Deflection at point 'C'

$$(\Delta_c)_{CB} = \frac{Rl^3}{3EI} \quad \text{...(ii)}$$

From equation (i) and (ii)

$$(\Delta_C)_{AC} = (\Delta_C)_{CB}$$

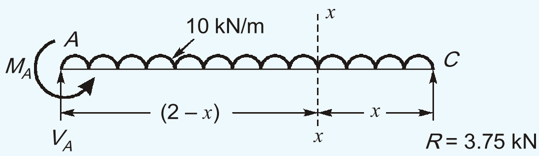
$$\Rightarrow \quad \frac{wl^4}{8EI} - \frac{Rl^3}{3EI} = \frac{Rl^3}{3EI}$$

$$\Rightarrow \quad \frac{wl^4}{8EI} = \frac{2Rl^3}{3}$$

$$\Rightarrow \quad \frac{10 \times 2}{8} = \frac{2R}{3}$$

$$\Rightarrow \quad R = 3.75 \text{ kN}$$

Now, FBD of AC portion:



$\Sigma F_y = 0, \quad \Rightarrow \quad V_A + 3.75 = 10 \times 2$

$\Rightarrow \quad V_A = 16.25 \text{ kN } (\uparrow)$

$\Sigma M_A = 0, \quad \Rightarrow M_A + 3.75 \times 2 - 10 \times 2 \times 1 = 0$

$\Rightarrow \quad M_A = 12.5 \text{ kNm} \quad \text{(Hogging)}$

Consider a section x-x, at a distance x from point C,

Bending moment at x-x section,

$$M_{xx} = 3.75x - \frac{10x^2}{2}$$

$$\Rightarrow \quad M_{x\text{-}x} = 3.75x - 5x^2$$

For position of zero bending moment,

$$3.75x - 5x^2 = 0$$

$\Rightarrow$ $x = 0$ and 0.75 m (from C)

For maximum bending moment

$$V = \frac{dM_{xx}}{dx} = 0$$

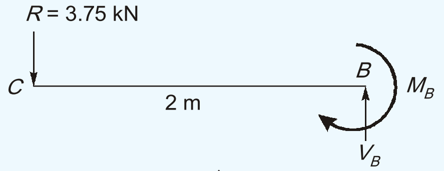
$\Rightarrow$ $3.75 - 10x = 0$

$\Rightarrow$ $x = 0.375$ m (from C)

Maximum bending moment

$$(M_{max})_{x = 0.375\text{ m}} = 3.75 \times 0.375 - 5 \times (0.375)^2$$
$$= 0.703 \text{ kNm (Sagging)}$$

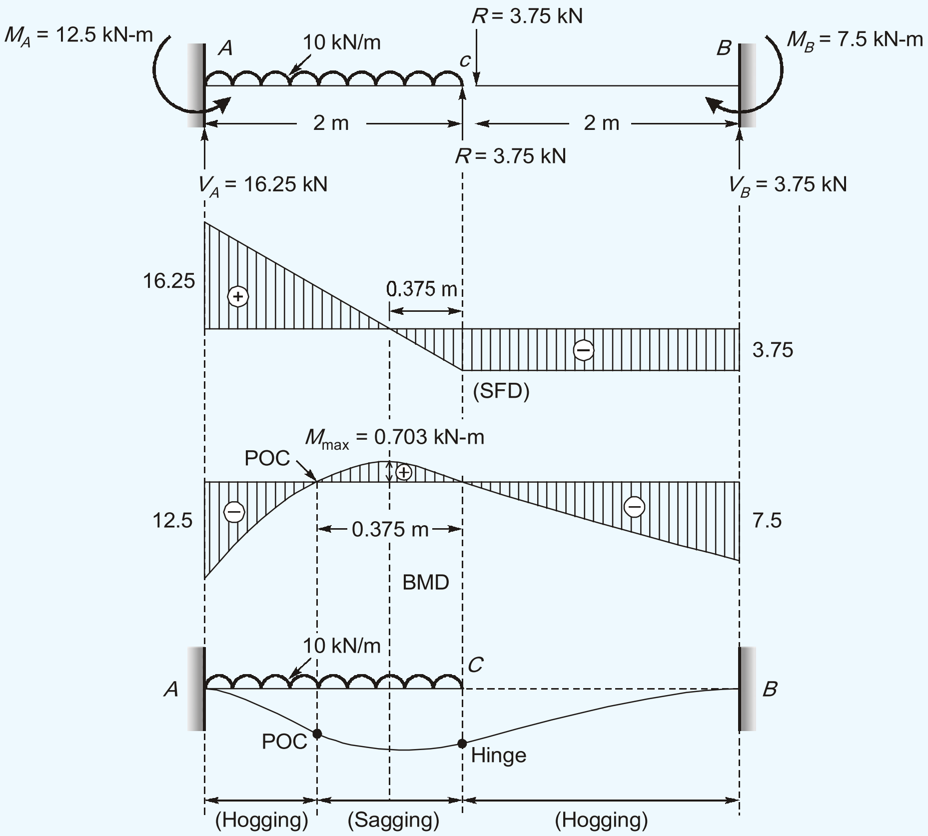
FBD of CB portion:



$\Sigma F_y = 0,$ $\Rightarrow$ $V_B = 3.75$ kN (↑)

$\Sigma M_B = 0,$ $\Rightarrow$ $M_B = 3.75 \times 2$

$M_B = 7.5$ kNm (Hogging)



Deflected shape

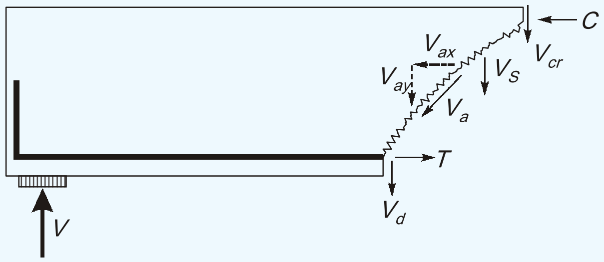
End of Solution

Q.1 (c) With the help of a sketch, briefly explain the major shear transfer mechanisms in a reinforced concrete beam, having shear reinforcement. Indicate the internal forces acting at a flexural-shear crack.

[12 marks : 2025]

Solution:

- **Major shear transfer mechanisms**: Shear in reinforced concrete beam is transferred between adjacent section through several mechanisms. These mechanisms are illustrated in figure, which shows a free body diagram of a beam segment separated by a flexural shear crack.



(Internal faces acting at a flexural shear crack)

The external shear force V, typically reaches its maximum value near the beam support (equal to the reaction force).

This shear is resisted by different mechanisms.

1. Shear resistance of uncracked portion of concrete (V_{cr}).
2. Vertical component of the force due to aggregate interlocking (V_{ay}).
3. Dowel force in the tension reinforcement (V_d).
4. Shear resistance carried by transverse shear reinforcement (V_s) if shear reinforcement is provided.

Thus, at equilibrium:

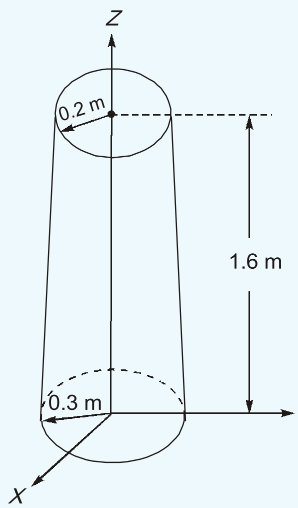
$$V = V_{cr} + V_{ay} + V_d + V_s$$

The relative contribution of the various components of the shear resisting forces depends on loading, extent of cracking, material type etc. Before the commencement of flexural cracking, the applied/superimposed shear is resisted entirely by the uncracked concrete (V_{cr}). As the concrete cracking commences, redistribution of stresses take place resulting in development of aggregate interlock force (V_{ay} vertical component) and dowel force V_d. As the commencement of diagonal tension crack occurs, shear reinforcement intercepts the cracks and undergoes a sudden increase in tensile stress and strain. At this stage, all the four major components which resist external shear are very much effective. The resulting behavior of concrete including the ultimate shear failure depends on the mode of shear transfer mechanism and on how the forces in shear resisting elements get redistributed.

Presence of longitudinal reinforcement in the flexural tension zone adds to dowel action (V_d) and also controls the propagation of flexural cracks. It also contributes in increasing the depth of neutral axis and thus the depth of uncracked concrete section increases in compression. This increases the contributions of V_{ay} and V_{cr}. This implies that, higher the percentage of tension reinforcement, greater is the shear resistance.

End of Solution

Q.1 (d) A pedestal in the shape of a frustum of a cone is made of concrete having a specific weight of 24 kN/m^3. Determine the average normal stress acting in the pedestal at its base:

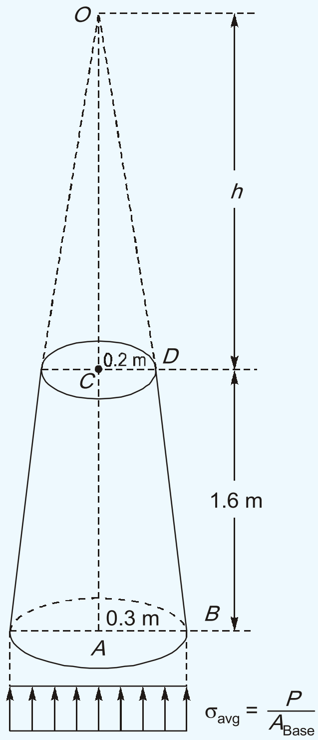


[12 marks : 2025]

Solution:

Given: Specific weight of concrete, $\gamma_C = 24$ kN/m^3

Average normal stress at base, $\sigma_{avg} = ?$



ΔBAO and ΔDCO are similar

$$\therefore \qquad \frac{CD}{OC} = \frac{AB}{AO}$$

$$\Rightarrow \quad \frac{0.2}{h} = \frac{0.3}{1.6+h}$$

$$\Rightarrow \quad 0.2 \times 1.6 + 0.2h = 0.3h$$

$$\Rightarrow \quad h = 3.2 \text{ m}$$

$$\text{Volume of frustum, } V = \frac{1}{3}\pi(0.3)^2 \times (1.6+3.2) - \frac{1}{3}\pi(0.2)^2 \times 3.2$$

$$\Rightarrow \quad V = \frac{\pi}{3}\left(0.3^2 \times 4.8 - 0.2^2 \times 3.2\right)$$

$$\Rightarrow \quad V = 0.318348 \text{ m}^3$$

Now, total load acting at base

$$P = \gamma_c(\text{Volume of frustum})$$

$$\Rightarrow \quad P = 24 \times 0.318348 \text{ kN}$$

$$\Rightarrow \quad P = 7.64 \text{ kN}$$

Average normal stress acting in the pedestal at base

$$\sigma_{avg} = \frac{P}{A_{Base}}$$

$$\Rightarrow \quad \sigma_{avg} = \frac{7.64}{\frac{\pi}{4}(0.6)^2}$$

$$\Rightarrow \quad \sigma_{avg} = 27.021 \text{ kN/m}^2$$

$$\Rightarrow \quad \sigma_{avg} = 2.7 \text{ N/cm}^2$$

End of Solution

Q.1 (e) A four-wheel tractor weighing 18000 kg has weight distribution between the front and the rear wheels of 40 percent and 60 percent respectively. It is operating on a level haul road whose rolling resistance is 45 kg/ton. What is the maximum rimpull of the tractor if the coefficient of traction between the road surface and the tyre is 0.65?

[12 marks : 2025]

Solution:

Total weight of tractor, $W_{total} = 18000 \times 9.81 = 176580$ N

Weight on rear wheels

$$W_{rear} = 0.6 \times 176580 = 105948 \text{ N}$$

Rolling resistance = 45 kg/ton (given)

Tractor weight = 18000 kg = 18 tons

Rolling resistance force = 45 × 18 = 810 kg

= 810 × 9.81 = 7946.1 N

Maximum tractive effort = Coefficient of traction × W_{rear}

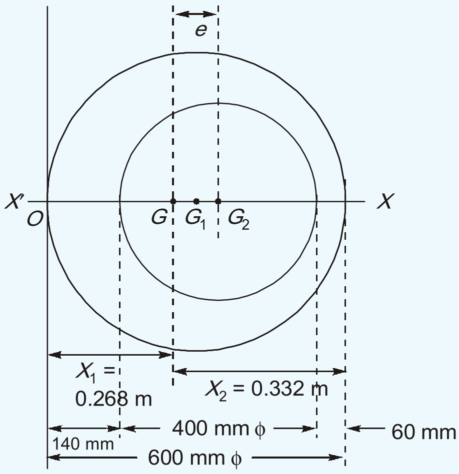
$$\Rightarrow \quad F_{tractive} = 0.65 \times 105948 = 68866.2 \text{ N}$$

Maximum rimpull = Gross rimull – Rolling resistance

= 68866.2 N – 7946.1 N = 60920.1 N = 6210 kg

End of Solution

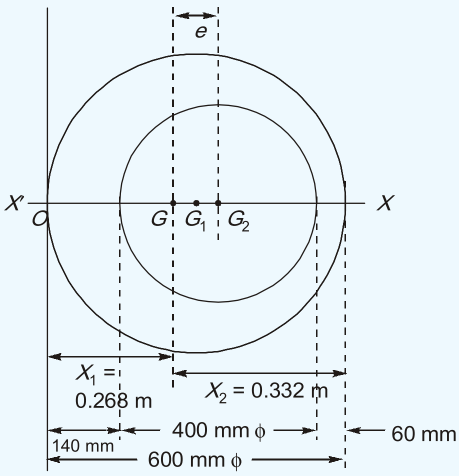
Q.2 (a) A short hollow cast iron column having an external diameter of 600 mm and inside diameter 400 mm was cast in a factory. On inspection, it was found that the bore is eccentric as shown in the figure below. If the column carries a load of 2000 kN along the axis of the bore, calculate the extreme intensities of stresses induced in the section.



[20 marks : 2025]

Solution:

Given: $D_1 = 600$ mm, $D_2 = 400$ mm, $P = 2000$ kN



From diagram,
$$GG_1 = OG_1 - X_1$$
$$GG_1 = 300 - 268 = 32 \text{ mm}$$

Now,
$$GG_2 = X_2 - 200 - 60$$

$\Rightarrow$
$$GG_2 = 332 - 260 = 72 \text{ mm} = e$$

$$\text{Area, } A = \frac{\pi}{4}\left(600^2 - 400^2\right) = 157079.6327 \text{ mm}^2$$

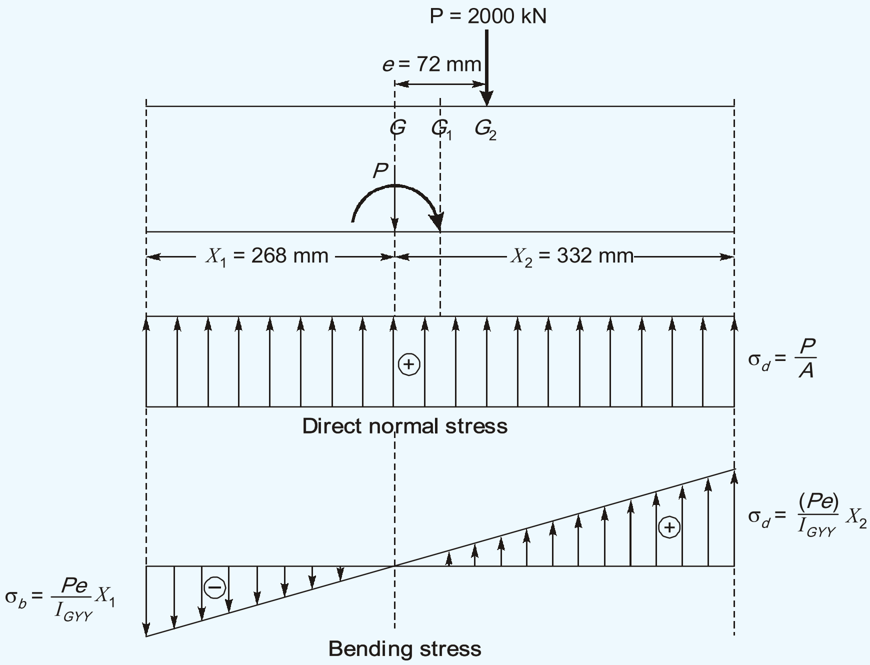
Area moment of inertia about centroidal y-axis is,

$$I_{Gyy} = \left[\frac{\pi}{64}D_1^4 + \frac{\pi}{4}D_1^2(GG_1)^2\right] - \left[\frac{\pi}{64}D_2^4 + \frac{\pi}{4}D_2^2(GG_2)^2\right]$$

$$\Rightarrow \quad I_{Gyy} = \left(\frac{\pi}{64}(600)^4 + \frac{\pi}{4}(600)^2 32^2 - \frac{\pi}{64}\times 400^4 - \frac{\pi}{4}\times 400^2 \times 72^2\right)$$

$$\Rightarrow \quad I_{Gyy} = 474.318 \times 10^7 \text{ mm}^4$$

Now,



Maximum stress:

$$\sigma_{max} = \sigma_d + \sigma_b = \frac{P}{A} + \frac{Pe}{I_{Gyy}}X_2$$

$$\Rightarrow \quad \sigma_{max} = \frac{2000\times 10^3}{157079.6327} + \frac{2000\times 10^3 \times 72}{474.318\times 10^7}\times 332$$

$$\Rightarrow \quad \sigma_{max} = 22.81 \text{ MPa (Compressive)}$$

$$\text{Minimum stress, } \sigma_{min} = \sigma_d - \sigma_b = \frac{P}{A} - \frac{Pe}{I_{Gyy}}X_1$$

$$\Rightarrow \quad \sigma_{min} = \frac{2000\times 10^3}{157079.6327} - \frac{2000\times 10^3 \times 72}{474.318\times 10^7}\times 268$$

$$\Rightarrow \quad \sigma_{min} = 4.596 \text{ MPa (Compressive)}$$

End of Solution

Q.2 (b) Explain the following terms:

(i) Autogenous shrinkage
(ii) Bogue compounds
(iii) Case-hardening
(iv) Elastomers
(v) Guniting
(vi) Scoriaceous aggregate
(vii) Self-desiccation
(viii) Shingling
(ix) Puddling
(x) Wet rot

[2 × 10 = 20 marks : 2025]

Solution:

(i) **Autogenous shrinkage:** It is the reduction in volume of concrete, mortar or cementitious paste caused by the consumption of water during hydration. It does not include volume change due to loss or ingress of substances, temperature variation and application of an external force. It happens mainly in high strength concrete with low water cement ratio. This shrinkage develops internally in the whole volume of concrete and is sometimes referred to as self-desiccation concrete.

(ii) **Bogue compounds:** These are the four principal chemical constituents of cement clinker formed during the burning process where different ingredients of cement combine and form clinkers. They have the properties of setting and hardening in the presence of water, thus govern such characteristics of cement. There are four such compounds:

1. Alite (Tricalcium silicate) provides early strength.
2. Belite (Dicalcium silicate) develops strength at later ages.
3. Celite (Tricalcium aluminate) affects initial setting and heat evaluation.
4. Felite (Tetracalcium alumino ferrite) has minor cementing value.

(iii) **Case-hardening:** It is a defect in timber that occurs when outside of timber becomes hard and dry while the inner core remains moist. The exposed surface of the timber dries very rapidly and therefore shrinks and is under compression while the interior is under tension. It mostly occurs during kiln seasoning.

OR

Case hardening: It is a surface hardening process in which low-carbon steel is heated in a carbon-rich environment, causing carbon to diffuse into the surface. This increases surface carbon content, making it hard and wear resistant, while the core remains soft and ductile. It is used for making parts like gears, bearings etc.

(iv) **Elastomers:** These are polymers with high elasticity, capable of extending several times their original length under an applied load and returning to original dimensions upon load removal. They have low glass transition temperature, flexible long chain molecules, and are lightly cross linked to retain shape. They are best used in shock-absorbing mounts, seals etc.

Eg. Natural rubber (polyisoprene), SBR, CR, NBR etc.

(v) **Guniting:** It is the process of applying mortar or concrete under pneumatic pressure through a cement gun onto a surface to repair or protect it. The mortar mix (cement + sand) is conveyed in dry form and water is added at the nozzle, projecting the mix at high velocity to produce a dense, strong layer with excellent bond. Coarser particles rebound, leaving a fine, well-compacted surface. In the process, a thin layer of grout builds up and acts like a cushion reducing the percentage rebound in the successive layers. It is commonly used for construction of thin sections, repairing of deteriorated concrete, stabilising rocks and earth slopes.

(vi) **Scoriaceous aggregate:** They refer to lighteight, porous pieces of volcanic rock called scoria, used in concrete and construction. Scoria is characterized by its vesicular texture, dark colour and relatively high porosity, giving it a rough, sponge-like appearance. They are lighter than normal-weight aggregates and are valued for producing lightweight concrete due to its low density and ability to improve thermal insulation.

(vii) **Self-desiccation:** It is the process where concrete undergoes internal drying due to the hydration of cement, leading to a reduction in internal relative humidity without external moisture loss. This phenomenon occurs when the water inside the concrete is consumed by chemical reactions (mainly hydration) and insufficient water remains to keep the pores saturated. It can result in early age cracking and increased autogenous shrinkage, impacting the durability and performance of concrete structures.

(viii) **Shingling:** It is the process of removing slag from puddle balls or wrought iron blooms while still in the red-hot condition, by forging under a power hammer or squeezing between grooved cylinders. It helps to bind and weld the iron particles together before further shaping.

(ix) **Puddling:** It is the process of converting pig iron into wrought iron by stirring it in a molten state to oxidise impurities such as carbon, silicon and managanese. The oxidization is aided by the furnace lining and added oxidising agents. During stirring, impurities form slag, which is removed, and the purified iron gathers in the form of white spongy iron balls known as puddle balls.

(x) **Wet rot:** It is a fungal decay of timber caused by exposure to moisture, where wood becomes damp and the fungus breaks down its structure, resulting in soft, weak and spongy wood. Its tendency increases due to alternate wetting and drying and in unseasoned timber. It occurs due to leaks, poor ventilation or rising damp. It is localised to damp areas and leads to loss of strength.

End of Solution

Q.2 (c) Design a suitable double-angle discontinuous strut in a steel truss to carry a working axial compressive load of 200 kN. The effective length of the strut is 2·12 m. Use a gusset plate of 20 mm thick. Assume column buckling class c.

f_y = 250 MPa, f_u = 400 MPa, γ_{mb} = 1.25.

Relevant portion of the code books is enclosed.

[20 marks : 2025]

Solution:

Given: $P_{working} = 200$ kN

$P_{factored} = 1.5 \times 200$ kN $= 300$ kN

$l_{eff} = 2.12$ m, $f_y = 250$ MPa, $f_u = 400$ MPa, $\gamma_{mb} = 1.25$

Assume: Slenderness ratio

$\lambda = 110$

From Table 9(C) clause 7.1.2.1

$f_{cd} = 94.6$ N/mm^2

$$\text{Cross-section area required} = \frac{300 \times 10^3}{94.6} = 3171.247 \simeq 3172 \text{ mm}^2$$

$$\text{For single angle, area required} = \frac{3172}{2} = 1586 \text{ mm}^2$$

Try 2ISA 90 × 90 × 10

Area provided, $A = 1703$ mm^2

Minimum radius of gyration, $r = r_z = 27.3$ mm (From steel table)

Effective length, $l_{eff} = 2120$ m

Now, $$\lambda = \frac{l_{eff}}{r_z} = \frac{2120}{27.3} = 77.66$$

For $\lambda = 77.66$, $f_y = 250$ MPa and buckling curve c, the design compressive stress from table 9(C) is,

$$f_{cd} = 152 + \frac{(136-152)}{(80-70)}(77.65-70) = 139.76 \text{ N/mm}^2$$

Design compressive strength,

$P_d = A_{provided} \times f_{cd} = 2 \times 1703 \times 139.76$ N

$P_d = 476.02$ kN > 300 kN (OK)

Provide 2 ISA 90 × 90 × 10 mm on opposites sides of the gusset

Design of connection:

Use 20 mm diameter bolts, $d = 20$ mm

Diameter of bolt hole, $d_o = 22$ mm

Design shearing strength of bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$$

$$\Rightarrow \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times \left(1.78 \times \frac{\pi}{4} \times 20^2\right) N$$

$$\Rightarrow \quad V_{dsb} = 103.314 \text{ kN}$$

Design bearing strength of bolt

$$V_{dPb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$$

End distance $= 1.5 d_0 = 33$ mm

$$\text{Pitch, } p = 2.5d = 50 \text{ mm}$$

$$\text{Provide, } e = 40 \text{ mm}, \; p = 50 \text{ mm}$$

$$k_b = \min\left\{\frac{e}{3d_0}, \left(\frac{p}{3d_0} - 0.25\right), \frac{f_{ub}}{f_u}, 1\right\}$$

$$\Rightarrow \quad k_b = \min\left\{\frac{40}{3\times 22}, \left(\frac{50}{3\times 22} - 0.25\right), \frac{400}{410}, 1\right\}$$

$$\Rightarrow \quad k_b = \min\{0.606, 0.507, 0.975, 1\}$$

$$\therefore \quad k_b = 0.507$$

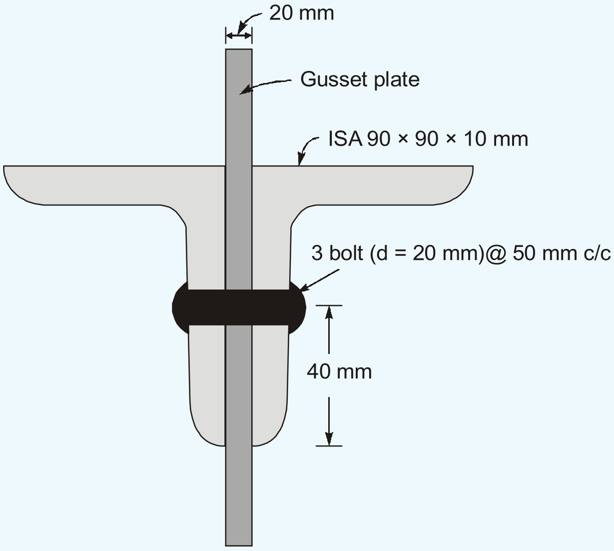
Now,
$$V_{dPb} = \frac{2.5\times 0.507\times 20\times 20\times 410}{1.25} \text{N}$$

$$V_{dPb} = 166.296 \text{ kN}$$

$$\text{Bolt value, } V_{db} = \min.\{V_{dsb}, V_{dPb}\} = 103.314 \text{ kN}$$

$$\text{Required number of bolts} = \frac{P_u}{V_{dp}} = \frac{300}{103.314} = 2.903 \simeq 3 \text{ (say)}$$

Provide 3 bolts of 20 mm diameter @ 50 mm c/c

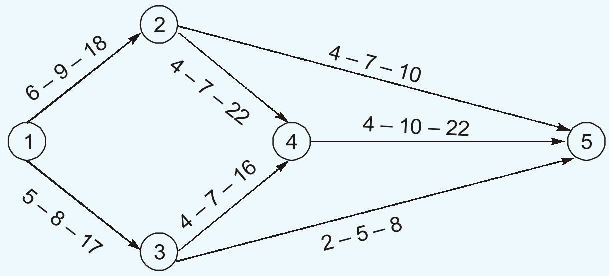


End of Solution

Q.3 (a) For the network shown below, the time estimates (in days) for each activity are mentioned. Determine the probability of completing the project in 35 days.
Given:

Standard normal distribution function

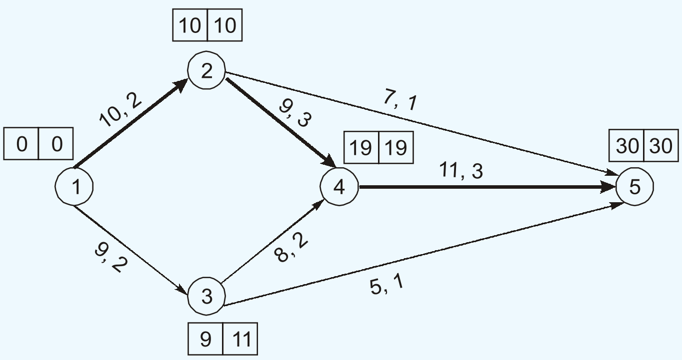
Z	P% (% probability)
0.8	78.51
0.9	81.59
1.0	84.13
1.1	86.43
1.2	88.49



[20 marks : 2025]

Solution:

Activity	Time duration (in days)			Expected duration $t_e = \left(\frac{t_o + 4t_m + t_p}{6}\right)$	Standard deviation $\sigma = \frac{t_p - t_p}{6}$
	Optimistic (t_o)	Most likely (t_m)	Pessimistic (t_e)		
1 – 2	6	9	18	10	2
1 – 3	5	8	17	9	2
2 – 4	4	7	22	9	3
3 – 4	4	7	16	8	2
2 – 5	4	7	10	7	1
4 – 5	4	10	22	11	3
3 – 5	2	5	8	5	1



From network diagram critical path is 1– 2 – 4 – 5

Critical path duration, $T_c = 10 + 9 + 11 = 30$ days

Now, standard deviation of critical path,

$$\sigma_c = \sqrt{\sigma_{1-2}^2 + \sigma_{2-4}^2 + \sigma_{4-5}^2}$$

$$\Rightarrow \qquad \sigma_c = \sqrt{2^2 + 3^2 + 3^2}$$

$$\sigma_c = 4.6904 \text{ days}$$

Now, For scheduled time, $T_s = 35$ days

$$\text{Probability factor, } Z = \left(\frac{T_s - T_c}{\sigma_c}\right) = \frac{35-30}{4.6904}$$

$$\Rightarrow \qquad Z = 1.066$$

From given table,

For Z = 1.0, P% = 84.13

For Z = 1.1, P% = 86.43

$$P\%(Z = 1.066) = 84.13 + \left(\frac{86.43 - 84.13}{1.1 - 1.0}\right) \times (1.066 - 1)$$

$$= 85.648\%$$

End of Solution

Q.3 (b) Design a short reinforced concrete column subjected to a working axial load of 1400 kN and service moments of 60 kN-m and 40 kN-m about its major and minor axes respectively. The least cross-sectional dimension of the column shall be 300 mm. Adopt limit state design. Use M 30 concrete and Fe 500 grade steel. The effective concrete cover to longitudinal reinforcement is 60 mm. Sketch the reinforcement details.

Relevant portions of IS 456 and SP 16 are enclosed.

[20 marks : 2025]

Solution:

Given: Short reinforced column.

Factored axial load, $P_u = 1.5 \times 1400 \text{ kN} = 2100 \text{ kN}$

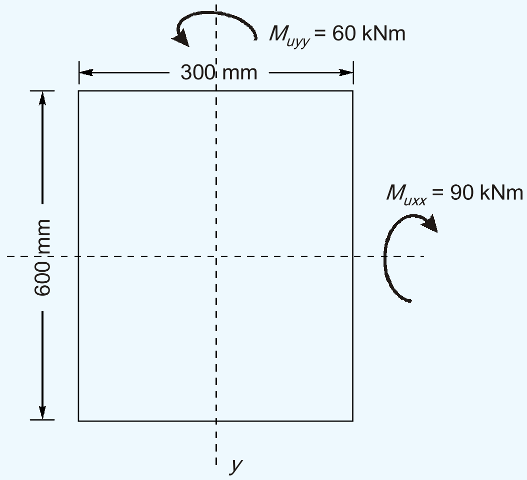
$$M_{uxx} = 1.5 \times 60 \text{ kNm} = 90 \text{ kNm}$$

$$M_{uyy} = 1.5 \times 40 \text{ kNm} = 60 \text{ kNm}$$

$$f_{ck} = 30 \text{ N/mm}^2,\ f_y = 500 \text{ N/mm}^2,\ d' = 60 \text{ mm}$$

$$D = 300 \text{ mm}$$

$$\therefore \qquad \frac{d'}{D} = \frac{60}{300} = 0.2$$



Now, $$\frac{P_u}{f_{ck}bD} = \frac{2100 \times 10^3}{30 \times 300 \times 300} = 0.78$$

It is designed on the basis of a trial section for uniaxial eccentricity considering approximately 15%.

$$\text{Design moment, } M_u = 1.15\sqrt{(M_{ux})^2 + (M_{uy})^2}$$

$$\Rightarrow \quad M_u = 1.15\sqrt{90^2 + 60^2}$$

$$\Rightarrow \quad M_u = 124.40 \text{ kNm}$$

Now, $$\frac{M_u}{f_{ck}bD^2} = \frac{124.4 \times 10^6}{30 \times 300 \times 300^2} = 0.15$$

From SP.: 16 (chart 50), equal reinforcement on all sides

$$\frac{p}{f_{ck}} = 0.22$$

$$\Rightarrow \quad p = 0.22 \times 30 = 6.6\%$$

$$\Rightarrow \quad \frac{A_{sc}}{300 \times 300} \times 100 = 6.6$$

$$\therefore \quad \text{Required steel, } A_{sc} = 5940 \text{ mm}^2$$

$$\text{Steel provide d, } A_{\text{provided}} = 16 \text{ bars - } 22 \text{ mm}\phi = 16 \times \frac{\pi}{4}(22)^2$$

$$= 6082.12 \text{ mm}^2$$

$$\simeq 6090 \text{ mm}^2 > 5940 \text{ mm}^2$$

$$\therefore \quad p = \frac{6090}{300 \times 300} \times 100 = 6.77\%$$

$$\therefore \quad \frac{p}{f_{ck}} = \frac{6.77}{30} = 0.226$$

From chart $\dfrac{M_{ux1}}{f_{ck}bD^2} = 0.163$

$\Rightarrow$ $M_{ux1} = M_{uy1}$

$= 0.163 \times 30 \times 300 \times 300^2 \text{ Nmm} = 132.03 \text{ kNm}$

Which is significantly greater than $M_{ux} = 90$ kNm and $M_{uy} = 60$ kNm

Now, $P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$

$\Rightarrow$ $P_{uz} = 0.45 \times 30 \times 300^2 + (0.75 \times 500 - 0.45 \times 30) \times 6090 \text{ N}$

$\Rightarrow$ $R_{uz} = 3416.535 \text{ kN}$

$\therefore$ $\dfrac{P_u}{P_{uz}} = \dfrac{2100}{3416.535} = 0.615$

$$\alpha_n = 1 + \frac{(2-1)}{(0.8-0.2)} \times (0.615 - 0.2) = 1.69$$

Check safety under biaxial loading

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{90}{132.03}\right)^{1.69} + \left(\frac{60}{132.03}\right)^{1.69}$$

$= 0.787 < 1$ (OK)

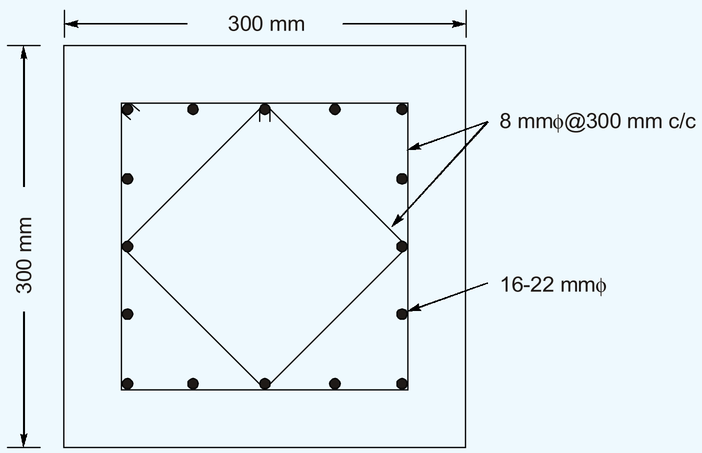
Transverse reinforcement:

The minimum diameter ϕ and maximum spacing 's_t' of lateral ties are specified by code:

$$\text{Diameter, } \phi_t > \max.\begin{cases} \dfrac{\phi_{main}}{4} = \dfrac{25}{4} = 6.25\text{mm} \\ 6\text{mm} \end{cases}$$

$\therefore$ Provide 8 mm ϕ ties.

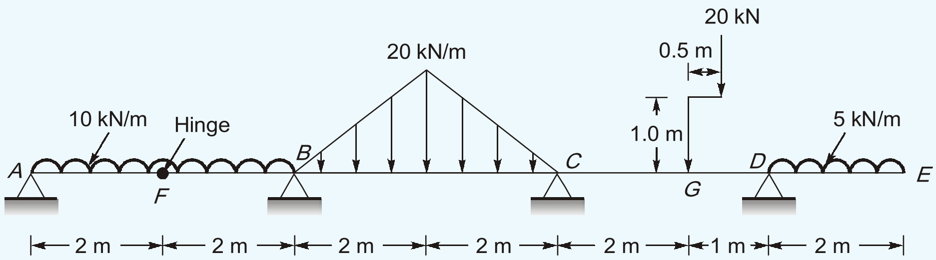
$$\text{Spacing, } s_t < \min.\begin{cases} D = 300\text{mm} \\ 16\phi = 16 \times 25\text{mm} = 400 \text{ mm} = 300 \text{ mm c/c} \\ 300\text{mm} \end{cases}$$



Clear cover $= 60 - \dfrac{22}{2} - 8 = 41 \text{ mm} > 40 \text{ mm}$ (OK)

End of Solution

Q.3 (c) For the continuous beam shown below, draw the bending moment diagram using Clapeyron's theorem (three-moment equation). The support C sinks by 1 cm. Take E = 200 GPa, I = 10000 cm^4:



[20 marks : 2025]

Solution:

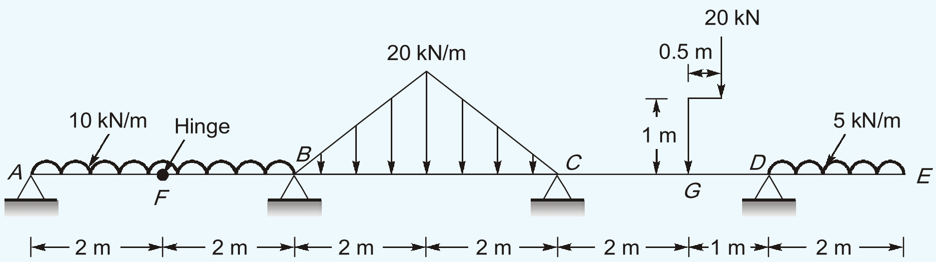
Note: In this question span DE value not given.

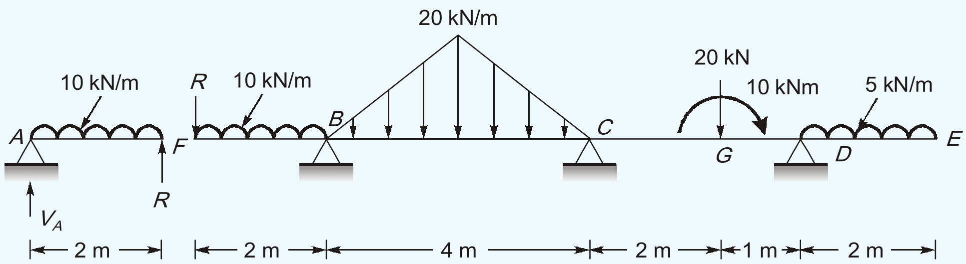
Assume: DE = 2 m

Given: E = 200 GPa = 200 × 10^6 kN/m^2

I = 10000 cm^4 = 10^{-4} m^4

EI = 20000 kNm^2





AF Portion:

$\Sigma F_y = 0 \quad \Rightarrow \qquad V_A + R = 10 \times 2$

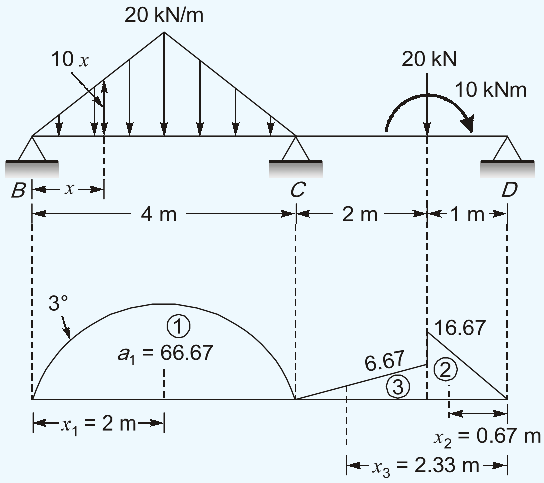
$\Sigma M_A = 0 \quad \Rightarrow \qquad R \times 2 = 10 \times 2 \times 1$

$\therefore \qquad V_A = R = 10$ kN

BM at B, $M_B = 10 \times 2 + 10 \times 2 \times 1 = 40$ kNm (Hogging)

BM at D, $M_D = 5 \times 2 \times 1 = 10$ kNm (Hogging)

Now,



Free BMD

$$\text{Maximum free } BM \text{ for } BC = \frac{20\times 4^2}{12} = 26.67 \text{ kNm}$$

Area of the free BM diagram for BC

$$a_1 = 2\int_0^2\left(20x-\frac{1}{2}x\times 10x\times\frac{x}{3}\right)dx$$

$$\Rightarrow \qquad a_1 = 2\int_0^2\left(20x-\frac{5}{3}x^3\right)dx = 2\left[10x^2-\frac{5}{12}x^4\right]_0^2$$

$$\Rightarrow \qquad a_1 = 2\left(10\times 2^2-\frac{5}{12}\times 2^4\right)$$

$$\Rightarrow \qquad a_1 = 66.67 \text{ kNm}$$

$$a_1 = 66.67 \text{ kNm}, \ x_1 = 2 \text{ m}$$

$$a_2 = \frac{1}{2}\times 1\times 16.67 = 8.335 \text{ kNm},$$

$$x_2 = \frac{2}{3}\times 1 = 0.67 \text{ m}$$

$$a_3 = \frac{1}{2}\times 2\times 6.67 = 6.67 \text{ kNm},$$

$$x_3 = 1+\frac{2}{3}\times 2 = 2.33 \text{ m}$$

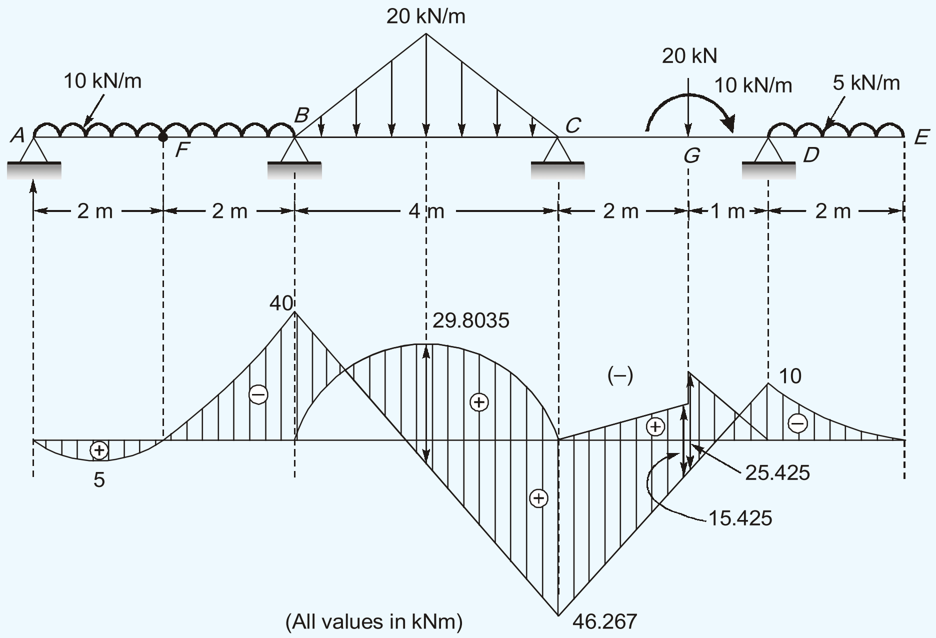
Now, applying the theorem of three moments for the span BC and CD,

$$M_B l_1 + 2M_C(l_1+l_2) + M_D l_2 = -\sum_{n=1}^{3}\left(\frac{6a_n x_n}{l_n}\right)+\frac{6EI\delta_B}{l_1}+\frac{6EI\delta_D}{l_2}$$

$$\Rightarrow \quad -40 \times 4 + 2M_C(4+3) - 10 \times 3 = \frac{-6(66.67 \times 2)}{4} - \frac{6 \times 8.335 \times 0.67}{3} - \frac{6 \times 6.67 \times 2.33}{3} + \frac{6 \times 20000 \times 0.01}{4} + \frac{6 \times 20000 \times 0.01}{3}$$

$$\Rightarrow \quad -160 + 14M_C - 30 = 457.7389$$

$$\Rightarrow \quad M_C = 46.267 \text{ kNm (Sagging)}$$



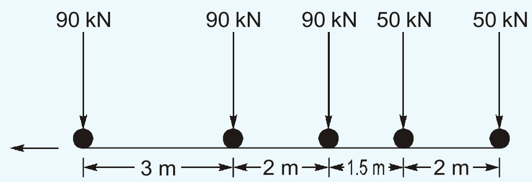
Bending moment diagram

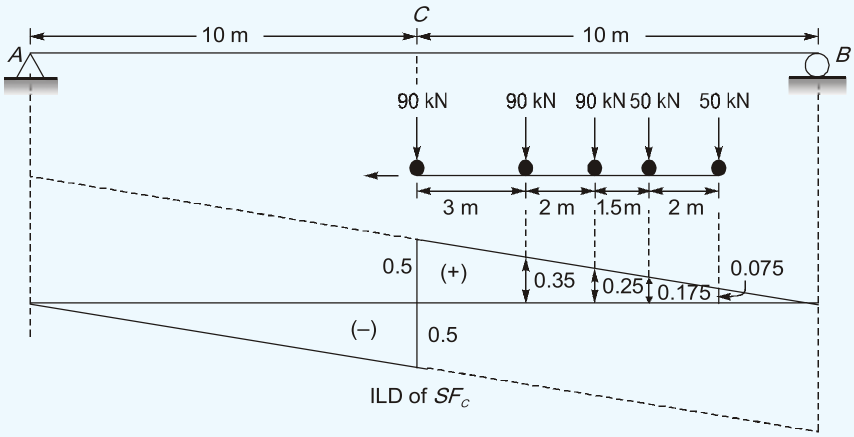
End of Solution

Q.4 (a) Five wheel loads of 90 kN, 90 kN, 90 kN, 50 kN and 50 kN magnitudes spaced 3 m, 2 m, 1.5 m and 2 m apart respectively cross a simply supported girder of 20 m span from right to left with 90 kN load leading. Calculate the maximum positive and negative shear force at the centre of the span and absolute maximum value of bending moment that occurs anywhere in the girder.

[20 marks : 2025]

Solution:





For maximum positive shear force at C:

$$\frac{\Sigma W}{l} = \frac{50+50+90+90+90}{20} = 18.5$$

$$\frac{W_1}{a_1} = \frac{90}{3} = 30 > 18.5$$

$$\therefore \quad \frac{W_1}{a_1} = \frac{\Sigma W}{l}$$

Maximum positive shear force at C occurs when load W_1 = 90 kN is placed over C (i.e. just on the right side of C)

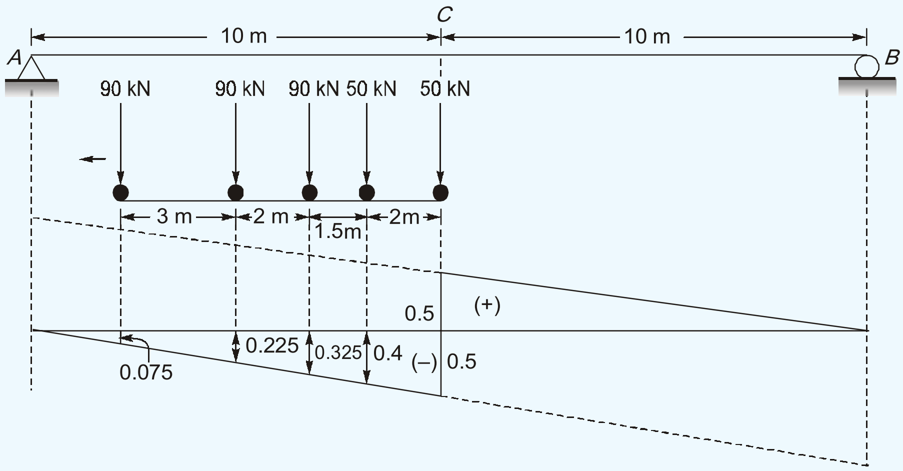
Maximum +ve SF at C = 90 × 0.5 + 90 × 0.35 + 90 × 0.25 + 50 × 0.175 + 50 × 0.075
= 111.5 kN

For maximum negative shear force at C:

$$\frac{W_5}{a_4} = \frac{50}{2} = 25 > 18.5$$

$$\therefore \quad \frac{W_1}{a_1} = \frac{\Sigma W}{l}$$

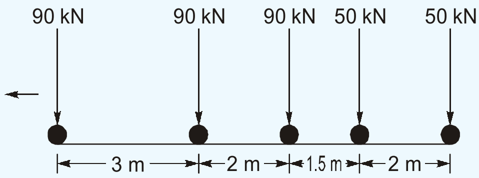
Maximum negative shear force at C occurs when load W_5 = 50 kN is placed over C (i.e. just on the left side of C)



Maximum –ve shear force at C = 50 × 0.5 + 50 × 0.4 + 90 × 0.325 + 90 × 0.225 + 90 × 0.075

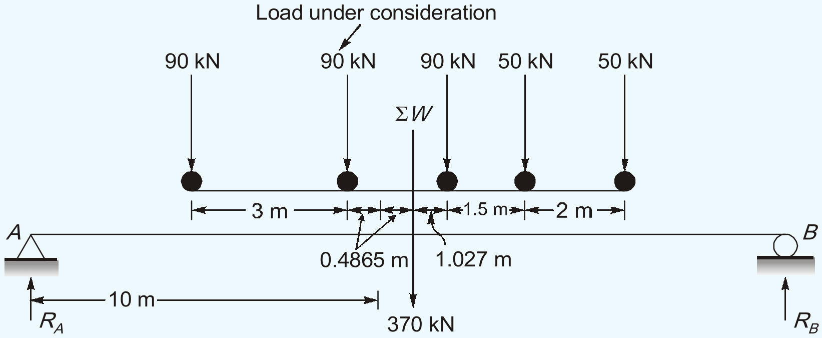
= 101.25 kN

For absolute maximum value of bending moment



$$\bar{x} = \frac{90\times3+90\times5+50\times6.5+50\times8.5}{90+90+90+50+50} = 3.973 \text{ m}$$

Case 1:



$\Sigma M_B = 0 \quad \Rightarrow \quad R_A \times 20 = 90 \times 13.4865 + 90 \times 10.4865 + 90 \times 8.4865 + 50 \times 6.9865 + 50 \times 4.9865$

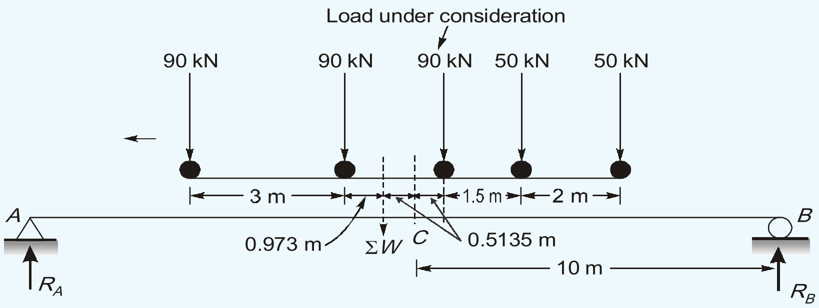
$\Rightarrow \quad R_A = 176.00025 \text{ kN} \simeq 176 \text{ kN} (\uparrow)$

Absolute maximum bending moment

$$(\text{BM})_{abs} = 176(10-0.4865)-90\times(3)$$

$$\Rightarrow \quad (\text{BM})_{abs} = 1404.376 \text{ kNm}$$

Case 2: Consider right 90 kN load under consideration:



$$\Sigma M_A = 0 \quad \Rightarrow \quad R_B \times 20 = 90 \times (10 - 3 - 0.973 - 0.5135) + 90\,(10 - 0.973 - 0.5135) + 90(10 + 0.5135) + 50(10 + 0.5135 + 1.5) + 50(10 + 0.5135 + 1.5 + 2)$$

$$\Rightarrow \quad R_B = 175.459975 \text{ kN} \simeq 175.50 \text{ kN}$$

$$\therefore \quad (BM)_{abs} = 175.50 \times (10 - 0.5135) - 50 \times 1.5 - 50 \times 3.5 = 1414.88 \text{ kNm}$$

$\therefore$ Absolute maximum BM = 1414.88 kNm at 9.4865 m from right end and under third 90 kN load (from left load)

End of Solution

Q.4 (b) The initial cost of an equipment is Rs. 1,100, salvage value is Rs.100, life of the equipment is 5 years. The rate of interest for sinking fund is 8%. Calculate the yearly depreciation and book value at the end of each year by straight line method, declining balance method, sum of years digital method and sinking fund method. Present the value in tabular form.

[20 marks : 2025]

Solution:

Given:

Initial cost of equipment, C_i = Rs. 1100

Salvage value, C_s = Rs. 100

Life of the equipment, n = 5 years

Rate of interest for sinking fund, i = 8%

Yearly depreciation of mth year = D_m = ?

Book value at the end of mth year = B_m = ?

By straight line method:

$$D_1 = D_2 = D_3 = D_4 = D_5 = \frac{C_i - C_s}{n}$$

$$= \frac{1100 - 100}{5} = \text{Rs.}200$$

Now,

$$B_1 = C_i - D_1 = 1100 - 200 = \text{Rs. } 900$$

$$B_2 = 1100 - 2 \times 200 = \text{Rs. } 700$$

$$B_3 = 1100 - 3 \times 200 = \text{Rs. } 500$$

$$B_4 = 1100 - 4 \times 200 = \text{Rs. } 300$$
$$B_5 = 1100 - 5 \times 200 = \text{Rs. } 100$$

By declining balanced method:

$$FDB = \text{Depreciation factor} = 1-\left(\frac{C_s}{C_i}\right)^{1/n}$$
$$= 1-\left(\frac{100}{1100}\right)^{1/5} = 0.381$$
$$D_1 = C_i \times FBD = 1100 \times 0.381 = \text{Rs. } 419.1$$
$$B_1 = 1100 - 419.1 = \text{Rs. } 680.9$$
$$D_2 = 680.9 \times 0.381 = \text{Rs. } 259.4$$
$$B_2 = 680.9 - 259.4 = \text{Rs. } 421.5$$
$$D_3 = 421.5 \times 0.381 = \text{Rs. } 160.6$$
$$B_3 = 421.5 - 160.6 = \text{Rs. } 260.9$$
$$D_4 = 260.9 \times 0.381 = \text{Rs. } 99.4$$
$$B_4 = 260.9 - 99.4 = \text{Rs. } 161.5$$
$$D_5 = B_4 - \text{Salvage value} = 161.5 - 100 = \text{Rs. } 61.5$$

By sum of year's digital method:

Sum of year's digits = 1 + 2 + 3 + 4 + 5 = 15

$$D_m = (C_i - C_s)\left(\frac{(n-m+1)}{\frac{n(n+1)}{2}}\right)$$

$$D_1 = (1100-100)\times\frac{5}{15} = \text{Rs.}333.3$$
$$B_1 = 1100 - 333.3 = \text{Rs. } 766.70$$
$$D_2 = (1100-100)\times\frac{4}{15} = \text{Rs.}266.70$$
$$B_2 = 766.7 - 266.7 = \text{Rs. } 500$$
$$D_3 = (1100-100)\times\frac{3}{15} = \text{Rs.}200$$
$$B_3 = 500 - 200 = \text{Rs. } 300$$
$$D_4 = (1100-100)\times\frac{2}{15} = \text{Rs.}133.30$$
$$B_4 = 300 - 133.30 = \text{Rs. } 166.7$$
$$D_5 = (1100-100)\times\frac{1}{15} = \text{Rs.}66.7$$
$$B_5 = 166.7 - 66.7 = \text{Rs. } 100$$

By sinking fund method:

$$D = (C_i - C_s)\left(\frac{i}{(1+i)^n - 1}\right)$$

$$D = (1100 - 100)\left(\frac{0.08}{1.08^5 - 1}\right) = 170.46$$

Using $D_m = D(1 + i)^{m-1}$

$D_1 = D = \text{Rs. } 170.46$

$D_2 = 170.46(1 + 0.08)^{2-1} = \text{Rs. } 184.09$

$D_3 = 170.46(1 + 0.08)^{3-1} = \text{Rs. } 198.83$

$D_4 = 170.46(1 + 0.08)^{4-1} = \text{Rs. } 214.73$

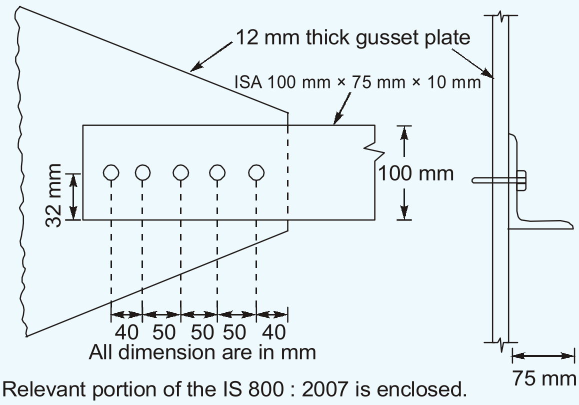
$D_5 = 170.46(1 + 0.08)^{5-1} = \text{Rs. } 231.90$

Values by all methods in tabular form.

Methods		$n=1$	$n=2$	$n=3$	$n=4$	$n=5$
Straight line method	D	200	200	200	200	200
	B	900	700	500	300	100
Declining Balanced method	D	419.1	259.4	160.6	99.4	61.5
	B	680.9	421.5	260.9	161.5	100
Sum of year's digit method	D	333.3	266.7	200	133.3	66.7
	B	766.7	500	300	166.7	100
Sinking fund method	D	170.46	184.09	198.83	214.73	231.90
	B	929.54	745.45	546.62	331.89	100

End of Solution

Q.4 (c) A single angle ISA 100 mm × 75 mm × 10 mm is connected to a gusset plate of 12 mm thick with five numbers of 16 mm diameter bolts. Determine its tensile capacity if the gusset plate is connected to the 100 mm leg. The cross-sectional area of the angle is 1650 mm^2, f_u = 410 MPa, f_y = 250 MPa, γ_{mo} = 1.1, γ_{m1} = 1.25.



Relevant portion of the IS 800 : 2007 is enclosed.

[20 marks : 2025]

Solution:

Given: ISA 100 mm × 75 mm × 10 mm

Thickness of gusset plate = 12 mm

Dia of bolt, d = 16 mm

Dia of bolt hole, d_o = 16 + 2 = 18 mm

Cross-sectional area, A_g = 1650 mm^2

$f_u = 410\ \text{N/mm}^2,\ f_y = 250\ \text{N/mm}^2,\ \gamma_{mo} = 1.1,\ \gamma_{m1} = 1.25$

Design strength due to yielding of gross section

$$T_{dg} = \frac{f_y}{\gamma_{mo}} \times A_g = \frac{250}{1.1} \times 1650\text{N} = 375\ \text{kN}$$

Design strength due to rupture of critical section,

$$T_{dn} = \frac{0.9 f_u}{\gamma_{m1}} A_{nc} + \beta\left(\frac{f_y}{\gamma_{mo}}\right) A_{go}$$

where,
$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u}{f_y}\right)\left(\frac{\gamma_{mo}}{\gamma_{m1}}\right) \ge 0.7$$

Net area of connected leg, $A_{nc} = \left(100 - 18 - \frac{10}{2}\right) \times 10 = 770\ \text{mm}^2$

Gross area of outstanding leg, $A_{go} = \left(75 - \frac{10}{2}\right) \times 10 = 700\ \text{mm}^2$

$L_c = 3 \times 50 + 40 = 190$ mm

$b_s = 75 + 32 - 10 = 97$ mm

$$\beta = 1.4 - 0.076 \times \left(\frac{75}{10}\right) \times \left(\frac{250}{410}\right)\left(\frac{97}{190}\right) \le \left(\frac{410}{250}\right)\left(\frac{1.1}{1.25}\right)$$

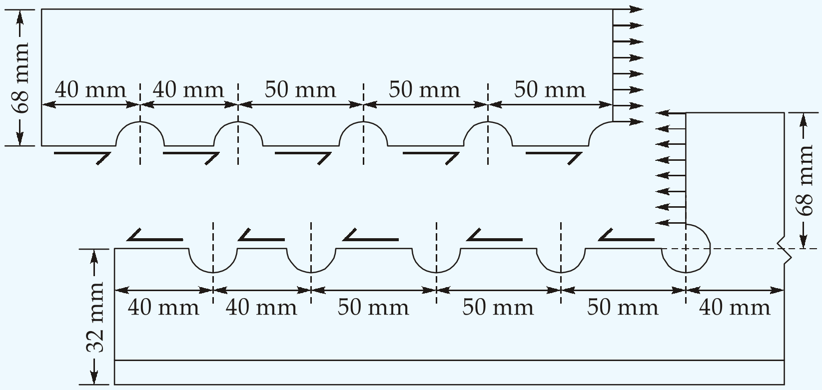
$$\ge 0.7$$

$$\beta = 1.223 \le 1.4432$$

$\because$ $0.7 < \beta < 1.4432$ ....OK

$\therefore$
$$T_{dn} = 0.9 \times \frac{410}{1.25} \times 770 + 1.223 \times \left(\frac{250}{1.1}\right) \times 700\text{N} = 421.87\ \text{kN}$$

Block shear strength,



Gross area in shear, $A_{vg} = (2 \times 40 + 3 \times 50) \times 10 = 2300\ \text{mm}^2$

Net area in shear, $A_{vn} = (230 - 4.5 \times 18) \times 10 = 1490\ \text{mm}^2$

Gross area in tension, $A_{tg} = 32 \times 10 = 680\ \text{mm}^2$

Net area in tension, $A_{tn} = (32 - 0.5 \times 18) \times 10 = 590\ \text{mm}^2$

$$T_{db1} = \frac{f_y}{\sqrt{3}\gamma_{mo}} A_{vg} + \frac{0.9 f_u}{\gamma_{m1}} A_{tn}$$

$$\Rightarrow \quad T_{db1} = \frac{250}{\sqrt{3} \times 1.1} \times 2300 + \frac{0.9 \times 410}{1.25} \times 590 = 475.96 \text{ kN}$$

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

$$\Rightarrow \quad T_{db2} = \frac{0.9 \times 410}{\sqrt{3} \times 1.25} \times 1490 + \frac{250}{1.1} \times 680 = 408.49 \text{ kN}$$

$$\therefore \quad T_{db} = \min.\{T_{db1}, T_{db2}\} = 408.49 \text{ kN}$$

$$\text{Tensile capacity, } T = \min \begin{cases} T_{dg} = 375 \text{ kN} \\ T_{dn} = 421.87 \text{ kN} \\ T_{db} = 408.49 \text{ kN} \end{cases}$$

$$\Rightarrow \quad T = 375 \text{ kN}$$

End of Solution

SECTION : B

Q.5 (a) (i) What is smart concrete? Write the key features and benefits of smart concrete.

(ii) What is self-compactng concrete? How is it obtained? Explain the advantages and disadvantages of it.

[6 + 6 = 12 marks : 2025]

Solution:

(i)

Smart concrete: It is concrete reinforced with a small volume of carbon fibres (0.2 to 0.5% by volume) to increase its ability to sense strain while still retaining good mechanical properties. Adding small amount of short carbon fibres into concrete lets it change its electrical resistivity in response to changing stress or strain. Application of stress or deformation of concrete affects the contact between fibre and cement matrix which causes measureable change in bulk electrical resistivity. Strain is detected through measurement of the electrical resistance.

Key features:

1. Detects strain, stress and onset of cracking before visible failure.
2. Real time structural health monitoring as the concrete itself works as sensor.
3. It takes greater force to bend and can absorb more energy before fracture.

Benefits:

1. Presence of carbon fibres controls cracking of concrete so the cracks do not propagate ferociously.
2. It increases cost by 30% but is still cheaper than embedding sensors into structures.

3. It is stronger than conventional concrete because of carbon fibres.
4. It can be used for measuring the weights of vehicles on the highways if used as material for pavement.
5. It can be used for real time vibration sensing of bridges.
6. It can be used in buildings to dampen vibrations and reduce earthquake damage.

(ii)

Self-compacting concrete: It is a special type of concrete characterised by high flowability that enables the concrete to spread through and around reinforcement bars, encapsulating them and filling formwork through the action of gravity without the need of any external or internal vibration, tamping etc. It deaerates by itself.

It is obtained by the usage of extra admixtures (superplasticizers and viscosity modifying admixtures). The amount of cementitious materials is kept high (about 70% of the total powder content) while water powder ratio is kept low (0.35 – 0.45). The maximum size of aggregates is generally kept as 20 mm. Aggregates used are of uniform quality regarding the shape and grading to reduce internal friction.

Advantages:

1. Better surface finish
2. Reduction in manpower at site.
3. Easier placing.
4. Faster construction.
5. Greater freedom of design.
6. Improved durability.

Disadvantages:

1. Higher material cost.
2. Requires precise mix design and quality control.
3. Sensitivity to variations in moisture content.

End of Solution

Q.5 (b) **A vibration test is conducted on the model of a tank. A cable attached to the tank induces a force of 20 kN horizontally and pulls the tank by 5 mm. The cable is cut and the resulting vibration is recorded. At the end of four complete cycles, the time elapsed is 2 seconds and the amplitude is 0.5 mm. Compute the damping ratio, natural period of vibration (undamped), stiffness and damping coefficient. Also find the number of cycles required for the displacement amplitude to decrease to $\left(\frac{1}{10}\right)$th of the initial amplitude.**

[12 marks : 2025]

Solution:

Given: Horizontal force, F = 20 kN

Initial amplitude, A_1 = 5 mm

Final amplitude, A_f = 0.5 mm

Number of cycles, n = 4

Damping ratio, ξ = ?

Natural period of vibration (undamped), $T_n = ?$
Damping coefficient, $C = ?$

$$\text{Damping ratio, } \xi = \frac{\log_e (A_1/A_f)}{2\pi n}$$

$$\Rightarrow \quad \xi = \frac{\log_e\left(\frac{5}{0.5}\right)}{2\pi \times 4} = 0.0916$$

Now, Total time for 4 cycles, $T = 2$ sec

$$\text{Natural period of vibration (undamped), } T_n = \frac{T}{n} = \frac{2}{4} = 0.5 \text{ sec}$$

$$\text{Natural frequency, } \omega_n = \frac{2\pi}{T_n} = \frac{2\pi}{0.5} = 12.57 \text{ rad/sec}$$

Damping coefficient

$$C = 2\xi\omega_n m = 2 \times 0.0916 \times 12.57 \times \left(\frac{20 \times 10^3}{9.81}\right)$$

$$= 4694.85 \text{ N sec/m}$$

Number of cycles required for the displacement aplitude to decrease to $\left(\frac{1}{10}\right)^{th}$ of the initial amplitude is given by

$$n = \frac{\log_e\left(\frac{5}{5/10}\right)}{2\xi} = \frac{\log_e(10)}{2 \times 0.0916}$$

$$\Rightarrow \quad n = 12.57 \text{ cycles}$$

End of Solution

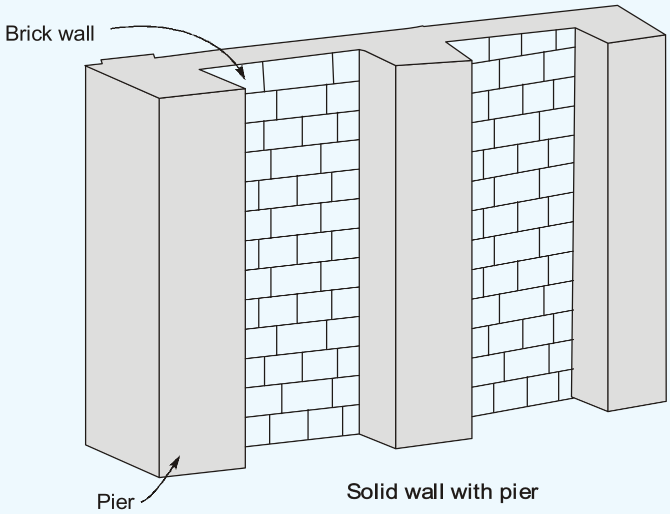
Q.5 (c) Explain the following terms, with sketches, pertaining to the masonry walls :
(i) Solid wall with piers
(ii) Cavity wall
(iii) Faced wall
(iv) Veneered wall

[12 marks : 2025]

Solution:

(i) **Solid wall with piers:** It is a masonry wall where main wall surface is continuous (solid), but at interval along its length, vertical masonry projections called piers are built integrally with the wall.

- These piers, project from wall face (usually from one or both sides)
- These piers act like buttresses increasing the wall's strength. These also help in resisting lateral load such as wind or earth pressure ultimately increasing stability in long stretches of wall.

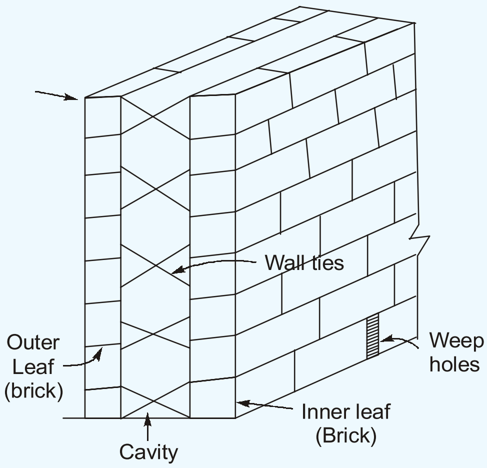


Solid wall with pier

(ii) **Cavity wall:** A cavity wall is a type of wall construction that consists of two separate walls (called leaves or skin) with a cavity/gap between them.

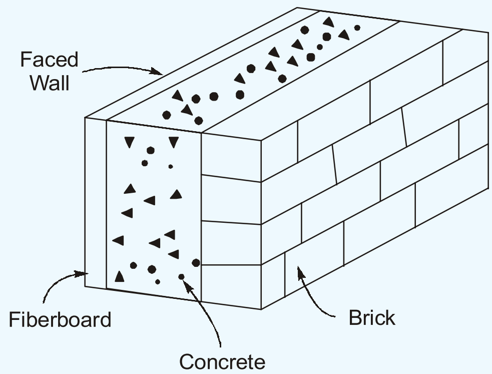
Construction detail:

1. **Outer leaf:** Usually made up of brick or stone; protects against weather.
2. **Inner leaf:** Usually made up of concrete block; provides structural strength and insulation.
3. **Cavity**: Air space (50-100 mm wide) improves thermal insulation.
4. **Wall ties:** Stainless steel connectors between leaves.
5. **DPC and weep holes**: For drainage and damp prevention; generally provided at the base.



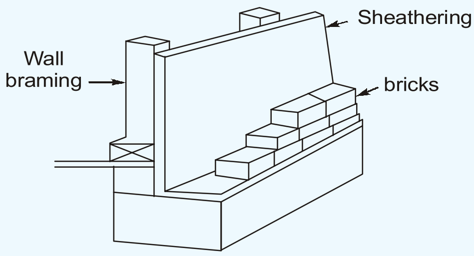
(iii) **Faced wall:** A faced wall is a type of wall in which the face (outer surface) is made of high quality material for appearance and durability while backing is made up of different, often a cheaper, material for economy.

The two parts are bonded together so they act as a unit under load ensuring structural integrity and better performance.



(iv) **Veneered wall:** It refers to a wall construction where a thin, decorative layer, (veneer) is attached to a structural substrate or backing material such as concrete, masonry, timber or engineered boards like plywood or MDF (medium density fibreboard).

- Veneers are commonly made up of material such as
  1. Brick veneer/masonry veneer.
  2. Wood veneer



Veneered wall

End of Solution

Q.5 (d) (i) List the factors that affect strength of a steel column.

(ii) Briefly explain the possible modes of failure of axially loaded steel column.

[4 + 8 = 12 marks : 2025]

Solution:

(i)

Factors affecting strength of steel columns:

1. **Material properties :** Chemical composition including alloying elements such as carbon, manganese, nickel, vanadium, chromium and titanium can affect steel's yield strength.
2. **Geometry and cross section**: Shape and dimension of column cross section play a vital role. Hollow or solid sections, width, thickness and overall size, all impact its load carrying capacity.
3. **End connection and effective length**: The way column is supported at its ends (pinned, fixed or partially restrained) affects its effective length and critical buckling load.
4. **Load characteristics:** Type of load (axial, eccentric or combined) influences stresses and potential for adding bending moment or failure modes.
5. **Environmental factors**: Corrosive environment may weaken the load carrying capacity of steel over time.

(ii)

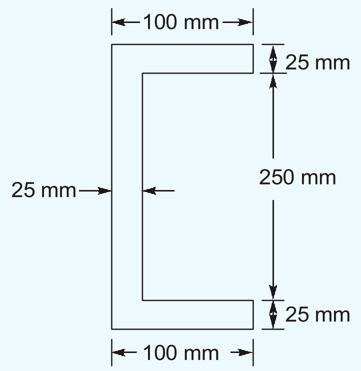
Axially loaded steel column can fail in several distinct modes, primarily influenced by their slenderness, shape and loading conditions.

The most common failure modes are:

1. **Squashing (crushing)**: Occurs in short column where load exceeds yield strength of material leading to compression failure without significant lateral deformations.
2. **Overall flexural thermal buckling**: Hapens typically in slender columns under compressive loads. Columns buckle laterally in plane of its weaker axis.
3. **Local buckling**: Happens when individual plate elements buckle before overall column fails. This is more likely if cross section elements are thin.
4. **Torsional and flexural torsional buckling**: In sections lacking symmetry, column may twist about longitudinal axis or combined twisting and bending leading to failure.
5. **Shear failures**: Occurs in steel column subjected to large transverse loads.

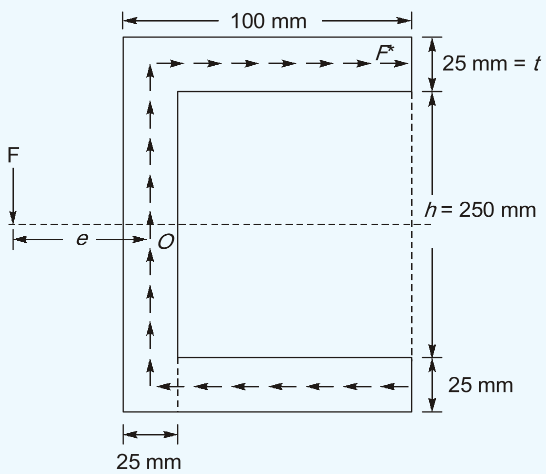
End of Solution

Q.5 (e) If the vertical shearing force acting on the thin-walled channel section shown in the figure below is 200 N, compute and show the shear flow, and determine the shear centre:



[12 marks : 2025]

Solution:



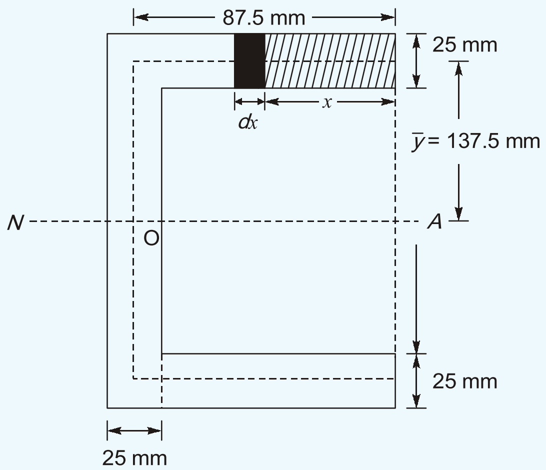
For equilibrium

Counter clockwise moment about O = Clockwise moment about O

$\Rightarrow$ $F \times e = F^* \cdot h$ ...(i)

The vertical force in web passes through O and hence moment of this force is zero.

where F^* = Shear force due to shear stress



Shear stress in flanges are equal and it is given by

$$\tau = \frac{F}{I_{NA}b}(A\bar{y}) \quad \text{...(ii)}$$

where, F = Applied shear force on the section

Consider a length x and thickness dx from tip of the flange as shown in figure.

Area of flange upto length x, $A = 25x$

Distance of C.G. of area, A from NA, $\bar{y} = \frac{250}{2} + \frac{25}{2} = 137.5$ mm

Moment of inertia of complete section about NA,

$$I_{NA} = 2\left[\frac{100 \times 25^3}{12} + 100 \times 25(137.5)^2\right] + \frac{25 \times 250^3}{12}$$

$\Rightarrow$ $I_{NA} = 127.34375 \times 10^6$ mm^4

Total shear force in each flange

$$F^* = \int \tau dA = \int_0^{87.5} \frac{F \times 25x \times (137.5)}{127.34375 \times 10^6 \times 25} \times (25dx)$$

$\Rightarrow$ $$F^* = F(2.699 \times 10^{-5}) \times \left(\frac{87.5^2}{2}\right)$$

$\Rightarrow$ $F^* = 0.103321F$

From equation (i)

$$F \times e = 0.103321F \times \left(250 + \frac{25}{2} + \frac{25}{2}\right)$$

$\Rightarrow$ $e = 28.41$ mm (From mid of web)

Alternate solution for shear center

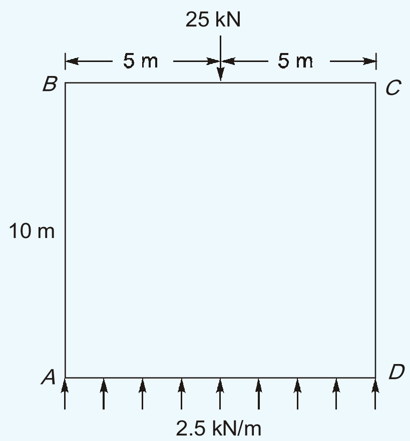
$$e = \frac{b^2h^2t}{4I_{NA}}$$

$$\Rightarrow \quad e = \frac{87.5^2 \times 275^2 \times 25}{4 \times 127.34375 \times 10^6}$$

$$e = 28.42 \text{ mm (from mid of web)}$$

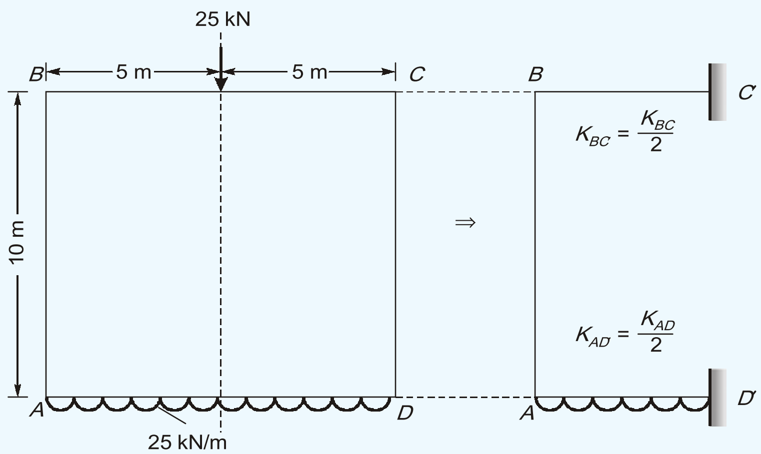
End of Solution

Q.6 (a) The culvert shown below is of constant section throughout and the top beam is subjected to a central concentrated load of 25 kN. Assuming that the base pressure is uniform throughout, analyze the box culvert. Draw the bending moment diagram using moment distribution method:



[20 marks : 2025]

Solution:



Given structure is symmetrical about vertical line passing through mid of structure.
Fixed end moments.

$$M_{FAD} = +\frac{wl^2}{12} = \frac{+2.5\times10^2}{12} = +20.83\,\text{kNm}$$

$$M_{FDA} = -\frac{wl^2}{12} = \frac{-2.5\times10^2}{12} = -20.83\,\text{kNm}$$

$$M_{FBC} = -\frac{Pl}{8} = -\frac{25\times10}{8} = -31.25\,\text{kNm}$$

$$M_{FCD} = +\frac{Pl}{8} = +\frac{25\times10}{8} = +31.25\,\text{kNm}$$

Moment distribution factor:

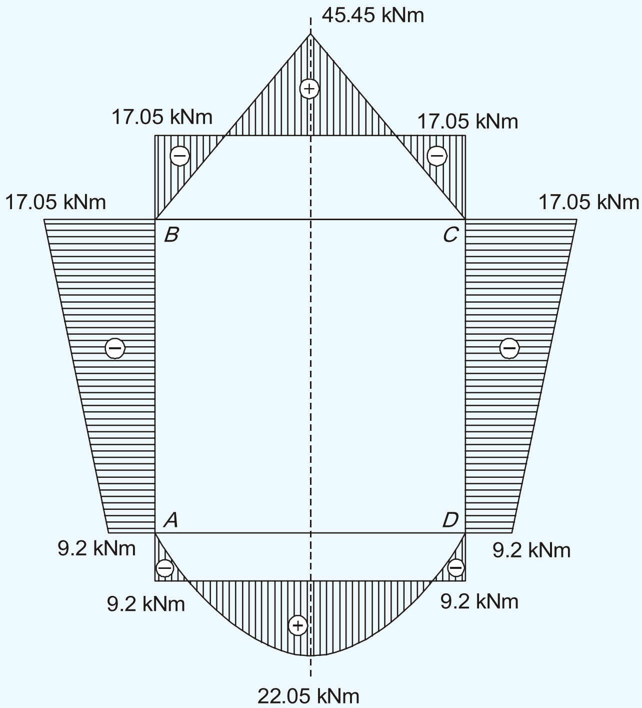
Joint	Member	Stiffness	Total stiffness	D.F.
B	BC′	$\frac{4EI}{10\times2} = \frac{EI}{5}$	$\frac{3EI}{5}$	0.33
	BA	$\frac{4EI}{10} = \frac{2EI}{5}$		0.67
A	AB	$\frac{4EI}{10} = \frac{2EI}{5}$	$\frac{3EI}{5}$	0.67
	AD′	$\frac{4EI}{10\times2} = \frac{EI}{5}$		0.33

Distribution table:

Joint	*A*		*B*	
Member	AD′	AB	BA	BC′
DF	0.33	0.67	0.67	0.33
FEM	+20.83	0	0	–31.25
Balancing	– 6.87	–13.95	20.94	10.31
COM		10.47	–6.98	
Balancing	– 3.46	–7.01	+4.68	2.3
COM		2.34	–3.5	
Balancing	– 0.77	–1.56	2.35	1.16
COM		1.18	0.78	
Balancing	–0.39	–0.79	0.62	0.25
COM		0.26	–0.395	
Balancing	– 0.0858	–0.1742	0.26	0.13
COM		0.13	–0.087	
Balancing	– 0.04	–0.09	0.058	0.0287
COM		0.020	–0.045	
Balancing	– 0.0086	–0.010	0.09	0.015
Final moment	9.2	–9.2	17.05	–17.05

$$M_{AD'} = 9.2\text{ kNm},\ M_{AB} = -9.2\text{ kNm}$$
$$M_{BA} = 17.05\text{ kNm},\ M_{BC'} = -17.05\text{ kNm}$$

Bending moment diagram:



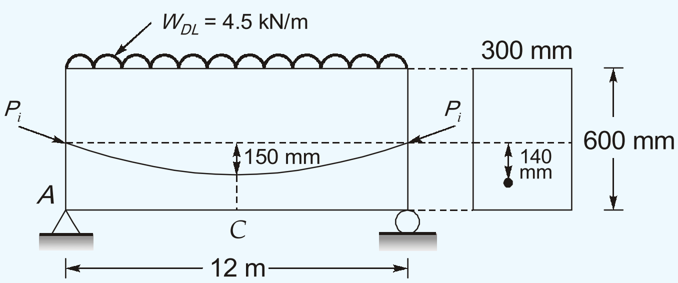
Bending moment diagram

End of Solution

Q.6 (b) A rectangular beam 300 mm × 600 mm is prestressed with parabolic cables having cross-sectional area 1200 mm^2. The parabolic cables have an eccentricity of 150 mm at mid-span and zero eccentricity at the ends. The beam is simply supported over a span of 12 m and the initial prestress in the cables is 1100 MPa. Estimate the deflection of the beam due to initial prestress plus self-weight of the beam. Assume losses of prestress as 18%. The beam is subjected to a live load of 20 kN/m over its entire span. Estimate the final deflection. Also derive only the expression for deflection due to prestress of the parabolic cable. The unit weight of concrete is 25 kN/m^3. Assume E_s = 206 kN/mm^2 and E_c = 35 kN/mm^2.

[20 marks : 2025]

Solution:



Given: $b = 300$ mm, $d = 500$ mm, $l = 12$ m, $e_{mid} = 150$ mm

$\sigma_{st} = 1100$ MPa, $A_{st} = 1200$ mm^2, $\gamma_c = 25$ kN/m^3

$E_s = 206$ kN/mm^2, $E_c = 35$ kN/mm^2

$P_i = 1100 \times 1200$ N $= 1320$ kN

Deflection due to initial prestress and self weight of beam

Self weight of beam

$$w_{DL} = \gamma_c \times \text{Volume per unit length}$$
$$= 25 \times (0.3 \times 0.6 \times 1) = 4.5 \text{ kN/m} = 0.0045 \text{ kN/mm}$$

Deflection due to self weight of beam

$$\Delta_{DL} = \frac{5w_{DL}l^4}{384E_cI}$$

$$\Rightarrow \quad \Delta_{DL} = \frac{5 \times 0.0045 \times (12000)^4}{384 \times 35 \times \left(\frac{300 \times 600^3}{12}\right)}$$

$$\Rightarrow \quad \Delta_{DL} = 6.4286 \text{ mm} \simeq 6.43 \text{ mm} (\downarrow)$$

Deflection due to initial prestress

$$D_{Pi} = \frac{5Pel^2}{48E_cI}$$

$$\Rightarrow \quad \Delta_{Pi} = \frac{5 \times 1320 \times 150 \times (12000)^2}{48 \times 35 \times \left(\frac{300 \times 600^3}{12}\right)}$$

$$\Rightarrow \quad \Delta_{Pi} = 15.714 \text{ mm} \simeq 15.71 \text{ mm} (\uparrow)$$

Net deflection at mid span,

$$\Delta_{net} = 15.71 - 6.43$$
$$\Rightarrow \quad \Delta_{net} = 9.28 \text{ mm} (\uparrow)$$

Now, beam is subjected to a live load of 20 kN/m

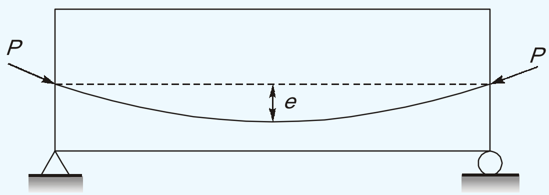
$$w_{Total} = w_{DL} + w_{LL}$$
$$w_{Total} = 4.5 + 20 = 24.5 \text{ kN/m} = 0.0245 \text{ kN/mm}$$

Final deflection

$$\Delta_{final} = \frac{-5 \times 0.0245 \times (12000)^4}{384 \times 35 \times \left(\frac{300 \times 600^3}{12}\right)} + 0.72 \times 15.71$$

$$\Rightarrow \quad \Delta_{final} = -35 \text{ mm} + 11.3112 = -23.6888 \text{ mm}$$
$$\Rightarrow \quad \Delta_{final} = 23.69 \text{ mm} (\downarrow)$$

Derivation for deflection due to prestress of the parabolic cable:



Net upward load intensity due to parabolic cable.

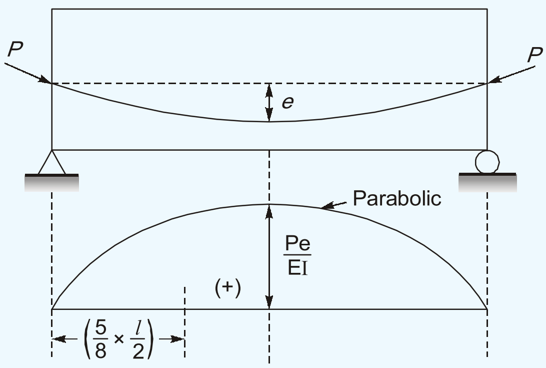
$$w_e = \frac{8Pe}{l^2}$$

$$\text{Upward deflection, } \Delta = \frac{5P_e l^2}{384 E_c I}$$

$$\Rightarrow \qquad \Delta = \frac{5l^4}{384 E_c I} \times \frac{8Pe}{l^2}$$

$$\Rightarrow \qquad \Delta = \frac{5P_e l^2}{48 E_c I}$$

Alternate solution:



By moment area method maximum upward deflection at mid-span

$$\Delta = \left(\frac{2}{3} \times \frac{l}{2} \times \frac{P.e}{E_c I}\right) \times \left(\frac{5}{8} \times \frac{l}{2}\right)$$

$$\Delta = \frac{5Pel^2}{48 E_c I}$$

End of Solution

Q.6 (c) (i) An element in a stressed material has tensile stress of 400 MN/m^2 and a compressive stress of 300 MN/m^2 acting on two mutually perpendicular planes and equal shear stresses of 80 MN/m^2 on these planes. Find the principal stresses and position of the principal planes. Find also the maximum shearing stress. Solve using analytical method.

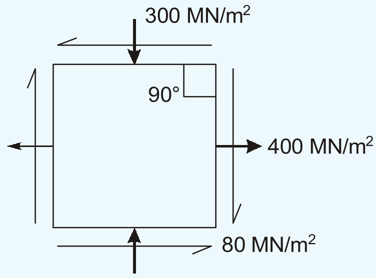
[10 marks : 2025]

(ii) A simply supported beam 150 mm wide and 220 mm deep is 4 m long and carries a load of 20 kN at mid-span. The load is inclined at an angle of 30° to the vertical. The line of action is passing through the centroid of the section. Find the locations and magnitudes of maximum tensile and compressive stresses set up due to bending.

[10 marks : 2025]

Solution:

(i)



Here $\sigma_x = 400 \text{ MN/m}^2 = 400 \text{ N/mm}^2$

$\sigma_y = -300 \text{ MN/m}^2 = -300 \text{ N/mm}^2$

$\tau_{xy} = 80 \text{ MN/m}^2 = 80 \text{ N/mm}^2$

Principal stresses:

$$\sigma_{1,2} = \left(\frac{\sigma_x + \sigma_y}{2}\right) \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

$$\Rightarrow \quad \sigma_{1,2} = \left(\frac{400-300}{2}\right) \pm \sqrt{\left(\frac{400+300}{2}\right)^2 + 80^2}$$

$$\Rightarrow \quad \sigma_{1,2} = 50 \pm 359.03$$

$$\sigma_1 = 409.03 \text{ MN/mm}^2, \ \sigma_2 = -309.03 \text{ N/mm}^2$$

Maximum shearing stress

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

$$\Rightarrow \quad \tau_{max} = \sqrt{\left(\frac{400+300}{2}\right)^2 + 80^2}$$

$$\Rightarrow \quad \tau_{max} = 359.03 \text{ N/mm}^2$$

Position of principal planes with respect to vertical plane

$$\tan(2\theta_P) = \frac{2\tau_{xy}}{(\sigma_x - \sigma_y)}$$

$$\Rightarrow \quad \tan(2\theta_P) = \frac{2\times 80}{400+300}$$

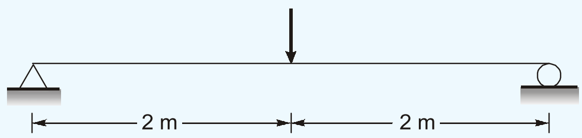
$\therefore$ $\theta_{P1} = 6.44°$ (From vertical in clockwise direction)

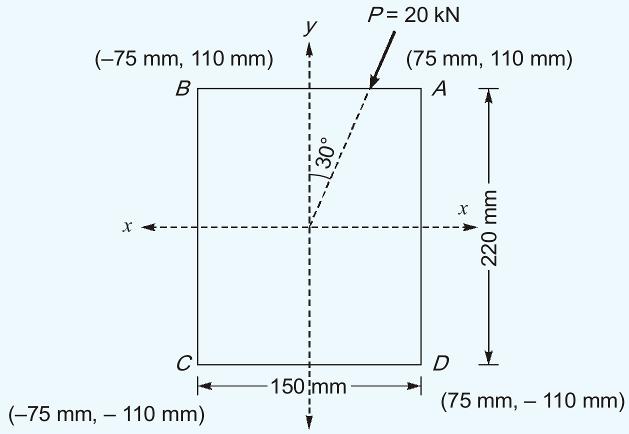
and, $\theta_{P2} = 96.44°$ (From vertical in clockwise direction)

(ii)

Given: $b = 150$ mm, $d = 220$ mm, $P = 20$ kN

$l = 4$ m





Cross-section of beam at mid span

$$M = \frac{Pl}{4} = \frac{20 \times 4}{4} = 20 \text{ kNm} = 20 \times 10^6 \text{ Nmm}$$

Bending stress at any point whose co-ordinate is (x, y) is given by

$$\sigma_{(x,\,y)} = \frac{(M\cos\theta)}{I_{xx}}y + \frac{(M\sin\theta)}{I_{yy}}x$$

$$\Rightarrow \quad \sigma_{(x,\,y)} = \frac{(20 \times 10^6 \times \cos 30^\circ)}{\left(\dfrac{150 \times 220^3}{12}\right)}y + \frac{(20 \times 10^6 \times \sin 30^\circ)}{\left(\dfrac{220 \times 150^3}{12}\right)}x$$

$$\Rightarrow \quad \sigma_{(x,\,y)} = 0.13013154y + 0.1616162x$$

Bending stress at A (75 mm, 110 mm)

$$\sigma_{A(75,\,110)} = 0.13013154 \times 110 + 0.1616162 \times 75$$
$$= 26.44 \text{ N/mm}^2 \text{ (Compressive)}$$

Bending stress at B (– 75 mm, 110 mm)

$$\sigma_{B(-75,\,110)} = 0.13013154 \times 110 + 0.1616162 \times (-75)$$
$$= 2.19 \text{ N/mm}^2$$

Bending stress at C (–75 mm, – 110 mm)

$$\sigma_{C(-75,\,-110)} = 0.13013154 \times (-110) + 0.1616162 \times (-75)$$
$$= -26.44 \text{ N/mm}^2 \text{ (Tensile)}$$

Bending stress at D (75 mm, – 110 mm)

$$\sigma_{D(75,\,-110)} = 0.13013154 \times (-110) + 0.1616162 \times (75)$$
$$= -2.19 \text{ N/mm}^2 \text{ (Tensile)}$$

Now,

Maximum compressive bending stress occurs at A = 26.44 N/mm^2

Maximum tensile bending stress occurs at C = 26.44 N/mm^2

End of Solution

Q.7 (a) (i) What is ferrocement and fiber-reinforced concrete? Write their advantages and disadvantages.

(ii) Describe, in short, the various methods of proportioning concrete.

(iii) Write, in short, the various advantages of RC structures over other masonry structures.

[8 + 6 + 6 = 20 marks : 2025]

Solution:

(i)

Ferrocement: Ferrocement is a type of thin-wall reinforced concrete construction where small-diameter wire meshes are used as reinforcement and are impregnated with a rich cement mortar mix. It does not include coarse aggregates. The reinforcement consists of layers of mesh (such as chicken mesh or woven wire mesh), sometimes supported with small-diameter steel rods (skeletal reinforcement).

Advantages of ferrocement:

1. High tensile strength due to closely spaced wire meshes.
2. Better crack control and durability.
3. Can be moulded into thin, curved sections.
4. Economical for small-scale applications.

Disadvantages of ferrocement:

1. Labour intensive fabrication process
2. Corrosion risk for mesh
3. Uneconomical for large sections
4. Requires skilled workmanship for placing and finishing.

Fibre reinforced concrete: Fibre Reinforced Concrete (FRC) is concrete containing fibrous materials uniformly distributed and randomly oriented. These fibres act as crack arrestors and improve the tensile strength, toughness, and ductility of concrete. Common types of fibres include steel, glass, synthetic fibres (like polypropylene), and natural fibres. FRC improves the behavior of concrete under load and especially enhances post-cracking performance. It is used where control of plastic shrinkage cracks and improved impact resistance are important.

Advantages of fibre reinforced concrete:

1. Reduces microcracks and plastic shrinkage.
2. Enhances tensile and flexural strength.

3. Improves resistance to impact and fatigue.
4. Increases ductility and post-cracking toughness.
5. Enhances abrasion and freeze-thaw resistance.

Disadvantages of FRC:

1. Fibres reduce the workability of the mix and may cause entrainment of air.
2. Steel fibres tend to intermesh and form balls during mixing.

(ii)

Nominal mix:

1. A mix where the cement concrete is specified by proportions of different ingredients.
2. It is presumed that by these proportions satisfactory performance can be achieved.
3. Variations in materials is not considered.
4. Generally used for concrete grades upto M20.
5. Used in small scale, non-structural or general construction works.

Design mix

1. A mix where proportions are determined and the properties of concrete are specified is called design mix concrete.
2. Target performance is achieved by mix design process based on calculations and trials using properties of materials.
3. Mix is tailored considering actual properties of materials.
4. Used for higher grades of concrete say M20 and above.
5. Used in large-scale structural and critical infrastructure works.

(iii)

Various advantages of RC structures over other masonry structures are as below:

1. RC structures combine concrete's compressive strength with steel's tensile strength, making RC structures much more stronger and capable of supporting heavier loads as compared to other masonry structures.
2. It provides design flexibility as columns can be designed to take the load, thus freeing the walls from load bearing. This allows versatile architectural designs.
3. They are more durable with much more resistance to weathering, corrosion, fire and natural calamities like earthquake.
4. In-situ RC members are cast as a single unit, reducing the presence of weak joints that are common in brick or stone masonry.
5. RC structures are capable of resisting high bending moments due to reinforcement whereas masonries are weak in flexure. It allows RC structures to deform without sudden failure.
6. While masonry is slow and labour intensive, RC structures can be built faster and more efficiently by use of mechanical placement, formwork and precast elements.

End of Solution

Q.7 (b) A simply supported reinforced concrete (RC) slab, having a clear span of 3 m, is supported only on two opposite sides on brick walls of 230 mm thick. If the live load on the slab is 3 kN/m² and floor finish being 1 kN/m², design the RC slab as per limit state method. Use M 30 concrete and Fe 500 grade steel. Also check for deflection. Sketch the reinforcement details. Assume clear cover to the reinforcement as 20 mm. Relevant portion of the IS 456 is enclosed.

[20 marks : 2025]

Solution:

Given:

Clear span, l_{clear} = 3 m, f_{ck} = 30 N/mm², f_y = 500 N/mm²

Brickwall thickness, w = 230 mm

Live load, w_{LL} = 3 kN/m²

Floor finish, w_{FF} = 1 kN/m²

Assume: d = 120 mm

$\therefore$ $D = d$ + effective cover = 120 + 20 = 140 mm

$$L_{eff} = \min.\begin{cases} L_c + d = 3 + 0.12 = 3.12\text{m} \\ L_c + w = 3 + 0.23 = 3.23\text{m} \end{cases}$$

$\Rightarrow$ L_{eff} = 3.12 m

Dead load, DL = 0.14 × 1 × 1 × 25 = 3.5 kN/m²

Live load, LL = 3 kN/m²

Floor finish, FF = 1 kN/m²

Total load, W = 7.5 kN/m²

Factored load, W_u = 1.5 × 7.5 = 11.25 kN/m²

Slab is supported on two opposite edges and hence it will behave as one way slab.

$$\text{Factored bending moment, } M_u = \frac{W_u l_{eff}^2}{8} = \frac{11.25 \times 3.12^2}{8}$$

$$M_u = 13.689 \text{ kNm}$$

$$\text{Factored shear force, } V_u = \frac{W_u l_{eff}^2}{2} = \frac{11.25 \times 3.12}{2} = 17.55 \text{ kN}$$

$$\text{Effective depth required, } d_{req} = \sqrt{\frac{M_u}{Q.b}}$$

$$\Rightarrow \quad d_{req} = \sqrt{\frac{13.689 \times 10^6}{0.133 \times 30 \times 1000}}$$

$$\Rightarrow \quad d_{req} = 58.573 \text{ mm} < 120 \text{ mm} \quad \text{(OK)}$$

$$\text{Area of steel, } \quad A_{st\ req} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] bd$$

$$\Rightarrow \quad A_{st\ req} = \frac{0.5 \times 30}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 13.689 \times 10^6}{30 \times 1000 \times 120^2}}\right] \times 1000 \times 120$$

$$\Rightarrow \qquad A_{st\ req} = 272.7\ mm^2$$

$$A_{st\ provided} = 3\times\frac{\pi}{4}(12)^2 = 339.3\ mm^2$$

$$\text{c/c spacing} = \frac{1000}{272.7}\times\frac{\pi}{4}(12)^2 = 414.7\ mm > 300\ mm$$

Provide 12 mm bars @ 300 mm c/c

Distribution reinforcement:

$$A_{st\ min} = \frac{0.12bD}{100} = \frac{0.12\times1000\times140}{100} = 168\ mm^2$$

$$\text{Using 8 mm bars, spacing} = \frac{1000}{168}\times\frac{\pi}{4}(8)^2 = 299.2\ mm$$

Provide 8 mm @ 280 mm c/c

Check for deflection

$$A_{st\ req.} = 272.7\ mm^2$$

$$A_{st\ provided} = \frac{1000}{300}\times\frac{\pi}{4}(12)^2 = 377 mm^2$$

Percentage of tensile reinforcement provided

$$P_t\% = \frac{377}{1000\times120}\times100 = 0.314\%$$

Now,
$$f_s = 0.58f_y\times\frac{(A_{st})_{required}}{(A_{st})_{Provided}}$$

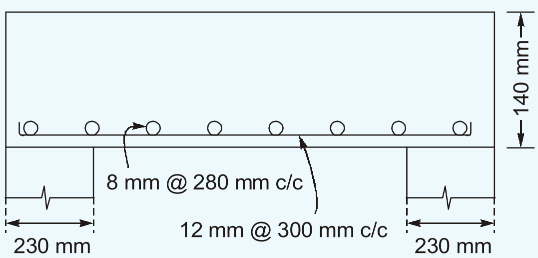
$$\Rightarrow \qquad f_s = 0.58\times500\times\frac{272.7}{377}$$

$$\Rightarrow \qquad f_s = 209.77\ N/mm^2,\ P_t = 0.314\%$$

From graph, $MF_t = 1.75$

$$\therefore \qquad d_{req} = \frac{l_{eff}}{A\times MF_t} = \frac{3.12\times10^3}{20\times1.75}$$

$$= 89.14\ mm < 120\ mm \qquad \text{OK}$$



End of Solution

Q.7 (c) In a material, the principal stresses are 60 MN/m^2, 48 MN/m^2 and –36 MN/m^2. Calculate:

(i) Total strain energy

(ii) Volumetric strain energy

(iii) Shear strain energy

(iv) Factor of safety on total strain energy criterion if the material yields at 120 MN/m^2.

Take E = 200 GN/m^2 and $\frac{1}{m}$ = 0.3.

[20 marks : 2025]

Solution:

Given: Principal stress

$$\sigma_1 = 60 \text{ MN/m}^2 = 60 \text{ N/mm}^2$$
$$\sigma_2 = 48 \text{ MN/m}^2 = 48 \text{ N/mm}^2$$
$$\sigma_3 = -36 \text{ MN/m}^2 = -36 \text{ N/mm}^2$$
$$E = 200 \text{ GN/m}^2 = 2 \times 10^5 \text{ N/mm}^2$$
$$\mu = 0.3$$

$$\text{Shear modulus, } G = \frac{E}{2(1+\mu)} = \frac{2\times10^5}{2\times1.3} = 0.769 \times 10^5 \text{ N/mm}^2$$

(i) Total strain energy per unit volume is given by

$$U_{\text{Total}} = \frac{1}{2E}\left[\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\mu\left(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1\right)\right]$$

$$\Rightarrow \quad U_{\text{Total}} = \frac{1}{2\times2\times10^5}\left[60^2 + 48^2 + (-36)^2 - 2\times0.3(60\times48 - 48\times36 - 36\times60)\right]$$

$$\Rightarrow \quad U_{\text{Total}} = 0.019512 \text{ Nmm/mm}^3$$

$$\Rightarrow \quad U_{\text{Total}} = 19.512 \text{ kNm/m}^3$$

(ii) Volumetric strain energy per unit volume.

$$U_{\text{Volume}} = U_{\text{Total}} - U_{\text{shear}}$$

$$\Rightarrow \quad U_{\text{Volume}} = 19.512 - 17.79 \qquad (U_{\text{shear}} \text{ calculated in (iii) below})$$

$$\Rightarrow \quad U_{\text{Volume}} = 1.722 \text{ kNm/m}^3$$

Alternatively,

$$U_{\text{volume}} = \frac{1}{6E}(\sigma_1 + \sigma_2 + \sigma_3)^2(1 - 2\mu)$$

$$\Rightarrow \quad U_{\text{volume}} = \frac{1}{6\times2\times10^5}(60 + 48 - 36)^2(1 - 2\times0.3)$$

$$\Rightarrow \quad U_{\text{volume}} = 1.728 \times 10^{-3} \text{ Nmm/mm}^3$$

$$\Rightarrow \quad U_{\text{volume}} = 1.728 \text{ kNm/m}^3$$

(iii) Shear strain energy per unit volume is given by

$$U_{shear} = \frac{1}{12G}\left[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right]$$

$$\Rightarrow \quad U_{shear} = \frac{1}{12\times 0.769\times 10^5}\left[(60-48)^2+(48+36)^2+(-36-60)^2\right]$$

$$\Rightarrow \quad U_{shear} = 0.01779 \text{ Nmm/mm}^3$$

$$\Rightarrow \quad U_{shear} = 17.79 \text{ kNm/m}^3$$

(iv) FOS on total strain energy criterion ($\sigma_y = 120$ N/mm^2)

$$\frac{1}{2E}\left[\sigma_1^2+\sigma_2^2+\sigma_3^2-2\mu(\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_3\sigma_1)\right] \le \frac{1}{2E}\left(\frac{\sigma_y^2}{FOS}\right)$$

$$\Rightarrow \quad 60^2+48^2+(-36)^2-2\times 0.3\,(60\times 48-48\times 36-36\times 60) \le \frac{(120)^2}{FOS}$$

$$\Rightarrow \quad \text{FOS} \le 1.845$$

End of Solution

Q.8 (a) (i) A three-hinged stiffening girder of suspension bridge of span 130 m is subjected to two point loads of 450 kN and 600 kN at distances 25 m from left support and 40 m from right support respectively. The dip of the cable is 12 m. Determine:

(1) Maximum tension in the cable

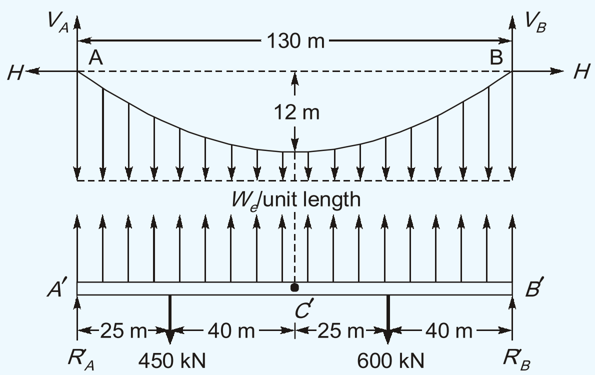
(2) Shear force and bending moment values for girder at 40 m from the left support.

(ii) A parabolic arch has a span of 20 m and is supported at different levels such that the crown C is 12 m from left support A and 8 m from right support B. The right support is higher than the left support by 2 m and the crown is higher by 1.5 m with respect to right support. The arch is hinged at the two supports and at the crown. Find the bending moment in the arch at a section Q, 4.5 m from the left support.

[10 marks : 2025]

Solution:

(i)



Consider girder:

Let the reactions at A' and B' due to given loading are R_A' and R_B' respectively.

Taking moments about A', $\Sigma M_A' = 0$

$$\Rightarrow \quad R_B' \times 130 - 450 \times 25 - 600 \times 90 = 0$$

$$R_B' = 501.923 \text{ kN}$$

Also, $$\Sigma M_C' = 0$$

$$\Rightarrow \quad R_B' \times \frac{l}{2} - \frac{w_e l^2}{8} = 0$$

$$\Rightarrow \quad w_e = \frac{4R_B'}{l} = \frac{4 \times 501.923}{130}$$

$$w_e = 15.44 \text{ kN/m}$$

$$\Sigma F_y = 0$$

$$\Rightarrow \quad R_A' + R_B' = 450 + 600$$

$$R_A' = 548.077 \text{ kN}$$

Now, $$V_A = \frac{w_e l}{2} = \frac{15.44 \times 130}{2} = 1003.6 \text{ kN}$$

$$H = \frac{w_e l^2}{8h} = \frac{15.44 \times 130^2}{8 \times 12} = 2718.083 \text{ kN}$$

1. Maximum tension in cable

$$T_{max} = \sqrt{V_A^2 + H^2} = \sqrt{1003.6^2 + 2718.083^2}$$

$$T_{max} = 2897.45 \text{ kN}$$

2. Shear force and bending moment values for girder at 40 m from the left support.

$$(SF)_{40\,m} = \text{Shear force due to given loading} + \text{Shear force due to } w_e$$

$$\Rightarrow \quad (SF)_{40\,m} = R_A' - 450 - w_e\left(\frac{130}{2} - 40\right)$$

$$\Rightarrow \quad (SF)_{40\,m} = 548.077 - 450 - 15.44\left(\frac{130}{2} - 40\right)$$

$$\Rightarrow \quad (SF)_{40\,m} = -287.923 \text{ kN}$$

$$\Rightarrow \quad (BM)_{40\,m} = \text{Moment due to given loading} + \text{Moment due to } w_e$$

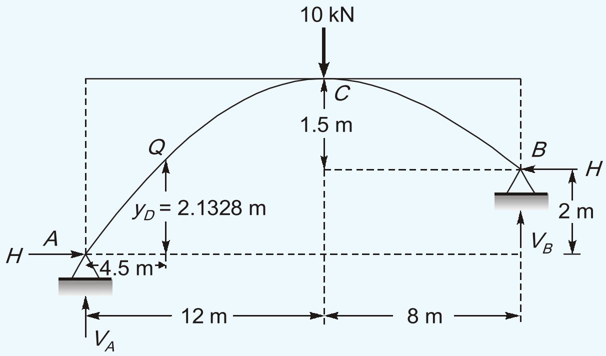
$$\Rightarrow \quad (BM)_{40\,m} = R_A'(40) - 450(40 - 25) - \frac{w_e \times 40}{2}(l - 40)$$

$$\Rightarrow \quad (BM)_{40\,m} = 548.077 \times 40 - 450 \times 15 - 15.44 \times 20\,(130 - 40)$$

$$(BM)_{40\,m} = -12618.92 \text{ kNm}$$

(ii)

Note: In this question loading is not given consider 10 kN load at the crown.



Taking moments about C (consider left side)

$$H \times 3.5 = V_A \times 12$$

$$\Rightarrow \quad V_A = \frac{7H}{24} \quad \text{...(i)}$$

Taking moments about C (consider right side)

$$H \times 1.5 = V_B \times 8$$

$$\Rightarrow \quad V_B = \frac{3H}{16} \quad \text{...(ii)}$$

$$\Sigma F_y = 0$$

$$\Rightarrow \quad V_A + V_B = 10$$

$$\Rightarrow \quad \left(\frac{7}{24} + \frac{3}{16}\right)H = 10$$

$$H = 20.87 \text{ kN}$$

From equation (i) and (ii), V_A = 6.087 kN and V_B = 3.913 kN

Equation of parabola (considering origin at A) is given by

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

Ordinate at point Q,

$$y_{D(x\,=\,4.5)} = \frac{4 \times 3.5 \times 4.5(24-4.5)}{24^2}$$

$$y_D = 2.1328 \text{ m}$$

Bending moment at Q,

$$M_Q = V_A \times 4.5 - H \times y_Q$$

$$= 6.087 \times 4.5 - 20.87 \times 2.1328 = -17.12 \text{ kNm}$$

End of Solution

Q.8 (b) A rectangular cantilever reinforced concrete (RC) beam of 300 mm x 600 mm cross-section has an effective span of 3 m. It is subjected to a dead load (self-weight plus floor finishes) of 35 kN/m and a live load of 1.5 kN/m at service state. The beam is reinforced with four rebars (reinforcing bars) of 25 mm diameter in tension zone and two rebars of 20 mm diameter in compression zone. Assume effective cover to both tension and compression reinforcement as 50 mm. Use M25 concrete and Fe 500 grade steel. Estimate only the 'initial plus creep' deflection due to permanent loads. Creep coefficient = 1.6, $E_C = 5000\sqrt{f_{ck}}$.

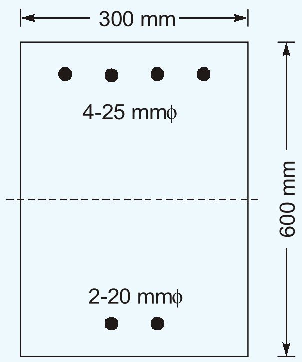
Flexural strength of concrete $f_{cr} = 0.7\sqrt{f_{ck}}$.

Relevant portion of the IS 456 is enclosed.

[20 marks : 2025]

Solution:

Given: Cantilever beam of size 300 mm × 600 mm, i.e. b = 300 mm, D = 600 mm, l_{eff} = 3 m.



$$A_{st} = 4 \times \frac{\pi}{4}(25)^2 = 1963.5 \text{ mm}^2$$

$$A_{sc} = 2 \times \frac{\pi}{4}(20)^2 = 628.32 \text{ mm}^2$$

$$f_{ck} = 25 \text{ N/mm}^2,\ f_y = 500 \text{ mm}^2$$

Effective cover = 50 mm

Creep coefficient, θ = 1.6

Modulus of elasticity, $E_c = 5000\sqrt{25} = 25000 \text{ N/mm}^2$

Flexural strength of concrete, $f_{cr} = 0.7\sqrt{25} = 3.5 \text{ N/mm}^2$

Load calculation:

Self weight + Floor finishes = 35 kN/m

Live load = 1.5 kN/m

Total service load, w_T = 36.5 kN/m = 36.5 N/mm

$$\text{Maximum bending moment, M} = \frac{w_T l_{eff}^2}{2} = \frac{36.5 \times 3^2}{2} = 164.25 \text{ kN.m}$$

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

Short term deflection:

Let actual depth of NA from bottom fibre of beam = x_a

Moment of compressive area about NA = Moment of tensile area about NA

$$\Rightarrow \quad \frac{bx_a^2}{2} + (m-1)A_{sc}(x_a - d') = mA_{st}(d - x_a)$$

$$\Rightarrow \quad \frac{300x_a^2}{2} + (8-1)\times 628.32(x_a - 50) = 8 \times 1963.5\,(550 - x_a)$$

$$\Rightarrow 150x_a^2 + 20106.24x_a - 8859312 = 0$$

$$x_a = 185.078 \text{ mm (From bottom fibre)}$$

$$\therefore \quad x = 600 - 185.078 = 414.922 \text{ mm (From top fibre)}$$

$$z = \text{Lever arm}$$

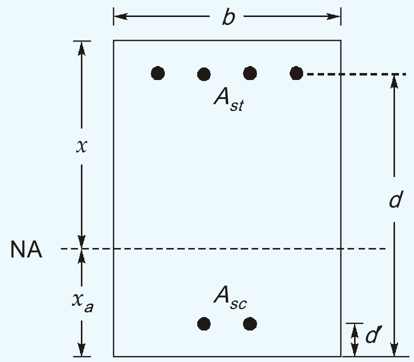
$$= d - \frac{x_a}{3} = 550 - \frac{185.078}{3} = 488.307 \text{ mm}$$

Now, moment of inertia of the gross section of beam is,

$$I_{gr} = \frac{bD^3}{12} = \frac{300\times 600^3}{12} = 5400\times 10^6 \text{ mm}^4$$

$$\text{Cracking moment, } M_{cr} = \frac{f_{cr}}{y_t}\times I_{gr} = \frac{3.5}{300}\times 5400\times 10^6 \text{ Nmm}$$

$$\Rightarrow \quad M_{cr} = 63 \text{ kN.m}$$



Moment of inertia of the cracked section,

$$I_{cr} = \frac{bx_a^3}{3} + mA_{st}\left(d - x_a^2\right) + (m-1)A_{sc}(x_a - d')^2$$

$$\Rightarrow \quad I_{cr} = \frac{300\times 185.078^3}{3} + 8\times 1963.5(550 - 185.078)^2 + (8-1)\times 628.32(185.078 - 50)^2$$

$$I_{cr} = 2806.018 \times 10^6 \text{ mm}^4$$

$$\text{Effective moment of inertia, } \quad I_{\text{eff}} = \frac{I_{cr}}{1.2 - \frac{M_{cr}}{M}\times\frac{z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$

$$\Rightarrow \quad I_{\text{eff}} = \frac{2806.018\times 10^6}{1.2 - \frac{63}{164.25}\times\frac{488.307}{550}\left(1 - \frac{414.922}{550}\right)\frac{300}{300}}$$

$$\Rightarrow \qquad I_{eff} = 2513.53 \times 10^6 \text{ mm}^4$$

$$I_{cr} > I_{eff} < I_{gr} \qquad \text{Ok}$$

Short term deflection, $\delta_1 = \dfrac{w_T l_{eff}^4}{8E_c I_{eff}} = \dfrac{36.5 \times 3000^4}{\left(8 \times 25000 \times 2513.53 \times 10^6\right)} = 5.88$ mm

Deflection due to creep (θ = 1.60)

$$\delta_2 = 1.60 \times 5.88 = 9.408 \simeq 9.41 \text{ mm}$$

Total long term deflection $= \delta_1 + \delta_2 = 5.88 + 9.41 = 15.29$ mm

Deflection due to shrinkage, $\delta_3 = k_3 \psi_{cs} l_{eff}^{\;2}$

Where, $k_3 = 0.5$ (for cantilever beam)

$$\psi_{cs} = k_4 \times \frac{\varepsilon_{cs}}{D}$$

$$\epsilon_{cs} = 3 \times 10^{-4}$$

$$k_4 = \begin{cases} 0.72\left(\dfrac{P_t - P_c}{\sqrt{P_t}}\right) \le 1 \text{ for } 0.25 \le (P_t - P_c) < 1.0 \\ 0.65\left(\dfrac{P_t - P_c}{\sqrt{P_t}}\right) \le 1 \text{ for } (P_t - P_c) \ge 1.0 \end{cases}$$

Here, $P_t = \dfrac{A_{st} \times 100}{bd} = \dfrac{1963.5}{300 \times 550} \times 100 = 1.19\%$

$$P_c = \frac{A_{sc} \times 100}{bd} = \frac{628.32}{300 \times 550} \times 100 = 0.3808\%$$

$$(P_t - P_c) = 0.8092\% < 100\%$$

$\therefore \qquad k_4 = 0.72 \times \dfrac{0.8092}{\sqrt{1.19}} = 0.5341 < 1$

Now, $\psi_{cs} = k_4 \times \dfrac{\epsilon_{cs}}{D} = 0.5341 \times \dfrac{3 \times 10^{-4}}{600} = 2.67 \times 10^{-7}$

$$\delta_3 = k_3 \psi_{cs} l_{eff}^2 = 0.5 \times 2.67 \times 10^{-7} \times 3000^2$$

$$\Rightarrow \qquad \delta_3 = 1.2 \text{ mm}$$

Total deflection, $\delta_{total} = (\delta_1 + \delta_2) + \delta_3$

$= 15.29 + 1.2 = 16.49 \text{ mm} \simeq 16.5$ mm

End of Solution

Q.8 (c) (i) Describe the different types of contract in brief. How is a tender document prepared?

(ii) Estimate the number of carriers, if the data for the project are the following:

Quantity of material to be handled = 5000000 m^3

Capacity of the loaders to be engaged = 2.3 m^3

Capacity of bottom dampers = 30 m^3

Project to be completed in two shifts in 5 years with yearly working hours = 2000 hr

Job and management factor = 0.70
Operating efficiency = 0.85
Bucket fill factor = 085
Swell factor = 0.90
Cycle time for loader = 0.50 minute
Lead distance = 6 km
Speed during empty haul @ 25 km/hr and loaded haul @ 20 km/hr

[10 + 10 = 20 marks : 2025]

Solution:

(i)

The following are the various types of contract:

1. **Item rate contract**: Contractor quotes rates for individual items, and payment is made based on actual quantities executed and measured on site. It is suitable for works which can be distinctly split into various items and quantities under each item can be estimated accurately.
2. **Percentage rate contract**: Contractor quotes a single percentage above, below or at par with departmental estimated rates, simplifying comparison among tenders.
3. **Lumpsum contract**: Contractor agrees to complete the entire work for a fixed sum, reducing the need for detailed item measurements. The contractor's profit mainly lie in the early completion time.
4. **Labour contract**: Only labour is supplied by the contractor, while materials are provided by the owner, making it suitable for private works.
5. **Cost plus percentage rate contract**: Contractor agrees to take the work of construction on the actual cost of work plus on agreed percentage in addition for his services. The contractor arranges materials and labour at his cost and keeps proper account which is paid by the department or owner with certain percentage of the cost of construction as his profit.
6. **Cost plus fixed fee contract**: Contractor receives actual cost plus an agreed fixed fee, giving certainty of contractor's profit irrespective of project cost.
7. **Cost plus sliding fee contract**: Contractor's fee varies inversely with project cost, encouraging both parties to minimise expenses.
8. **Target contract**: Contractor is paid on a cost plus percentage basis of work. Contractor receives a percentage plus or minus on savings or excess effected against a pre-agreed estimate.

Preparation of tender document involves the following items:

1. Preparatio of Notice inviting tender (NIT) document.
2. Tender form.
3. Preparation of Schedule of Quantities of work to be done, tools and plants to be supplied by the department.
4. General and special conditions of contract.
5. Complete specifications.
6. One set of drawings where necessary.

(ii)

Total working hours in 5 years

$= 2000 \times 5 \times 2 = 20000$ hrs

Quantity of material to be handled $= 5 \times 10^6$ m^3

Quantity of material to be handled per hour $= \dfrac{5 \times 10^6}{20000} = 250$ m^3/hr

Capacity of loader (adjusting with bucket fill factor),

$= 2.3 \times 0.85 = 1.955$ m^3

Actual capacity of loader (Volume of soil loaded from bank before swelling),

$= 1.955 \times 0.9 = 1.7595$ m^3

Number of cycles of loader in one hour,

$$= \frac{60}{0.5} = 120 \text{ cycles}$$

Effective number of cycles $= 120 \times 0.7 \times 0.85 = 71.4$ cycles

$\therefore$ Work done by loader in our hour,

$= 1.7595 \times 71.4 = 125.6283$ m^3/hr

Number of loaders required, $= \dfrac{250}{125.6283}$ $1.99 \simeq 2$ loaders

Capacity of dumpers $= 30$ m^3

Actual capacity of dumpers (volume of soil loaded from bank before swelling),

$= 30 \times 0.9 = 27$ m^3

Number of buckets required per dumper $= \dfrac{27}{1.7595} \approx 15.35$ buckets

Time taken to load dumper $= 15.35 \times 0.5 = 7.675$ minutes $\simeq 0.128$ hours

Travel time for dumper:

1. Empty haul $= \dfrac{6}{25} = 0.24$ hours

2. Loaded haul $= \dfrac{6}{20} = 0.3$ hours

Assuming dumping and turning time as 5 minutes i.e. 0.083 hours.

Total carrier cycle time $= 0.128 + 0.24 + 0.3 + 0.083 = 0.751$ hours

Trips per hour $= \dfrac{60}{60 \times 0.751} = 1.33156$ trips

Workdone per dumper per hour $= 1.33156 \times 27 = 35.952$ m^3/hr

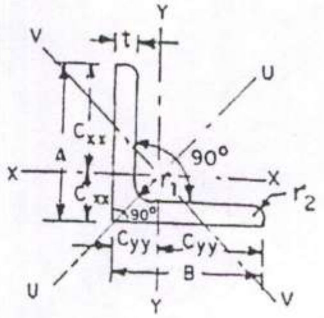
Effective workdone $= 35.952 \times 0.7 \times 0.85 = 21.39$ m^3/hr

Number of dumpers required $= \dfrac{250}{21.39} = 11.69$ dumpers $\simeq 12$ dumpers

End of Solution

■■■■

TABLE III ROLLED STEEL EQUAL ANGLES



DIMENSIONS AND PROPERTIES

(Continued)

Designation	Size $A \times B$	Thickness t	Sectional Area a	Weight per Metre w	Centre of Gravity $C_{xx} = C_{yy}$	Distance of Extreme Fibre $e_{xx} = e_{yy}$
	mm mm	mm	cm²	kg	cm	cm
ISA 7070	70 × 70	5·0	6·77	5·3	1·89	5·11
		6·0	8·06	6·3	1·94	5·06
		8·0	10·58	8·3	2·02	4·98
		10·0	13·02	10·2	2·10	4·90
ISA 7575	75 × 75	5·0	7·27	5·7	2·02	5·48
		6·0	8·66	6·8	2·06	5·44
		8·0	11·38	8·9	2·14	5·36
		10·0	14·02	11·0	2·22	5·28
ISA 8080	80 × 80	6·0	9·29	7·3	2·18	5·82
		8·0	12·21	9·6	2·27	5·73
		10·0	15·05	11·8	2·34	5·66
		12·0	17·81	14·0	2·42	5·58
ISA 9090	90 × 90	6·0	10·47	8·2	2·42	6·58
		8·0	13·79	10·8	2·51	6·49
		10·0	17·03	13·4	2·59	6·41
		12·0	20·19	15·8	2·66	6·34
ISA 100100	100 × 100	6·0	11·67	9·2	2·67	7·33
		8·0	15·39	12·1	2·76	7·24
		10·0	19·03	14·9	2·84	7·16
		12·0	22·59	17·7	2·92	7·08
ISA 110110	110 × 110	8·0	17·02	13·4	3·00	8·00
		10·0	21·06	16·5	3·08	7·92
		12·0	25·02	19·6	3·16	7·84
		15·0	30·81	24·2	3·27	7·73
ISA 130130	130 × 130	8·0	20·22	15·9	3·50	9·50
		10·0	25·06	19·7	3·58	9·42
		12·0	29·82	23·4	3·66	9·34
		15·0	36·81	28·9	3·78	9·22
ISA 150150	150 × 150	10·0	29·03	22·8	4·06	10·94
		12·0	34·59	27·2	4·14	10·86
		15·0	42·78	33·6	4·26	10·74
		18·0	50·79	39·9	4·38	10·62
ISA 200200	200 × 200	12·0	46·61	36·6	5·36	14·64
		15·0	57·80	45·4	5·49	14·51
		18·0	68·81	54·0	5·61	14·39
		25·0	93·80	73·6	5·88	14·12

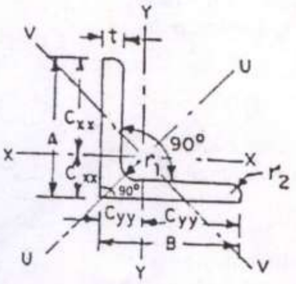


TABLE III ROLLED STEEL EQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)

Moments of Inertia			Radii of Gyration			Modulus of Section	Radius at Root	Radius at Toe	Product of Inertia	Designation
$I_{xx}=I_{yy}$	I_{uu}	I_{vv}	$r_{xx}=r_{yy}$	r_{uu}	r_{vv}	$Z_{xx}=Z_{yy}$	r_1	r_2	I_{xy}	
cm⁴	cm⁴	cm⁴	cm	cm	cm	cm³	mm	mm	cm⁴	
31·1	49·8	12·5	2·15	2·71	1·36	6·1	7·0	4·5	18·4	ISA 7070
36·8	58·8	14·8	2·14	2·70	1·36	7·3			21·7	
47·4	75·5	19·3	2·12	2·67	1·35	9·5			27·9	
57·2	90·7	23·7	2·10	2·64	1·35	11·7			33·3	
38·7	61·9	15·5	2·31	2·92	1·46	7·1	7·0	4·5	22·8	ISA 7575
45·7	73·1	18·4	2·30	2·91	1·46	8·4			27·0	
59·0	94·1	24·0	2·28	2·88	1·45	11·0			34·8	
71·4	113·3	29·4	2·26	2·84	1·45	13·5			41·7	
56·0	89·6	22·5	2·46	3·11	1·56	9·6	8·0	4·5	33·0	ISA 8080
72·5	115·6	29·4	2·44	3·08	1·55	12·6			42·7	
87·7	139·5	36·0	2·41	3·04	1·55	15·5			51·4	
101·9	161·4	42·4	2·39	3·01	1·54	18·3			59·2	
80·1	128·1	32·0	2·77	3·50	1·75	12·2	8·5	5·5	47·2	ISA 9090
104·2	166·4	42·0	2·75	3·47	1·75	16·0			61·5	
126·7	201·9	51·6	2·73	3·44	1·74	19·8			74·5	
147·9	234·9	60·9	2·71	3·41	1·74	23·3			86·5	
111·3	178·1	44·5	3·09	3·91	1·95	15·2	8·5	5·5	65·7	ISA 100100
145·1	231·8	58·4	3·07	3·88	1·95	20·0			85·8	
177·0	282·2	71·8	3·05	3·85	1·94	24·7			104·4	
207·0	329·3	84·7	3·03	3·82	1·94	29·2			121·6	
195·0	311·7	78·2	3·38	4·28	2·14	24·4	10·0	6·0	115·1	ISA 110110
238·4	380·5	96·3	3·36	4·25	2·14	30·1			140·6	
279·6	445·3	113·8	3·34	4·22	2·13	35·7			164·5	
337·4	535·4	139·3	3·31	4·17	2·13	43·7			197·0	
328·3	525·1	131·4	4·03	5·10	2·55	34·5	10·0	6·0	194·2	ISA 130130
402·7	643·4	162·1	4·01	5·07	2·54	42·7			238·3	
473·8	755·9	191·8	3·99	5·03	2·54	50·7			279·9	
574·6	914·2	235·0	3·95	4·98	2·53	62·3			337·8	
622·4	995·4	249·4	4·63	5·86	2·93	56·9	12·0	8·0	368·2	ISA 150150
735·4	1 174·8	296·0	4·61	5·83	2·93	67·7			435·0	
896·8	1 429·7	363·8	4·58	5·78	2·92	83·5			529·1	
1 048·9	1 668·2	429·5	4·54	5·73	2·91	98·7			616·0	
1 788·9	2 862·0	715·9	6·20	7·84	3·92	122·2	15·0	10·0	1 058·9	ISA 200200
2 197·7	3 511·8	883·7	6·17	7·79	3·91	151·4			1 301·2	
2 588·7	4 130·8	1 046·5	6·13	7·75	3·90	179·9			1 530·5	
3 436·3	5 460·9	1 411·6	6·05	7·63	3·88	243·3			2 015·7	

10.3.1.3 In the calculation of thread length, allowance should be made for tolerance and thread run off.

10.3.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition

$$V_{sb} = V_{db}$$

where V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (*see* **10.3.3**) and bearing, V_{dpb} (*see* **10.3.4**).

10.3.3 *Shear Capacity of Bolt*

The design strength of the bolt, V_{dsb} as governed shear strength is given by:

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows:

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

where

f_u = ultimate tensile strength of a bolt;

n_n = number of shear planes with threads intercepting the shear plane;

n_s = number of shear planes without threads intercepting the shear plane;

A_{sb} = nominal plain shank area of the bolt; and

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread.

10.3.3.1 *Long joints*

When the length of the joint, l_j of a splice or end connection in a compression or tension element containing more than two bolts (that is the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds $15d$ in the direction of load, the nominal shear capacity (*see* **10.3.2**), V_{db} shall be reduced by the factor β_{lj}, given by:

$$\beta_{lj} = 1.075 - l_j/(200\,d) \text{ but } 0.75 \le \beta_{lj} \le 1.0$$

$$= 1.075 - 0.005(l_j/d)$$

where

d = Nominal diameter of the fastener.

NOTE — This provision does not apply when the distribution of shear over the length of joint is uniform, as in the connection of web of a section to the flanges.

10.3.3.2 *Large grip lengths*

When the grip length, l_g (equal to the total thickness of the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg}, given by:

$$\beta_{lg} = 8\,d/(3\,d + l_g) = 8/(3 + l_g/d)$$

β_{lg} shall not be more than β_{lj} given in **10.3.3.1**. The grip length, l_g shall in no case be greater than $8d$.

10.3.3.3 *Packing plates*

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

$$\beta_{pk} = (1 - 0.0125\,t_{pk})$$

where

t_{pk} = thickness of the thicker packing, in mm.

10.3.4 *Bearing Capacity of the Bolt*

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

V_{npb} = nominal bearing strength of a bolt $= 2.5\,k_b\,d\,t\,f_u$

where

k_b is smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0;

e, p = end and pitch distances of the fastener along bearing direction;

d_0 = diameter of the hole;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively;

d = nominal diameter of the bolt; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb}, by the factors given below:

a) Over size and short slotted holes – 0.7, and
b) Long slotted holes – 0.5.

NOTE — The block shear of the edge distance due to bearing force may be checked as given in **6.4**.

Table 9(c) Design Compressive Stress, f_{cd} (MPa) for Column Buckling Class c
(*Clause* 7.1.2.1)

KL/r ↓	Yield Stress, f_y (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

39.3 Short Axially Loaded Members in Compression

The member shall be designed by considering the assumptions given in **39.1** and the minimum eccentricity. When the minimum eccentricity as per **25.4** does not exceed 0.05 times the lateral dimension, the members may be designed by the following equation:

$$P_u = 0.4 f_{ck} . A_c + 0.67 f_y . A_{sc}$$

where

P_u = axial load on the member,

f_{ck} = characteristic compressive strength of the concrete,

A_c = Area of concrete,

f_y = characteristic strength of the compression reinforcement, and

A_{sc} = area of longitudinal reinforcement for columns.

39.4 Compression Members with Helical Reinforcement

The strength of compression members with helical reinforcement satisfying the requirement of **39.4.1** shall be taken as 1.05 times the strength of similar member with lateral ties.

39.4.1 The ratio of the volume of helical reinforcement to the volume of the core shall not be less than $0.36\,(A_g/A_c - 1)\, f_{ck}/f_y$

where

A_g = gross area of the section,

A_c = area of the core of the helically reinforced column measured to the outside diameter of the helix,

f_{ck} = characteristic compressive strength of the concrete, and

f_y = characteristic strength of the helical reinforcement but not exceeding 415 N/mm².

39.5 Members Subjected to Combined Axial Load and Uniaxial Bending

A member subjected to axial force and uniaxial bending shall be designed on the basis of **39.1** and **39.2**.

NOTE — The design of member subject to combined axial load and uniaxial bending will involve lengthy calculation by trial and error. In order to overcome these difficulties interaction diagrams may be used. These have been prepared and published by BIS in 'SP : 16 Design aids for reinforced concrete to IS 456'.

39.6 Members Subjected to Combined Axial Load and Biaxial Bending

The resistance of a member subjected to axial force and biaxial bending shall be obtained on the basis of assumptions given in **39.1** and **39.2** with neutral axis so chosen as to satisfy the equilibrium of load and moments about two axes. Alternatively such members may be designed by the following equation:

$$\left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_n} \le 1.0$$

where

M_{ux}, M_{uy} = moments about x and y axes due to design loads,

M_{ux1}, M_{uy1} = maximum uniaxial moment capacity for an axial load of P_u, bending about x and y axes respectively, and

α_n is related to P_u/P_{uz}

where $P_{uz} = 0.45 f_{ck} . A_c + 0.75 f_y . A_{sc}$

For values of P_u/P_{uz} = 0.2 to 0.8, the values of α_n vary linearly from 1.0 to 2.0. For values less than 0.2, α_n is 1.0; for values greater than 0.8, α_n is 2.0.

39.7 Slender Compression Members

The design of slender compression members (*see* **25.1.1**) shall be based on the forces and the moments determined from an analysis of the structure, including the effect of deflections on moments and forces. When the effect of deflections are not taken into account in the analysis, additional moment given in **39.7.1** shall be taken into account in the appropriate direction.

39.7.1 The additional moments M_{ax} and M_{ay} shall be calculated by the following formulae:

$$M_{ax} = \frac{P_u D}{2\,000}\left\{\frac{l_{ex}}{D}\right\}^2$$

$$M_{ay} = \frac{P_u b}{2\,000}\left\{\frac{l_{ey}}{b}\right\}^2$$

where

P_u = axial load on the member,

l_{ex} = effective length in respect of the major axis,

l_{ey} = effective length in respect of the minor axis,

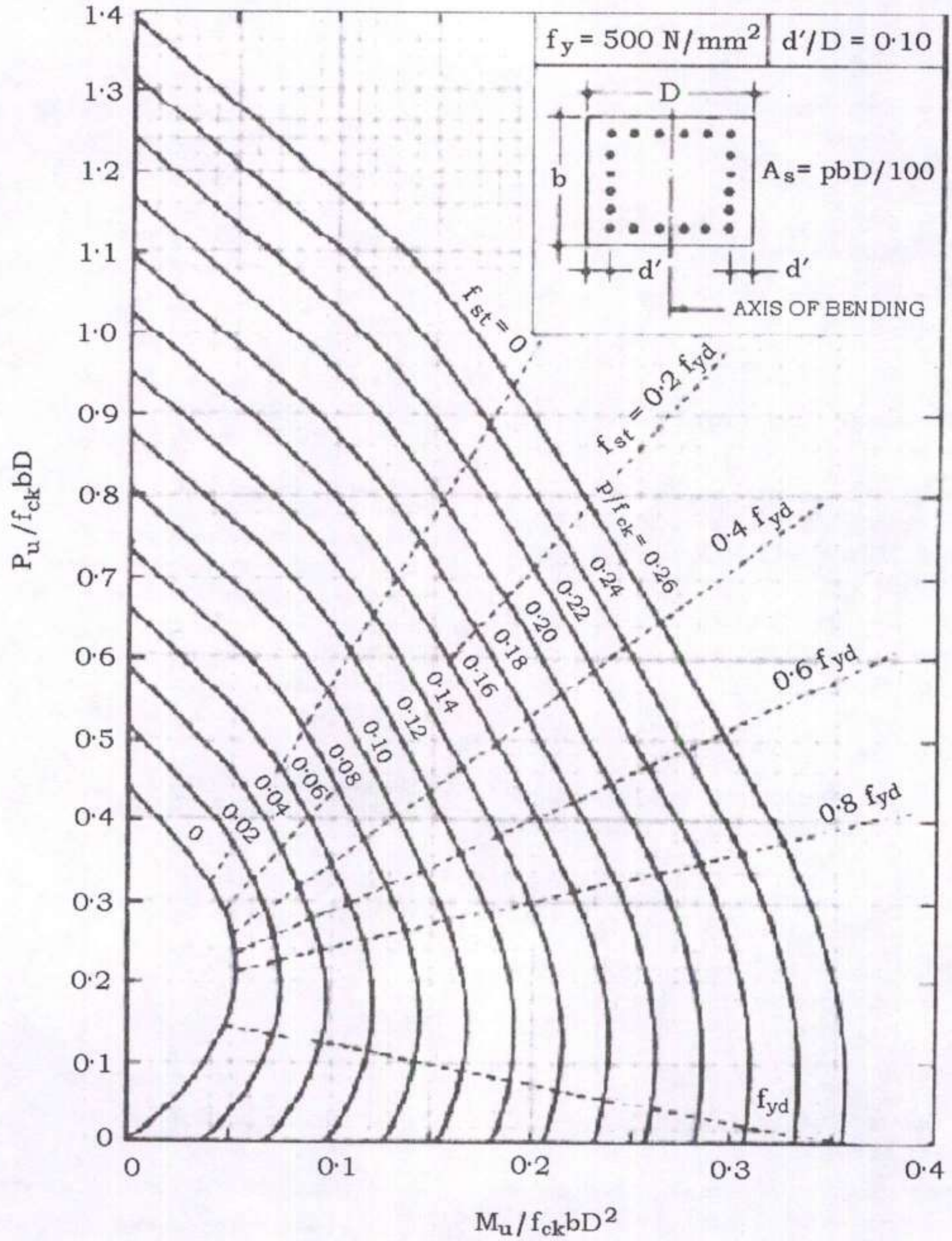
D = depth of the cross-section at right angles to the major axis, and

b = width of the member.

For design of section, **39.5** or **39.6** as appropriate shall apply.

f_y
500

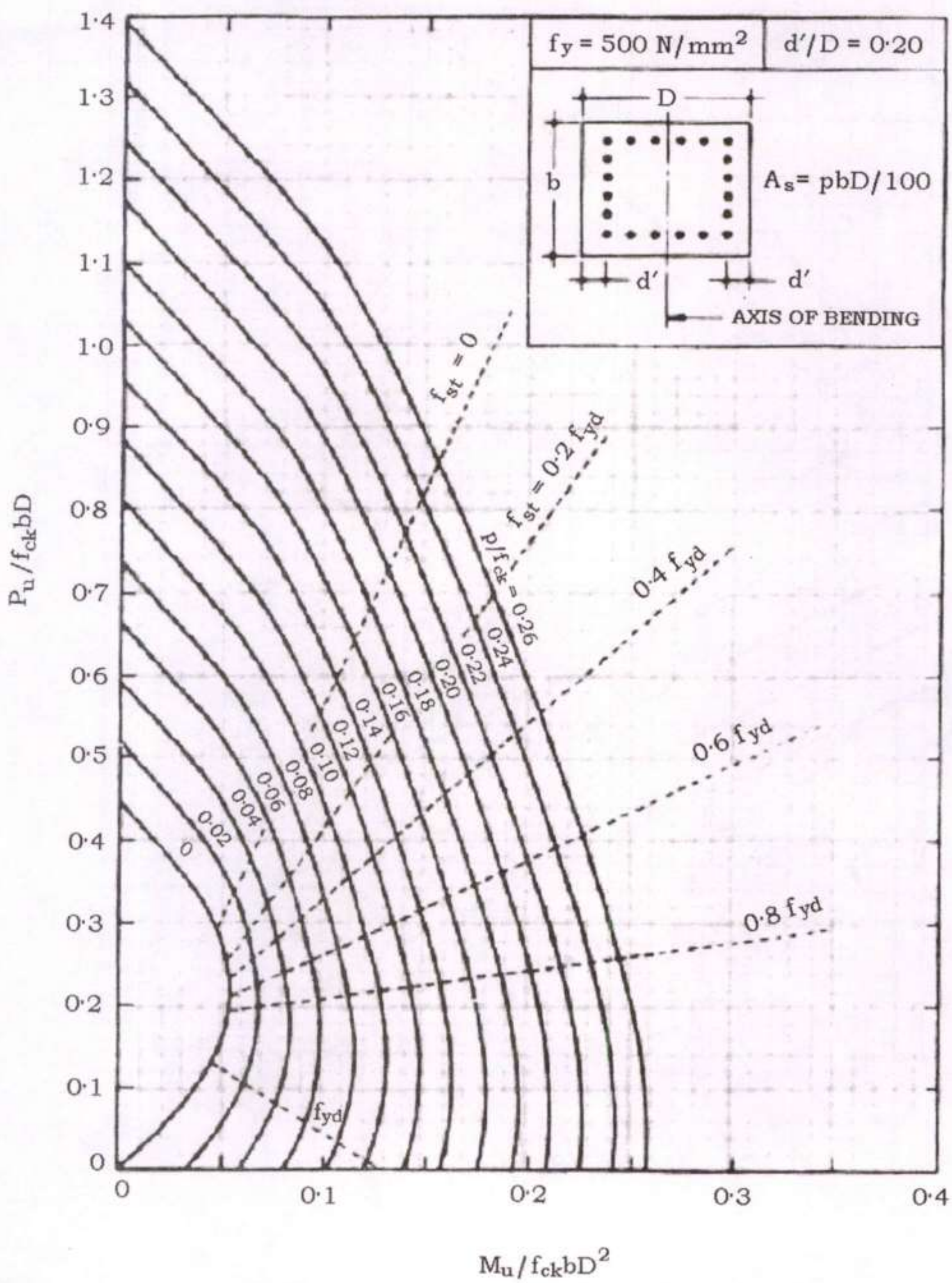
Chart 48 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides



Compression Members

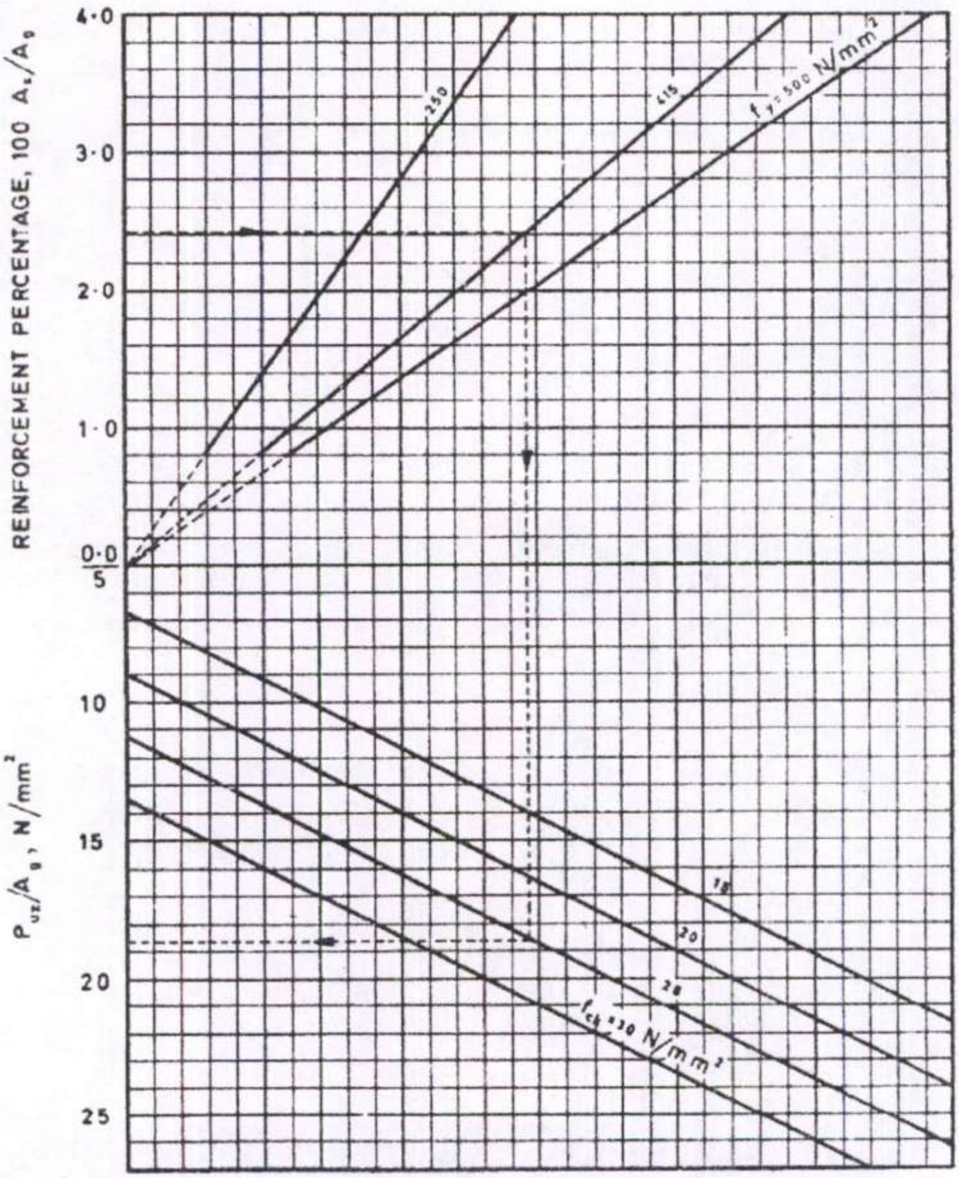
f_y
500

Chart 50 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides



Compression Members

Chart 63 VALUES OF P_{uz} for COMPRESSION MEMBERS



Design Aids for Reinforced Concrete

5.6.1.1 Where the deflection due to the combination of dead load and live load is likely to be excessive, consideration should be given to pre-camber the beams, trusses and girders. The value of desired camber shall be specified in design drawing. Generally, for spans greater than 25 m, a camber approximately equal to the deflection due to dead loads plus half the live load may be used. The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection. Roofs, which are very flexible, shall be designed to withstand any additional load that is likely to occur as a result of ponding of water or accumulation of snow or ice.

5.6.2 *Vibration*

Suitable provisions in the design shall be made for the dynamic effects of live loads, impact loads and vibration due to machinery operating loads. In severe cases possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to effective width of lateral load resistance system exceeding 5:1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to large number of cycles of loading shall be designed against fatigue failure, as specified in Section 13. Floor vibration effect shall be considered using specialist literature (*see* Annex C).

5.6.3 *Durability*

Factors that affect the durability of the buildings, under conditions relevant to their intended life, are listed below:

a) Environment,
b) Degree of exposure,
c) Shape of the member and the structural detail,
d) Protective measure, and
e) Ease of maintenance.

5.6.3.1 The durability of steel structures shall be ensured by following recommendations in Section 15. Specialist literature may be referred to for more detailed and additional information in design for durability.

5.6.4 *Fire Resistance*

Fire resistance of a steel member is a function of its mass, its geometry, the actions to which it is subjected, its structural support condition, fire protection measures adopted and the fire to which it is exposed. Design provisions to resist fire are briefly discussed in Section 16. Specialist literature may be referred to for more detailed information in design of fire resistance of steel structures.

SECTION 6
DESIGN OF TENSION MEMBERS

6.1 Tension Members

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads upto the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions (*see* **6.4.1**).

The factored design tension T, in the members shall satisfy the following requirement:

$$T < T_d$$

where

T_d = design strength of the member.

The design strength of a member under axial tension, T_d, is the lowest of the design strength due to yielding of gross section, T_{dg}; rupture strength of critical section, T_{dn}; and block shear T_{db}, given in **6.2**, **6.3** and **6.4**, respectively.

6.2 Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, is given by

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

f_y = yield stress of the material,
A_g = gross area of cross-section, and
γ_{m0} = partial safety factor for failure in tension by yielding (*see* Table 5).

6.3 Design Strength Due to Rupture of Critical Section

6.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, A_n, at the holes is given by

$$T_{dn} = 0.9\, A_n f_u / \gamma_{m1}$$

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5),
f_u = ultimate stress of the material, and
A_n = net effective area of the member given by,

$$A_n = \left[b - nd_h + \sum_i \frac{p_{si}^2}{4g_i}\right] t$$

where

b, t = width and thickness of the plate, respectively,

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),

g = gauge length between the bolt holes, as shown in Fig. 5,

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

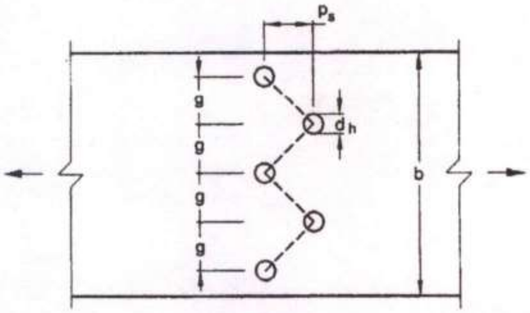


FIG. 5 PLATES WITH BOLTS HOLES IN TENSION

6.3.2 *Threaded Rods*

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

$$T_{dn} = 0.9\, A_n f_u / \gamma_{m1}$$

where

A_n = net root area at the threaded section.

6.3.3 *Single Angles*

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn} as governed by rupture at net section is given by:

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

where

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \leq (f_u\gamma_{m0}/f_y\gamma_{m1}) \geq 0.7$$

where

w = outstand leg width,

b_s = shear lag width, as shown in Fig. 6, and

L_c = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

For preliminary sizing, the rupture strength of net section may be approximately taken as:

$$T_{dn} = \alpha A_n f_u / \gamma_{m1}$$

where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length;

A_n = net area of the total cross-section;

A_{nc} = net area of the connected leg;

A_{go} = gross area of the outstanding leg; and

t = thickness of the leg.

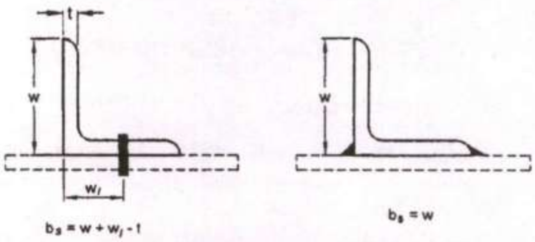


FIG. 6 ANGLES WITH SINGLE LEG CONNECTIONS

6.3.4 *Other Section*

The rupture strength, T_{dn}, of the double angles, channels, I-sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in **6.3.3**, where β is calculated based on the shear lag distance, b_s, taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross-section.

6.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given in **6.4.1**.

6.4.1 *Bolted Connections*

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y / (\sqrt{3}\,\gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

or

$$T_{db} = (0.9 A_{vn} f_u / (\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$

where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. 7A and 1-2 as shown in Fig. 7B),

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. 7B), and

f_u, f_y = ultimate and yield stress of the material, respectively.

6.4.2 *Welded Connection*

The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

SECTION 7
DESIGN OF COMPRESSION MEMBERS

7.1 Design Strength

7.1.1 Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c, or d as given Table 7.

7.1.2 The design compressive strength P_d, of a member is given by:

$$P < P_d$$

where

$$P_d = A_e f_{cd}$$

where

A_e = effective sectional area as defined in **7.3.2**, and

f_{cd} = design compressive stress, obtained as per **7.1.2.1**.

7.1.2.1 The design compressive stress, $f_{cd,}$ of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \le f_y / \gamma_{m0}$$

where

ϕ = 0.5 [1 + α (λ – 0.2)+ λ^2]

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r}\right)^2 / \pi^2 E}$$

f_{cc} = Euler buckling stress = $\dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

where

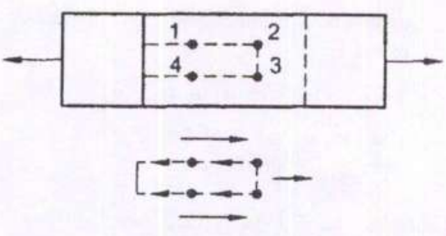
KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r;

α = imperfection factor given in Table 7;

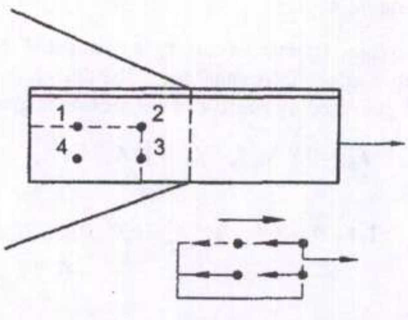
χ = stress reduction factor (*see* Table 8) for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{\left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right]}$$

λ_{m0} = partial safety factor for material strength.



7A Plate



7B Angle

FIG. 7 BLOCK SHEAR FAILURE

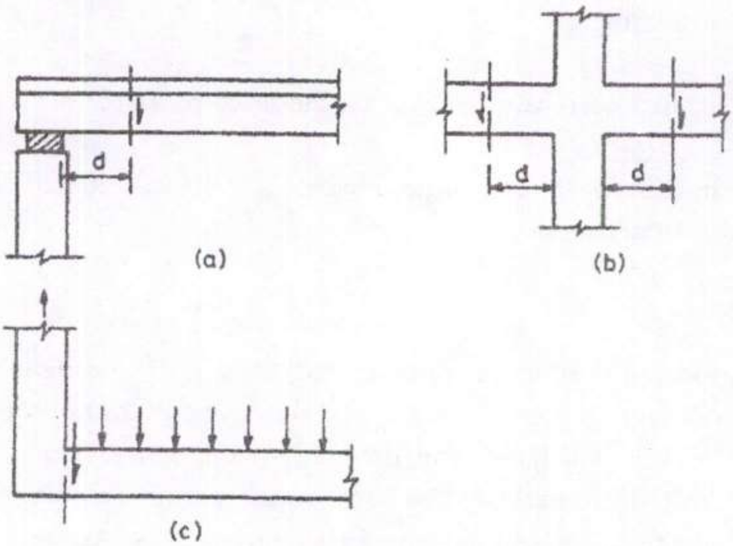


FIG. 2 TYPICAL SUPPORT CONDITIONS FOR LOCATING FACTORED SHEAR FORCE

but in no case greater than the breadth of the web plus half the sum of the clear distances to the adjacent beams on either side.

a) For T-beams, $b_f = \frac{l_0}{6} + b_w + 6\,D_f$

b) For L-beams, $b_f = \frac{l_0}{12} + b_w + 3\,D_f$

c) For isolated beams, the effective flange width shall be obtained as below but in no case greater than the actual width:

$$T-\text{beam},\ b_f = \frac{l_0}{\left(\frac{l_0}{b}\right)+4} + b_w$$

$$L-\text{beam},\ b_f = \frac{0.5\ l_0}{\left(\frac{l_0}{b}\right)+4} + b_w$$

where

b_f = effective width of flange,

l_0 = distance between points of zero moments in the beam,

b_w = breadth of the web,

D_f = thickness of flange, and

b = actual width of the flange.

NOTE — For continuous beams and frames, 'l_0' may be assumed as 0.7 times the effective span.

23.2 Control of Deflection

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed span/250.

b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

23.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.

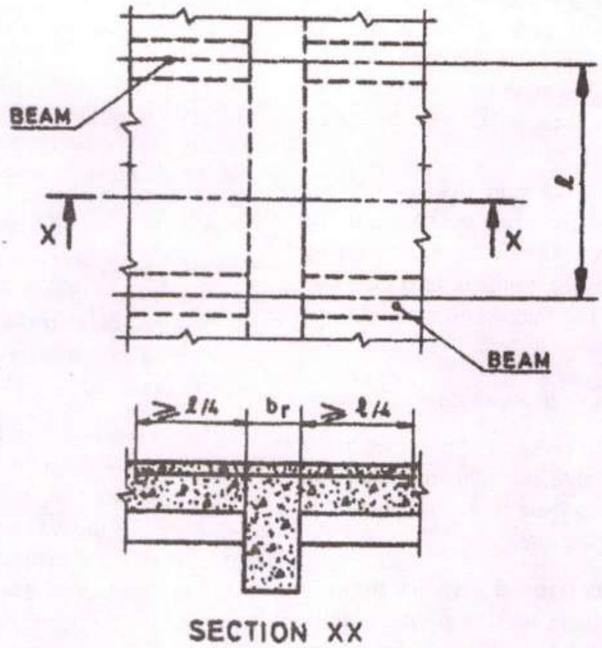
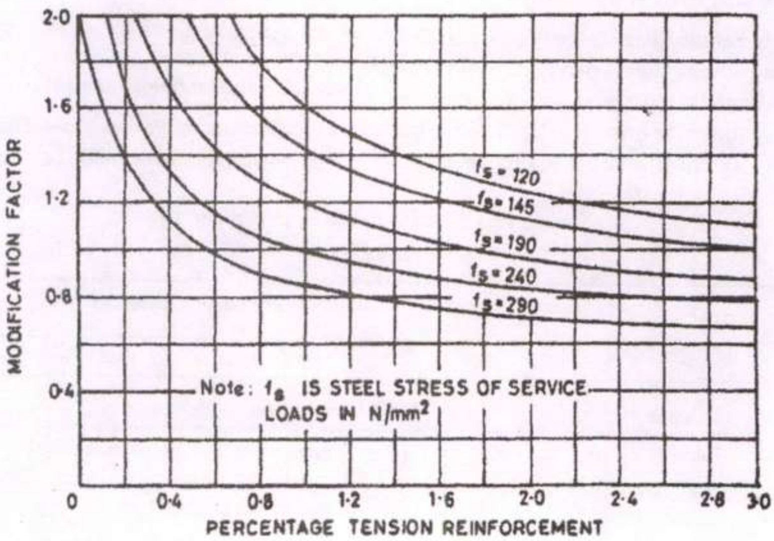


FIG. 3 TRANSVERSE REINFORCEMENT IN FLANGE OF T-BEAM WHEN MAIN REINFORCEMENT OF SLAB IS PARALLEL TO THE BEAM

e) For flanged beams, the values of (a) or (b) be modified as per Fig. 6 and the reinforcement percentage for use in Fig. 4 and 5 should be based on area of section equal to $b_f\, d$.

NOTE—When deflections are required to be calculated, the method given in Annex C may be used.



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT used the horizontal distance between bars of a group may be reduced to two-thirds the nominal maximum size of the coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

c) Where there are two or more rows of bars, the bars shall be vertically in line and the minimum vertical distance between the bars shall be 15 mm, two-thirds the nominal maximum size of aggregate or the maximum size of bars, whichever is greater.

26.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.

b) *Slabs*

1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

26.4 Nominal Cover to Reinforcement

26.4.1 *Nominal Cover*

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

26.4.2 *Nominal Cover to Meet Durability Requirement*

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in **8.2.3** shall be as given in Table 16.

26.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.

26.4.2.2 For footings minimum cover shall be 50 mm.

26.4.3 *Nominal Cover to Meet Specified Period of Fire Resistance*

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16A.

26.5 Requirements of Reinforcement for Structural Members

26.5.1 *Beams*

26.5.1.1 *Tension reinforcement*

a) *Minimum reinforcement*—The minimum area of tension reinforcement shall be not less than that

Table 15 Clear Distance Between Bars

(*Clause* 26.3.3)

f_y	Percentage Redistribution to or from Section Considered				
	-30	−15	0	+15	+30
	Clear Distance Between Bars				
N/mm²	mm	mm	mm	mm	mm
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195

NOTE — The spacings given in the table are not applicable to members subjected to particularly aggressive environments unless in the calculation of the moment of resistance, f_y has been limited to 300 N/mm² in limit state design and σ_{st} limited to 165 N/mm² in working stress design.

ANNEX G

(*Clause* 38.1)

MOMENTS OF RESISTANCE FOR RECTANGULAR AND T-SECTIONS

G-0 The moments of resistance of rectangular and T-sections based on the assumptions of **38.1** are given in this annex.

G-1 RECTANGULAR SECTIONS

G-1.1 Sections Without Compression Reinforcement

The moment of resistance of rectangular sections without compression reinforcement should be obtained as follows :

a) Determine the depth of netutral axis from the following equation :

$$\frac{x_u}{d} = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b.d}$$

b) If the value of x_u/d is less than the limiting value (*see* Note below **38.1**), calculate the moment of resistance by the following expression :

$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$

c) If the value of x_u/d is equal to the limiting value, the moment of resistance of the section is given by the following expression :

$$M_{u\cdot lim} = 0.36\, \frac{x_{u,\,max}}{d}\left(1 - 0.42\, \frac{x_{u,\,max}}{d}\right) bd^2\, f_{ck}$$

d) If x_u / d is greater than the limiting value, the section should be redesigned.

In the above equations,

x_u = depth of neutral axis,

d = effective depth,

f_y = characteristic strength of reinforcement,

A_{st} = area of tension reinforcement,

f_{ck} = characteristic compressive strength of concrete,

b = width of the compression face,

$M_{u,\,lim}$ = limiting moment of resistance of a section without compression reinforcement, and

$x_{u,\,max}$ = limiting value of x_u from **39.1**.

G-1.2 Section with Compression Reinforcement

Where the ultimate moment of resistance of section exceeds the limiting value, $M_{u,\,lim}$ compression reinforcement may be obtained from the following equation :

$$M_u - M_{u,\,lim} = f_{sc}\, A_{sc}\, (d - d')$$

where

M_u, $M_{u,\,lim}$, d are same as in **G-1.1**,

f_{sc} = design stress in compression reinforcement corresponding to a strain of

$$0.003\,5\, \frac{(x_{u,\,max} - d')}{x_{u,\,max}}$$

where

$x_{u,\,max}$ = the limiting value of x_u from **38.1**,

A_{sc} = area of compression reinforcement, and

d' = depth of compression reinforcement from compression face.

The total area of tension reinforcement shall be obtained from the following equation :

$$A_{st} = A_{st1} + A_{st2}$$

where

A_{st} = area of the total tensile reinforcement,

A_{st1} = area of the tensile reinforcement for a singly reinforced section for $M_{u,\,lim}$, and

$A_{st2} = A_{sc} f_{sc} / 0.87 f_y$.

G-2 FLANGED SECTION

G-2.1 For $x_u < D_f$, the moment of resistance may be calculated from the equation given in **G-1.1**.

G-2.2 The limiting value of the moment of resistance of the section may be obtained by the following equation when the ratio D_f / d does not exceed 0.2 :

$$M_u = 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) f_{ck}\, b_w d^2$$

$$+ 0.45\, f_{ck}\, (b_f - b_w)\, D_f \left(d - \frac{D_f}{2}\right)$$

where

M_u, $x_{u,max}$, d and f_{ck} are same as in **G-1.1**,

b_f = breadth of the compression face/flange,

b_w = breadth of the web, and

D_f = thickness of the flange.

ANNEX C

(*Clauses* 22.3.2, 23.2.1 *and* 42.1)

CALCULATION OF DEFLECTION

C-1 TOTAL DEFLECTION

C-1.1 The total deflection shall be taken as the sum of the short-term deflection determined in accordance with C-2 and the long-term deflection, in accordance with C-3 and C-4.

C-2 SHORT-TERM DEFLECTION

C-2.1 The short-term deflection may be calculated by the usual methods for elastic deflections using the short-term modulus of elasticity of concrete, E_c and an effective moment of inertia I_{eff} given by the following equation:

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

$$I_r \le I_{eff} \le I_{gr}$$

where

I_r = moment of inertia of the cracked section,

M_r = cracking moment, equal to $\frac{f_{cr} I_{gr}}{y_t}$ where

f_{cr} is the modulus of rupture of concrete, I_{gr} is the moment of inertia of the gross section about the centroidal axis, neglecting the reinforcement, and y_t is the distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension,

M = maximum moment under service loads,

z = lever arm,

x = depth of neutral axis,

d = effective depth,

b_w = breadth of web, and

b = breadth of compression face.

For continuous beams, deflection shall be calculated using the values of I_r, I_{gr} and M_r modified by the following equation:

$$X_e = k_1\left[\frac{X_1 + X_2}{2}\right] + (1 - k_1)X_o$$

where

X_e = modified value of X,

X_1, X_2 = values of X at the supports,

X_o = value of X at mid span,

k_1 = coefficient given in Table 25, and

X = value of I_r, I_{gr} or M_r as appropriate.

C-3 DEFLECTION DUE TO SHRINKAGE

C-3.1 The deflection due to shrinkage a_{cs} may be computed from the following equation:

$$a_{cs} = k_3 \psi_{cs} l^2$$

where

k_3 is a constant depending upon the support conditions,

0.5 for cantilevers,

0.125 for simply supported members,

0.086 for members continuous at one end, and

0.063 for fully continuous members.

ψ_{cs} is shrinkage curvature equal to $k_4 \frac{\varepsilon_{cs}}{D}$

where ε_{cs} is the ultimate shrinkage strain of concrete (*see* 6.2.4).

$$k_4 = 0.72 \times \frac{P_t - P_c}{\sqrt{P_t}} \le 1.0 \text{ for } 0.25 \le P_t - P_c < 1.0$$

$$= 0.65 \times \frac{P_t - P_c}{\sqrt{P_t}} \le 1.0 \text{ for } P_t - P_c \ge 1.0$$

Table 25 Values of Coefficient, k_1

(*Clause* C-2.1)

k_2	0.5 or less	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
k_1	0	0.03	0.08	0.16	0.30	0.50	0.73	0.91	0.97	1.0

NOTE — k_2 is given by

$$k_2 = \frac{M_1 + M_2}{M_{F1} + M_{F2}}$$

where

M_1, M_2 = support moments, and

M_{F1}, M_{F2} = fixed end moments.

where $P_t = \frac{100\,A_{st}}{bd}$ and $P_c = \frac{100\,A_{sc}}{bd}$

and D is the total depth of the section, and l is the length of span.

C-4 DEFLECTION DUE TO CREEP

C-4.1 The creep deflection due to permanent loads $a_{cc\,(perm)}$ may be obtained from the following equation:

$$a_{cc\,(perm)} = a_{i,cc\,(perm)} - a_{i\,(perm)}$$

where

$a_{i,cc\,(perm)}$ = initial plus creep deflection due to permanent loads obtained using an elastic analysis with an effective modulus of elasticity,

$E_{ce} = \frac{E_c}{1+\theta}$; θ being the creep coefficient,

and

$a_{i\,(perm)}$ = short-term deflection due to permanent load using E_c.