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Detailed Solutions

ESE-2021 Mains Test Series

Civil Engineering Test No : 12

Q.1 (a) Solution:

(i) Binary cements are two cementing constituent systems, in which one constituent is OPC and the other is one of the cementing pozzolanas like fly-ash (FA), ground granulated blast furnace slag (GBFS), silica fume (SF), and rice husk as (RHA).

(ii) The major advantages currently recognized are: improved and dense pore structure which reduces the micro cracks in the transition zone in concrete; reduced permeability enhances resistance to chemical attack, low diffusively to chloride ions and hence better resistance to corrosion of steel reinforcement and low heat of hydration.

(iii) However, these cements are often associated with shortcomings, such as the need for extended moist-curing, low-early age strengths, increased use of admixtures, increased cracking tendency due to plastic shrinkage and as such, these cements remain largely underutilized.

(iv) The general-purpose cements of this category are portland-pozzolana cement (OPC-FA), portland slag cement (OPC-GBFS) and super-sulphated cement.

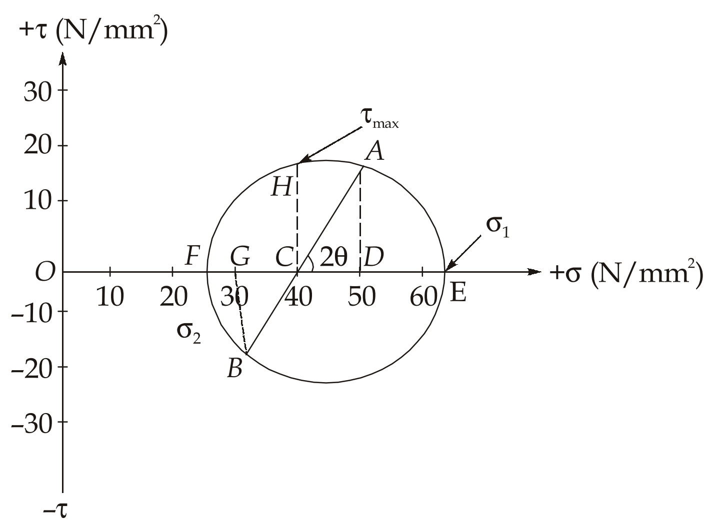
Portland-pozzolana cement can be produced either by intergrinding the predetermined quantities of portland cement clinker and pozzolana (15 to 35 percent by mass of portland-pozzolana cement) together with small amounts of gypsum, or by intimately and uniformly blending portland cement having predetermined fineness and fine pozzolana. Protland-pozzolana cement produces less heat of hydration and offers greater resistance to the sulphated attack and chloride-ion penetration due to impurities in water than normal portland cement. It is particularly useful in marine and hydraulic constructions, and other mass concrete structures like dam, bridge piers and thick foundations.

Super-sulphated cement:

This cement is manufactured by grinding together a mixture of (80 to 85 percent) well-granulated slag and 10 to 15 percent of calcium sulfate with about five percent of portland cement clinker. The total heat of hydration is very low, about 40 to 45 cal/g after seven days and 45 to 50 cal/g at 28 days, which make it suitable for mass concreting. Due to high sulphated resistance, it is particularly useful in the foundations exposed to chemically aggressive conditions, or in the manufacture of RCC pipes to be buried in sulphated bearing soils. The super-sulphated cement concrete may expand or contract slightly on setting according to the ambient conditions and hence should be properly cured.

Q.1 (b) Solution:

The shearing stress acting in conjunction with σ_x are counter clockwise, hence τ_{xy} is said to be positive on the vertical axis. Similarly, the shearing stresses acting in conjunction with σ_y are clockwise, hence τ_{xy} is said to be negative on the horizontal planes. On σ-τ diagram, construct a circle with the line joining the points (σ_x, τ_{xy}) or (50, 20) and point $(\sigma_y, -\tau_{xy})$(30, – 20) as diameter as shown by *A* and *B* respectively.



The principal stresses and their directions can be obtained from a scaled drawing, but we shall calculate σ_1, σ_2 and others values.

$$DA = 20\text{MPa}$$

$$OD = \sigma_x = 50\text{MPa}$$

$$OG = \sigma_y = 30\text{MPa}$$

$$OC = \frac{OD + OG}{2} = \frac{50 + 30}{2} = 40\text{MPa}$$

$$CD = OD - OC = 50 - 40 = 10\text{MPa}$$

$$AC^2 = CD^2 + DA^2$$

$$\Rightarrow \quad AC^2 = 10^2 + 20^2$$

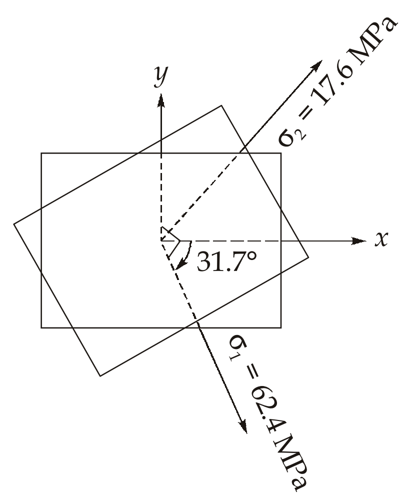
$$\Rightarrow \quad AC = 22.36 \text{ MPa}$$

Major principal stress, $\sigma_1 = OE = OC + AC = 40 + 22.36 = 62.36 \text{ MPa} \simeq 62.4 \text{ MPa}$

Minor principal stress, $\sigma_2 = OF = OC - AC = 40 - 22.36 = 17.64 \text{ MPa} \simeq 17.6 \text{ MPa}$

$$2\theta = \tan^{-1}\left(\frac{AD}{CD}\right) = \tan^{-1}\left(\frac{20}{10}\right) = 63.43°$$

$$\therefore \quad \theta = 31.7°$$

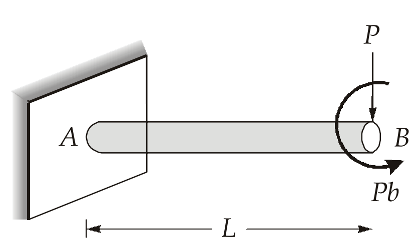


Maximum shear stress $= \tau_{max} = CH = AC = 22.36$ MPa

(which occurs on planes at 45° to those of the principal stresses i.e., 76.7°)

Q.1 (c) Solution:

Apply equal and opposite vertical load *P* at end *B* and now consider effect of *BC* on steel rod *AB*.



Torsional moment on AB, $T = P \times b$

Vertical load at $B = P$

(i) Bending of AB since BC is rigid

Hence, Deflection at C = deflection of point B due to point load P at B

$$\Rightarrow \quad \delta_c = \frac{P(L_{AB})^3}{3EI} = \frac{PL^3}{3EI}$$

$$\Rightarrow \quad \delta_c = \frac{PL^3}{3E\frac{\pi d^4}{64}} = \frac{64PL^3}{3\pi Ed^4}$$

$$\Rightarrow \quad \delta_c = \left(\frac{64}{3\pi d^4}\right)\frac{PL^3}{E}$$

(ii) Torsion of AB,

Torque, $T = Pb$

Due to torque, T, end B rotates by an angle θ_B in counter clockwise direction. Hence, rigid bar BC will deflect downward

$$\text{Rotation of rod } AB \text{ at } B,\ \theta_B = \frac{TL}{GI_P} = \frac{PbL}{G \times \frac{\pi d^4}{32}} = \left(\frac{32}{\pi d^4 G}\right)PbL$$

If rod AB rotates by an angle θ, then end C will deflect by,

$$= \theta_B \times b$$

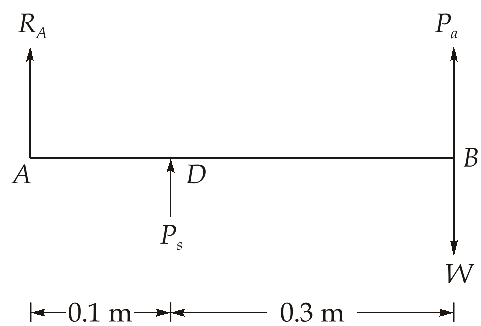
$$= \left(\frac{32}{\pi d^4}\right)\frac{PbL}{G} \times b = \left(\frac{32}{\pi d^4}\right)\frac{Pb^2L}{G} \qquad \text{(down ward)}$$

(iii) Both bending and torsion,

$$\delta_c = \left(\frac{64}{3\pi d^4}\right)\frac{PL^3}{E} + \left(\frac{32}{\pi d^4}\right)\frac{Pb^2L}{G}$$

Q.1 (d) Solution:

Let us use suffix 'a' for aluminium and suffix s for steel. There are four unknowns viz. R_A, P_s, P_a and W as shown in figure below:



Let P_a be the axial stress (tensile) in aluminium bar

P_s be the axial stress (compressive) is steel bar

$$\therefore \quad \frac{\Delta_s + \delta}{0.1} = \frac{\Delta_a}{0.4}$$

$$\Rightarrow \quad 4(\Delta_s + d) = \Delta_a$$

$$\Rightarrow \quad 4\left[\frac{p_s \times 200}{2 \times 10^5} + 0.1\right] = \frac{p_a \times 400}{0.8 \times 10^5}$$

$$\Rightarrow \quad p_a = 0.8p_s + 80 \qquad \text{...(i)}$$

If we have $p_s = 120$ N/mm^2, we get $p_a = 0.8 \times 120 + 80 = 176$ N/mm^2, which is more than the permissible value of 140 N/mm^2. Hence adopt $p_a = 140$ N/mm^2, corresponding to which we obtain $p_s = 75$ N/mm^2.

$$\therefore \quad P_a = 140 \times 800 \times 10^{-3} = 112 \text{ kN}$$

$$\text{And} \quad P_s = 75 \times 1500 \times 10^{-3} = 112.5 \text{ kN}$$

Now, taking moments about A, we get

$$112 \times 0.4 + 112.5 \times 0.1 = 0.4W$$

$$\Rightarrow \quad W = 140.125 \text{ kN}$$

Q.1 (e) Solution:

(i) Ingredients of an oil borne paint:

An oil paint essentially consists of the following ingredients:

(a) a base

(b) a vehicle or a carrier

(c) a drier
(d) a coloring pigment
(e) a solvent

Base:A base is a solid substance in a fine state of division and it forms the bulk of the paint. It determines the character of the paint and imparts durability to the surface which is painted. It reduces shrinkage cracks formed on drying and it also forms an opaque layer to obscure the surface of material to be painted.

Base for paints: White lead and red lead.

Vehicle: The vehicles are the liquid substances which hold the ingredients of a paint in liquid suspension for the following purposes:

(i) to make it possible to spread the paint evenly and uniformly on the surface in the form of a thin layer; and

(ii) to provide a binder for the ingredients of a paint so that they may stick or adhere to the surface.

Examples: Linseed oil and tung oil**.**

Driers: These substances accelerate the process of drying. A drier absorbs oxygen from the air and transfers it to the linseed oil, which in turn, gets hardened.

Example: Litharge red lead and sulphate of manganese.

Colouring pigments: When it is desired to have a different colour than the base of a paint, a colouring pigment is to be added. The pigments are available in the form of fine powders in various colours and qualities.

Examples: Black (Graphite, lamp black), Blue (Indigo, prussian blue)

Solvents: The function of a solvent is to make the paint thin so that it can be easily applied on the surface. It also helps the paint in penetrating through the porous surfaces.

Examples: Turpentine

(ii) PVCN stands for pigment volume concentration number and is given by:

$$\text{PVCN} = \frac{V_1}{V_1 + V_2}$$

V_1 = Volume of pigment in the paint

V_2 = Volume of non-volatile vehicle or carrier in the paint.

The higher the value of PVCN, the lower will be the durability and gloss of the paint.

Q.2 (a) Solution:

(i) As per **IS:6461 (Part VII)-1973** workability can be defined as that property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, compacted and finished.

The factors affecting the workability of concrete are:

(a) Water Content: Water content in a given volume of concrete will have significant influence on the workability. The higher the water content per cubic meter of concrete, the higher will be the fluidity of concrete, which is one of the important factors affecting workability. It should be noted that from the desirability point of view, increase of water content is the last recourse to be taken for improving the workability even in the case of uncontrolled concrete.

(b) Mix Proportions: Aggregate cement ratio is an important factor influencing workability. The higher the aggregate cement ratio, the leaner is the concrete. In lean concrete, less quantity of paste is available for providing lubrication per unit surface area of aggregate and hence the mobility of aggregate is restrained which gives bad workability. In case of rich concrete, the phenomenon is reversed.

(c) Size of Aggregates: The bigger the size of aggregate, the less is the surface area and hence less amount of water is required for wetting the surface and less matrix of paste is required for lubricating the surface to reduce internal friction. For a given quantity of water and paste, bigger size of aggregate will give higher workability.

(d) Shape of Aggregates: The shape of aggregates influences the workability in of concrete. Angular, elongated or flaky aggregates make the concrete very harsh when compared to rounded aggregates or cubical shaped aggregates. Contribution to better workability of rounded aggregate will come from the fact that for the given volume or weight, it will have less surface area and less voids than angular or flaky aggregate. Not only that, being round in shape, the frictional resistance is greatly reduced.

(e) Surface Texture: The influence of surface texture on workability is again due to the fact that the total surface area of rough textured aggregate is more than the surface area of smooth rounded aggregate of same volume. Thus the rough textured aggregate will show poor workability and smooth or glassy textured aggregate will give better workability.

(f) **Grading of Aggregates:** This is one of the factor which will have maximum influence on workability. A well graded aggregate is the one which has least amount of voids in a given volume. Other factors being constant, when the total voids are less, excess paste is available to give better lubricating effect giving high workability, also well graded aggregates provide domino effect which provides better workability.

(g) **Use of Admixtures:** This is the most important factor which affects the workability. Admixtures reduce the internal friction between the particles resulting in better workability.

(ii) Let volume of cement be V_c m^3, volume of sand be V_s m^3, volume of aggregate be V_a m^3, weight of cement be W_c kN, weight of sand be W_s kN, weight of aggregate be W_a kN, weight of water be W_w kN

Given that $V_c : V_s : V_a = 1 : 1.5 : 3$

$$\therefore \quad \frac{V_c}{V_s} = \frac{1}{1.5}$$

$$\Rightarrow \quad V_s = 1.5\,V_c \quad \text{...(i)}$$

$$\frac{V_c}{V_a} = \frac{1}{3} \Rightarrow V_a = 3V_c \quad \text{...(ii)}$$

Given, $\frac{W_w}{W_c} = 0.45$ and entrained air = 2.5%

Volume of concrete to be prepared = 1 m^3

Net volume of concrete considering air voids,

$$= 1 - \frac{2.5}{100} \times 1 = 0.975 \text{ m}^3$$

$$\therefore \quad 0.975 = V_c + V_s + V_a + \frac{W_w}{9810}$$

Using $\frac{W_w}{W_c} = 0.45$

$$\Rightarrow \quad W_w = 0.45\,W_c$$

$$V_s = 1.5V_c$$

$$V_a = 3V_c$$

$$\therefore \quad 0.975 = \frac{W_c}{3.15 \times 9810} + \frac{4.5 \times W_c}{3.15 \times 9810} + \frac{0.45 W_c}{9810}$$

$$\Rightarrow \quad 0.975 \times 9810 = \frac{W_c}{3.15} + \frac{4.5 W_c}{3.15} + W_c \times 0.45$$

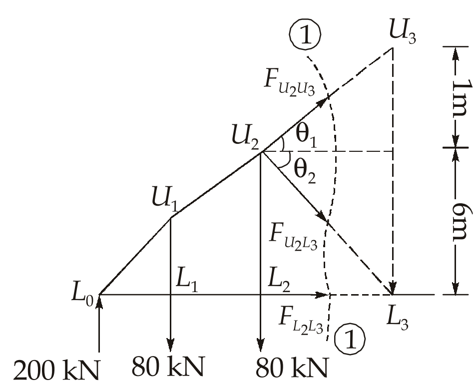
$$\Rightarrow \quad 9564.75 = 2.196\, W_c$$
$$\Rightarrow \quad W_c = 4355.53 \text{ N}$$
$$\therefore \quad V_c = \frac{4355.53}{3.15 \times 9810} = 0.1409 \text{ m}^3$$
$$V_s = 1.5\, V_c = 0.2114 \text{ m}^3$$
$$V_a = 3V_c = 0.4227 \text{ m}^3$$
$$V_s = \frac{W_s}{2.6 \times 9810} \Rightarrow W_s = 5391.97 \text{ N}$$
$$V_a = \frac{W_a}{2.5 \times 9810} \Rightarrow W_a = 10366.72 \text{ N}$$

Q.2 (b) Solution:

Due to symmetry, vertical reactions at both supports are,

$$\frac{1}{2} \times (80 \times 5) = 200 \text{ kN}$$

Consider the equilibrium of left side portion of section 1 - 1 as shown below. Let U_2U_3 and $U_2\, L_3$ make angles θ_1 and θ_2 respectively with the horizontal, then



$$\tan\theta_1 = \frac{(7-6)}{5} = 0.2$$
$$\Rightarrow \quad \theta_1 = 11.31°$$
$$\tan\theta_2 = \frac{6}{5} = 1.2$$
$$\Rightarrow \quad \theta_2 = 50.19°$$
$$\Sigma M_{u_2} = 0$$
$$\Rightarrow \quad 200 \times 10 - 80 \times 5 - F_{L_2L_3} \times 6 = 0$$

$$\Rightarrow \quad F_{L_2L_3} = 266.67 \text{ kN} \quad \text{(Tensile)}$$

$$\Sigma M_{L_3} = 0$$

$$\Rightarrow 200 \times 15 - 80 \times 10 - 80 \times 5 + F_{U_2U_3} \cos\theta_1 6 + F_{U_2U_3} \sin\theta_1 \times 5 = 0$$

$$\Rightarrow \quad F_{U_2U_3} = \frac{-200 \times 15 + 800 + 400}{6\cos 11.31° + 5\sin 11.31°} \quad (\because \theta_1 = 11.31°)$$

$$= -262.24 \text{ kN (compressive)}$$

Due to symmetry, $F_{U_3U_4} = 262.24$ kN (Compressive)

$$\Sigma H = 0$$

$$F_{U_2L_3} \cos\theta_2 + F_{U_2U_3} \cos\theta_1 + F_{L_2L_3} = 0$$

$$\Rightarrow F_{U_2L_3} \times \cos 50.19° - 262.24 \cos 11.31° + 266.67 = 0$$

$$\Rightarrow \quad F_{U_2L_3} = -14.87 \text{ kN (Compressive)}$$

Q.2 (c) Solution:

$$\Sigma F_y = 0$$

$$\Rightarrow \quad R_A + R_B = 48 + 10 \times 2 = 68 \text{ kN} \quad ...(i)$$

$$\Sigma M_B = 0$$

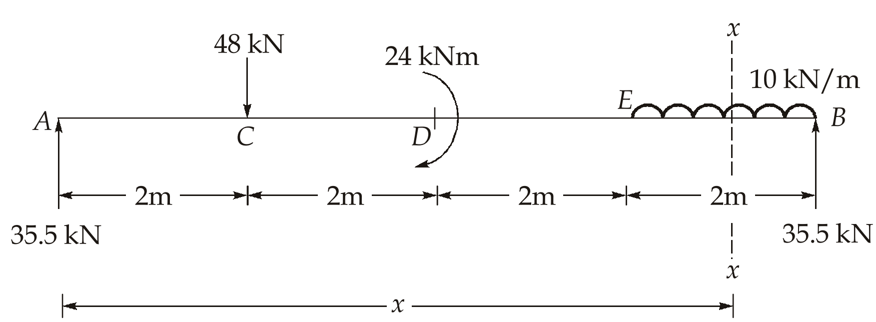
$$\Rightarrow \quad R_A \times 8 - 48 \times 6 + 24 - 10 \times 2 \times 1 = 0$$

$$\Rightarrow \quad R_A = 35.5 \text{ kN}$$

$\therefore$ From (i) $R_B = 68 - 35.5 = 32.5$ kN

Slope and deflection:

Consider a section x–x at a distance x from A in portion EB



$$M_x = 35.5x/-48\,(x-2)/\ +24\,(x-4)^\circ/\ -\frac{10(x-6)^2}{2}$$

$$\Rightarrow \quad EI\frac{d^2y}{dx^2} = 35.5x/-48\,(x-2)/+24\,(x-4)^\circ\ /\ -\frac{10(x-6)^2}{2} \quad \text{...(i)}$$

Integrating (i), we get

$$EI\frac{dy}{dx} = \frac{35.5x^2}{2} + C_1/-\frac{48(x-2)^2}{2}/+24(x-4)/-\frac{10(x-6)^3}{6} \quad \text{...(ii)}$$

Again integrating (ii), we get

$$EIy = 35.5\frac{x^3}{6} + C_1x + C_2/-\frac{48(x-2)^3}{6}/ + \frac{24(x-4)^2}{2}/-\frac{10(x-6)^4}{24} \quad \text{...(iii)}$$

Using end conditions,

At $x = 0$, $y = 0$

$$\therefore \quad C_2 = 0$$

Also at $x = 8$ m, $y = 0$

From equation (iii)

$$0 = \frac{35.5\times 8^3}{6} + C_1\times 8 - \frac{48(8-2)^3}{6} + \frac{24(8-4)^2}{2} - \frac{10(8-6)^4}{24}$$

$\Rightarrow C_1 = -185.83$

Substituting the values of C_1 and C_2 in (ii) and (iii), we get

$$EI\frac{dy}{dx} = \frac{35.5x^2}{2} - 185.33/-\frac{48(x-2)^2}{2}/+24(x-4) - \frac{10(x-6)^3}{6} \quad \text{...(iv)}$$

$$EIy = \frac{35.5x^2}{6} - 185.83x/-\frac{48(x-2)^3}{6}/+\frac{24(x-4)^2}{2} - \frac{10(x-6)^4}{24} \quad \text{...(v)}$$

Slope and deflection at point *E*

At point E, $x = 6$m

∴ Slope at point *E*

$$\frac{dy}{dx} = \theta_E = \frac{1}{EI}\left[\frac{35.5\times 6^2}{2} - 185.83 - \frac{48(6-2)^2}{2} + 24(6-4) - 0\right]$$

$$\Rightarrow \theta_E = \frac{117.47}{EI} = \frac{117.47}{18000} = 6.54 \times 10^{-3} \text{ radian}$$

$\therefore\ \theta_E = 6.54 \times 10^{-3}$ radian

Deflection at point *E*

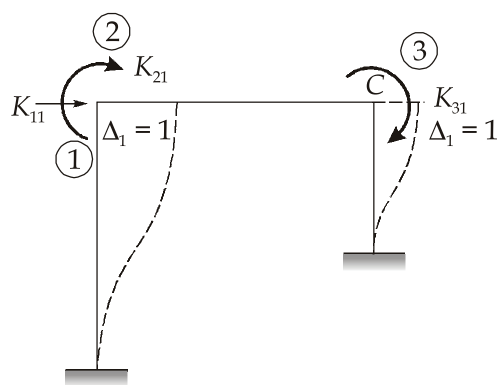
$$y = y_E = \frac{1}{EI}\left[\frac{35.5 \times 6^3}{6} - 185.83 \times 6 - \frac{48(6-2)^3}{6} + \frac{24(6-4)^2}{2} - 0\right]$$

$$\Rightarrow \quad y_E = -\frac{300.98}{EI} = -\frac{300.98}{18000} = -0.01672 \text{ m}$$

$$\Rightarrow \quad y_E = 16.72 \text{ mm} \quad \text{(downward)}$$

Q.3 (a) Solution:

The stiffness matrix can be developed by giving a unit displacement successively at coordinates 1, 2 and 3 without any displacement at other coordinates and determining the forces required at all the coordinates. To generate first column of the stiffness matrix, give a unit displacement at coordinate 1 as shown below:



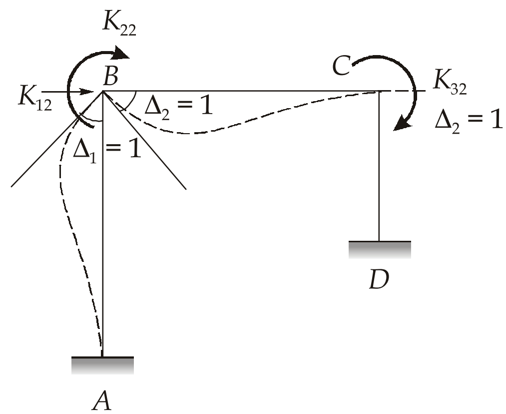
The force required at the coordinates are:

$$K_{11} = \frac{12E(4I)}{(10)^3} + \frac{12E(I)}{(5)^3} = 0.144\,EI$$

$$K_{21} = \frac{-6E(4I)}{(10)^2} = -0.24\,EI$$

$$K_{31} = \frac{-6\,EI}{(5)^2} = -0.24EI$$

To generate the second column of the stiffness matrix, give a unit displacement at coordinate 2 as shown below:

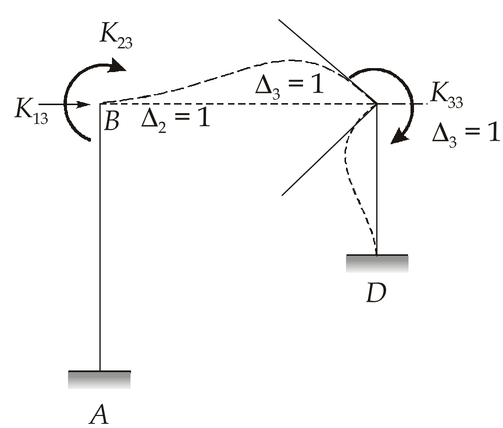


The forces required at coordinates are:

$$K_{12} = \frac{-6E(4I)}{(10)^2} = -0.24EI$$

$$K_{22} = \frac{4E(4I)}{10} + \frac{4E(4I)}{10} = 3.2EI$$

$$K_{32} = \frac{2E(4I)}{10} = 0.8EI$$



To generate the third column of the stiffness matrix, give a unit displacement at coordinate 3 as shown below. The forces required at the coordinates are:

$$K_{13} = \frac{-6E(4I)}{(10)^2} = -0.24EI$$

$$K_{23} = \frac{2E(4I)}{10} = 0.8EI$$

$$K_{33} = \frac{4E(4I)}{10} + \frac{4EI}{5} = 2.4EI$$

Hence, the required stiffness matrix [*K*] is

$$[K] = EI\begin{bmatrix} 0.144 & -0.24 & -0.24 \\ -0.24 & 3.20 & 0.80 \\ -0.24 & 0.80 & 2.40 \end{bmatrix}$$

Q.3 (b) Solution:

The load of 8 kN will give rise to a clockwise couple of 8 × 0.5 = 4 kNm at *C*.

The point *C* will be displaced by an amount δ.

Slope deflection equations:

There are two unknowns θ_C and δ. We shall assume θ_C to be positive as δ for *CA* is assumed positive and that for *CB*, is assumed negative.

$$M_{AC} = \frac{2EI}{3}\left(\theta_C - \frac{3\delta}{3}\right) \quad ...(i)$$

$$M_{CA} = \frac{2EI}{3}\left(2\theta_C - \frac{3\delta}{3}\right) \quad ...(ii)$$

$$M_{CB} = \frac{2EI}{2}\left(2\theta_C + \frac{3\delta}{2}\right) \quad ...(iii)$$

$$M_{BC} = \frac{2EI}{2}\left(\theta_C + \frac{3\delta}{2}\right)$$

...(iv)

Equilibrium equations:

As there are two unknowns, two equations will be required for finding out the values of unknowns. One equation will be provided by the fact that the clockwise couple at *C*, causes clockwise moments in *CA* and *CB*.

$$\therefore \quad M_{CA} + M_{CB} = 4$$

$$\Rightarrow \quad \frac{2EI}{3}\left(2\theta_C - \frac{3\delta}{3}\right) + \frac{2EI}{2}\left(2\theta_C + \frac{3\delta}{2}\right) = 4$$

$$\Rightarrow \quad \frac{4}{3}EI\theta_C - \frac{2EI\delta}{3} + 2EI\theta_C + \frac{3}{2}EI\delta = 4$$

$$\Rightarrow \quad 20EI\theta_C + 5EI\delta = 24$$

$$\Rightarrow \quad 20\theta_C + 5\delta = \frac{24}{EI} \qquad \text{...(v)}$$

Shear equation:

The couple acting at C also gives rise to horizontal reaction at A and B, the two being equal in magnitude but opposite in direction.

Now, horizontal reaction at $A = \dfrac{M_{AC} + M_{CA}}{3}$

and horizontal reaction at $B = \dfrac{M_{CB} + M_{BC}}{2}$

As the two are equal so,

$$\frac{M_{AC} + M_{CA}}{3} = \frac{M_{CB} + M_{BC}}{2}$$

$$\Rightarrow \quad \frac{\frac{2EI}{3}(\theta_C - \delta) + \frac{2EI}{3}(2\theta_C - \delta)}{3} = \frac{\frac{2EI}{2}\left(2\theta_C + \frac{3}{2}\delta\right) + \frac{2EI}{2}\left(\theta_C + \frac{3}{2}\delta\right)}{2}$$

$$\Rightarrow \quad \frac{4}{3}EI\theta_C - \frac{4}{3}EI\delta + \frac{8}{3}EI\theta_C - \frac{4}{3}EI\delta = 6EI\theta_C + \frac{9}{2}EI\delta + 3EI\theta_C + \frac{9}{2}EI\delta$$

$$\Rightarrow \quad 30\,EI\theta_C = -70\,EI\delta$$

$$\Rightarrow \quad \theta_C = -\frac{7}{3}\delta$$

Substituting θ_c in equation (v), we get

$$-\frac{140}{3}\delta + 5\delta = \frac{24}{EI}$$

$$\Rightarrow \quad \frac{-125\delta}{3} = \frac{24}{EI}$$

$$\Rightarrow \quad \delta = \frac{-0.576}{EI} \qquad \text{...(i)}$$

$$\therefore \quad \theta_C = -\frac{7}{3}\delta = -\frac{7}{3} \times \frac{0.576}{EI} = \frac{1.344}{EI}$$

Final moments:

The values of moments may now be found out by substituting the values of θ_C and δ in equations (i) to (iv).

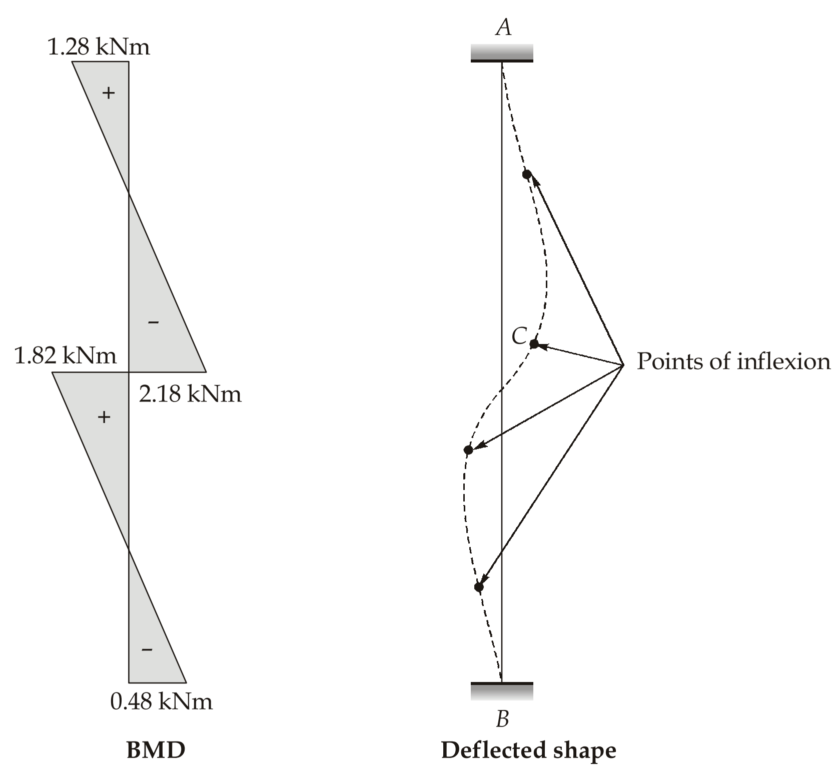
Thus,
$$M_{AC} = \frac{2EI}{3}\left(\frac{1.344}{EI} + \frac{0.576}{EI}\right) = 1.28 \text{ kNm}$$

$$M_{CA} = \frac{2EI}{3}\left(\frac{2 \times 1.344}{EI} + \frac{0.576}{EI}\right) = 2.18 \text{ kNm}$$

$$M_{CB} = \frac{2EI}{2}\left(\frac{2 \times 1.344}{EI} - \frac{3 \times 0.576}{2EI}\right) = 1.82 \text{ kNm}$$

and
$$M_{BC} = \frac{2EI}{2}\left(\frac{1.344}{EI} - \frac{3}{2} \times \frac{0.576}{EI}\right) = 0.48 \text{ kNm}$$

The BM diagram and deflected shape has been shown below.



Q.3 (c) Solution:

The beam *CD* is 10 m long. Let the two supports 6 m apart are at *A* and *B*.

Let x = distance of support *A* from *C* in metre then distance of support *B* from end $D = 10 - (6 + x) = (4 - x)$ m

First calculate the reactions R_A and R_B.

Taking moments about A, we get,

$$1 \times x \times \frac{x}{2} + R_B \times 6 = (10-x) \times 1 \times \frac{(10-x)}{2}$$

$$\Rightarrow \quad \frac{x^2}{2} + 6R_B = \frac{(10-x)^2}{2}$$

$$\Rightarrow \quad x^2 + 12\,R_B = (10-x)^2$$

$$\Rightarrow \quad x^2 + 12R_B = 100 + x^2 - 20x$$

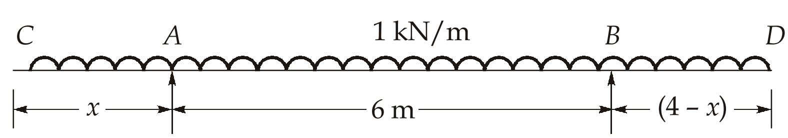
$$R_B = \frac{5}{3}(5-x)$$

$$\therefore \quad R_A = \text{Total load} - R_B = 10 \times 1 - \frac{5}{3}(5-x) = \frac{5}{3}(1+x)$$

In the present case of overhang beam, the maximum negative B.M. will be at either of the two supports and the maximum positive B.M will be in the span. If BM on the beam is as small as possible, then the length of the overhang portion should be so adjusted that the maximum negative B.M at the support is equal to the maximum positive B.M in the span AB.

The BM will be maximum in the span AB at a point where SF is zero.

Let BM is maximum (or SF is zero) at a section in AB at a distance y m from C.



But SF at this section, $\quad SF = y \times 1 - R_A = 0$

$$\Rightarrow \quad y \times 1 - R_A = 0$$

$$\Rightarrow \quad y \times 1 - \frac{5}{3}(1+x) = 0$$

$$\Rightarrow \quad y = \frac{5}{3}(1+x) \qquad \text{...(i)}$$

Now, $\quad$ BM at the support $A = -1 \times x \times \frac{x}{2} = -\frac{x^2}{2}$ $\qquad$ ...(ii)

And BM at a distance y from C,

$$= -1 \times y \times \frac{y}{2} + R_A(y-x) = -\frac{y^2}{2} + \frac{5}{3}(1+x)(y-x)$$

$$= -\frac{1}{2}\left[\frac{5}{3}(1+x)\right]^2 + \frac{5}{3}(1+x)\left[\frac{5}{3}(1+x) - x\right]$$

$$= \frac{5}{3}(1+x)\left[\frac{-5-5x+10+10x-6x}{6}\right]$$

$$= \frac{5}{3}(1+x)\left[\frac{5-x}{6}\right] = \frac{5}{18}\left[-x^2+4x+5\right] \quad \text{...(iii)}$$

For the condition that the BM shall be as small as possible, the hogging moment at the support A and the maximum sagging moment is the span AB should be numerically equal.

∴ Equating equations (ii) and (iii) and ignoring the negative sign of BM at A, we get

$$\frac{5}{18}\left(-x^2+4x+5\right) = \frac{x^2}{2}$$

$$\Rightarrow \quad -5x^2 + 20x + 25 = 9x^2$$

$$\Rightarrow \quad 14x^2 - 20x - 25 = 0$$

Solving for x, we get $x = 2.23$ m (Neglecting –ve value)

Substituting this value of x in equation (i), we get,

$$y = \frac{5}{3}(1+2.23) = \frac{5 \times 3.23}{3} = 5.38 \text{ m}$$

Now S.F. and B.M diagrams can be drawn as shown in figure.

SF diagram:

SF at $C = 0$

SF just on LHS of $A = -1 \times 2.23 = -2.23$ kN

(Shear force varies between linearly C and A)

SF just on LHS of $A = -2.23 + R_A = -2.23 + 5.38 = 3.15$ kN

SF just on LHS of $B = +3.15 - 1 \times 6 = -2.85$ kN

(shear force between A and B varies by a straight line law)

SF just on RHS of $B = -2.85 + R_B = -2.85 + 4.62 = +1.17$ kN

SF at $D = 1.17 - 1 \times 1.77 = 0$

(SF between B and D varies by a straight line law)

BM diagram BM at $C = 0$

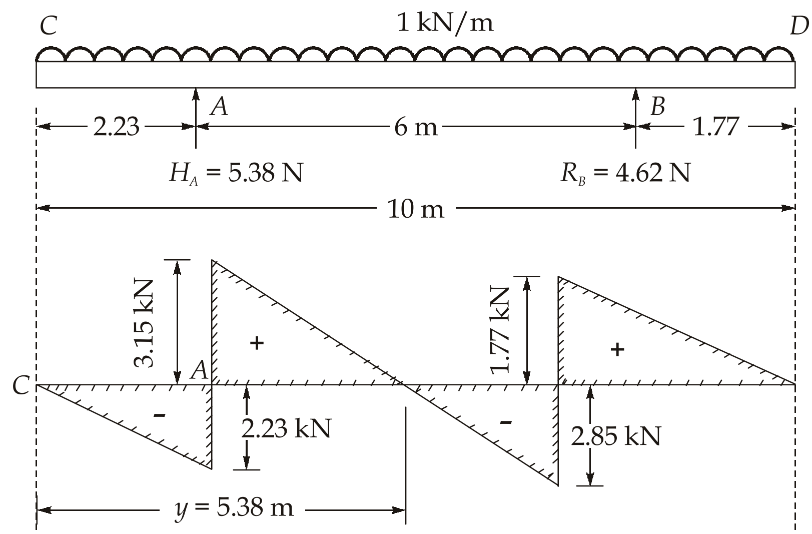
$$\text{BM at } A = -1 \times 2.23 \times \frac{2.33}{2} = -2.49 \text{ kNm}$$

BM at E i.e. at a distance $y = 5.38$ m from point C

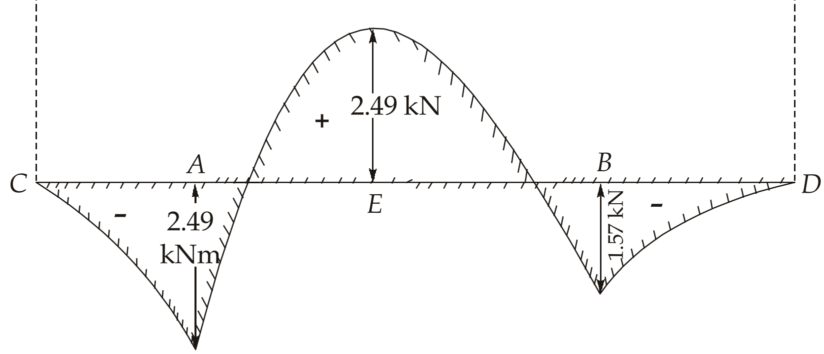
$$= -1 \times 5.38 \times \frac{5.38}{2} + R_A(5.38 - 2.23)$$

$$= 2.49 \text{ kNm} \simeq \text{BM at } A$$

$$\text{BM at } B = -1 \times 1.77 \times \frac{1.77}{2} = -1.57 \text{ kNm}$$



S.F. diagram



B.M. diagram

Q.4 (a) Solution:

(i) **Caluse 10.1** of **IS 456 : 2000** provides quality assurance measures to produce good quality concrete.

- Quality assurance measures are both technical and organizational.
- The job of quality control and assurance would involve quality audit of both the inputs as well as outputs.
- Inputs are in the form of materials for concrete; workmanship in all stages of batching, mixing, transportation, placing, compaction and curing; and related plant machinery and equipment; resulting in the output in the form of concrete in place.
- To ensure proper performance, it is necessary that each step in concreting which will be covered by next step is inspected as the work proceeds.
- Each party involved in the realization of a project should establish and implement a quality assurance plan.
- Quality assurance plan should generally include :
  - (a) test reports and manufactures' certificate for materials, concrete mix design details.
  - (b) pour cards for site organization and clearance for concrete placements.
  - (c) record of site inspection of workmanship, field tests.
  - (d) non-conformance reports, change orders.
  - (e) quality control charts
  - (f) statistical analysis.
- Volume batching may be allowed only where weight batching is not possible.
- The accuracy of the measuring equipment shall be within ± 2% of the quantity of cement being measured and within ± 3% of quantity of aggregate, admixtures and water being measured.
- Concrete shall be mixed in a mechanical mixer. The mixing time shall be atleast 2 min.

Repair : The main purpose of repairs is to bring back the architectural shape of the building so that all services start working and the functioning of building is resumed quickly. Repair does not pretend to improve structural strength of building. e.g. : Patching up of defects such as cracks.

Restoration : It is the restitution of strength the building had before the damage occurred. The main purpose is to carry out structural repairs to load bearing elements. It may involve cutting portions of elements and rebuilding them, inserting temporary supports, etc. e.g. : Injecting epoxy like material, which is strong in tension, into the cracks, in walls, etc.

Rehabilitation : Rehabilitation methods, in addition to restoring structural integrity and shape, mitigate or stop the process responsible for the damage. Because rehabilitation includes addressing the cause of the problem itself, and thus subsequent repairs last significantly longer.

(ii) Constituents of a good brick earth:

1. **Alumina :** 20% to 30%
2. **Silica :** 50% to 60%
3. **Lime :** Not exceeding 5%
4. **Oxides of iron :** 5% to 6%
5. **Magnesia :** Small quantity.

Test for bricks:

1. **Absorption:** A brick is taken and it is weighed dry. It is then immersed in water for a period of 16 hours. It is weighed again and the difference in weight indicates the amount of water absorbed by the brick. It should not exceed 20% of weight of dry brick.
2. **Crushing strength:** It is found out by placing brick in compression testing machine. It is pressed till it breaks. As per IS : 1077 - 1970, the minimum crushing strength of bricks is 3.50 N/mm^2.
3. **Hardness:** A scratch is made on brick surface with finger nail. If no impression is left on the surface, the brick is treated to be sufficiently hard.
4. **Presence of soluble salts:** Brick is immersed in water for 24 hours. It is then taken out and allowed to dry in shade. The absence of grey or white deposits on its surface indicates absence of soluble salts.
5. **Shape and Size:** Brick is closely inspected. It should be of standard size and its shape should be truly rectangular with sharp edges.
6. **Soundness:** Two bricks are struck with each other. The bricks should not break and clear ringing metallic sound should be produced.
7. **Structure:** A brick is broken and its structure is examined. It should be homogeneous, compact and free from any defects such as holes, lumps, etc.

Q.4(b) Solution:

Since loading is uniform loading. Cable will take parabolic shape.

Assuming A as origin, equation of cable is:

$$y = ax^2 + bx + c$$

At A,

$$0 = a(0)^2 + b(0) + c$$

$\Rightarrow \quad c = 0 \quad \text{... (i)}$

At C, $\quad -8 = a(80)^2 + b(80)$

$$\Rightarrow \quad 80a + b = \frac{-8}{80} \quad \text{... (ii)}$$

$\therefore$ B is the lowest point and $\left.\frac{dy}{dx}\right|_B = 0$

$\Rightarrow \quad 0 = 2ax_1 + b \quad \text{... (iii)}$

and $\quad -16 = ax_1^2 + bx_1 \quad \text{... (iv)}$

From equation (iii) and (iv) $\frac{16}{x_1} = ax_1$,

$$b = \frac{-32}{x_1} \quad \text{... (v)}$$

From equation (ii) and (v)

$$80 \times \frac{16}{x_1^2} - \frac{32}{x_1} = \frac{-8}{80}$$

$$\Rightarrow \quad x_1^2 - 320x_1 + 12800 = 0$$

$$x_1 = 273.14$$

$$x_2 = 46.86 \text{ m}$$

Neglecting $x_1 = 273.14$

$$\therefore \quad a = \frac{16}{(46.86)^2}, \quad b = \frac{-32}{46.86}$$

$\therefore$ Equation of cable is: $\quad y = \frac{16x^2}{(46.86)^2} - \frac{-32x}{46.86}$

At $x = 0$, $\quad \frac{dy}{dx}\theta_A = \tan^{-1}\left(\frac{32}{(46.86)^2}\right) = 34.32°$

At $x = 80$ m, $\quad \frac{dy}{dx}\theta_B = \tan^{-1}(1.8487) = 61.59°$

Tension at B, $\quad \Sigma M_A = 0$

$$\Rightarrow \quad \frac{40 \times (46.86)^2}{2} = T_B \times 16$$

$\Rightarrow$ $T_B = 2744.82$ kN

Tension at A, $T_A = \dfrac{T_B}{\cos 34.32} = \dfrac{2744.82}{\cos 34.32} = 3323.42$ kN

Tension at C, $T_C = \dfrac{T_B}{\cos 61.59°} = \dfrac{2744.82}{\cos 61.59°} = 5769.12$ kN

Q.4 (c) Solution:

At section 1-1, applied load is eccentric

Let us analyse this section 1-1.

Net area of the plate, $A = (20 \times 150) - (20 \times 40) = 2200 \text{ mm}^2$

Distance of the centroid from the bottom edge

$$= \frac{(20 \times 150 \times 75) - (20 \times 40 \times 125)}{2200} = 56.82 \text{ mm}$$

Moment of inertia of the section about the centroidal axis XX,

$$I = \left[\frac{20 \times 150^3}{12} + 20 \times 150 \times (75 - 56.82)^2\right] - \left[\frac{20 \times 40^3}{12} + 40 \times 20(125 - 56.82)^2\right]$$

$$= [5.625 \times 10^6 + 0.9915 \times 10^6] - [0.1067 \times 10^6 + 3.719 \times 10^6]$$

$$= 2.791 \times 10^6 \text{ mm}^4$$

Direct stress $-\dfrac{P}{A} = -\dfrac{120 \times 10^3}{2200} = -54.55 \text{ N/mm}^2$ (Tensile)

Eccentricity of the load, $e = 75 - 56.82 = 18.18$ mm

Stress at top due to eccentricity of the load,

$$= -\frac{120 \times 10^3 \times 18.18}{2.791 \times 10^6} \times 93.18$$

$$= -72.83 \text{ N/mm}^2 \text{ (Tensile)}$$

Stress at bottom due eccentricity of the load,

$$= \frac{120 \times 10^3 \times 18.18}{2.791 \times 10^6} \times 56.82$$

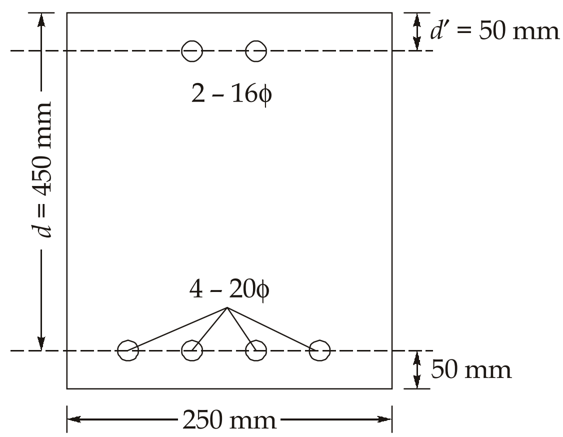
$$= 44.41 \text{ N/mm}^2 \text{ (Comp.)}$$

Resultant stress at top $= -54.55 - 72.83 = -127.38 \text{ N/mm}^2$ (Tensile)

Resultant stress at bottom $= -54.55 + 44.41 = -10.14 \text{ N/mm}^2$ (Tensile)

Q.5 (a) Solution:

Given: For a doubly reinforced beam:



M20, Fe415.

Limiting depth of neutral axis

$$\therefore \quad x_{u,\lim} = 0.48\,d = 0.48 \times 450 = 216\text{mm}$$

$$f_{ck} = 20\ \text{N/mm}^2$$

$$f_y = 415\ \text{N/mm}^2$$

Determination of actual depth of neutral axis:

$$C = T$$

$$\Rightarrow 0.36\,f_{ck}\,x_u b + (f_{sc} - 0.45\,f_{ck})\,A_{sc} = 0.87\,f_y\,A_{st} \qquad \text{...(i)}$$

$$\frac{d'}{d} = \frac{50}{450} = 0.111$$

From table given, using linear interpolation

$$f_{sc} = 350 + \frac{347.5 - 353}{0.125 - 0.100}(0.111 - 0.100) = 350.58\ \text{N/mm}^2$$

From equation (i)

$$\therefore \quad 0.36 \times 20 \times x_u \times 250 + (350.58 - 0.45 \times 20) \times 2 \times \frac{\pi}{4}(16)^2$$

$$= 0.87 \times 415 \times 4 \times \frac{\pi}{4}(20)^2$$

$$\Rightarrow \quad x_u = 175.75\ \text{mm} < x_{u,\lim}$$

$\therefore$ Moment of resistance,

$$\text{M.O.R} = 0.36\,f_{ck}\,x_u b\,(d - 0.42\,x_u) + (f_{sc} - 0.45\,f_{ck})\,A_{sc}\,(d - d')$$

$$= 0.36 \times 20 \times 175.75 \times 250\,(450 - 0.42 \times 175.75)$$

$$+(350.58 - 0.45 \times 20) \times 2 \times \frac{\pi}{4}(16)^2 \times (450 - 50)\,\text{Nmm}$$

$$= 173.95 \text{ kNm}$$

Q.5 (b) Solution:

For steel of grade Fe410 (E250)

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

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

Tension component, $P_T = P \times \cos 45° = 300 \times \frac{1}{\sqrt{2}} = \frac{300}{\sqrt{2}}$ kN.

Factored tensile force in one bolt,

$$T_b = \frac{P_T}{n} = \frac{300/\sqrt{2}}{6} = 35.355 \text{ kN}$$

Shear component $P_V = 300 \times \sin 45° = \frac{300}{\sqrt{2}}$ kN

∴ Factored shear force on one bolt,

$$V_{sb} = \frac{P_V}{n} = \frac{\frac{300}{\sqrt{2}}}{6} = 35.355 \text{ kN}$$

Design strength of bolt:

Here bolts are in single shear,

Shear strength of bolt, $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \times 0.78\, A_{sb}$

$$= \frac{400}{\sqrt{3} \times 1.25} \times 0.78 \times \frac{\pi}{4} \times (20)^2 \times 10^{-3} \text{ kN} = 45.27 \text{ kN}$$

Bearing strength of bolt $V_{dpb} = 2.5\, k_b\, dt \frac{f_u}{\gamma_{m1}}$

$$k_b = \text{minimum} \left\{ \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}; 1 \right\}$$

Adopt $e = 1.5d_o$, $p = 2.5d$

$$k_b = 0.5$$

$$V_{dpb} = 2.5 \times 0.5 \times 20 \times 10 \times \frac{410}{1.25} \times 10^{-3} \text{ kN} = 82 \text{ kN}$$

Design strength of bolt V_{db} = minimum $\begin{cases} V_{dsb} \\ V_{dpb} \end{cases}$

$$= \text{minimum} \begin{cases} 45.27 \text{ kN} \\ 82 \text{ kN} \end{cases} = 45.27 \text{ kN}$$

Tensile strength of bolt $T_{db} = 0.9 \dfrac{f_{ub}}{\gamma_{m1}} \times A_{nb}$

$$= 0.9 \times \frac{400}{1.25} \times 0.78 \times \frac{\pi}{4}(20)^2 \times 10^{-3} \text{ kN} = 70.57 \text{ kN}$$

$$\leq \frac{f_{yb}}{\gamma_{mo}} \times A_g = \frac{240}{1.1} \times \frac{\pi}{4} \times (20)^2 \times 10^{-3} \text{ kN} = 68.54 \text{ kN}$$

$\therefore$ $T_{db} = 68.54$ kN

For bolts subjected to tension and shear, the following interaction formula must be satisfied:

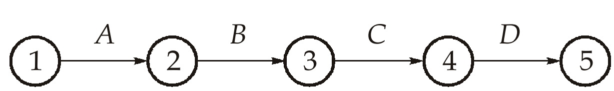
$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

$$\text{L.H.S} = \left(\frac{35.355}{45.27}\right)^2 + \left(\frac{35.355}{68.54}\right)^2 = 0.876 \leq 1.0$$

Thus, the bolts connecting the T-bracket to the column face are safe.

Q.5 (c) Solution:

(i)



Expected mean time $t_E = t_o + t_m + t_p$

$$t_{E_A} = \frac{9 + 4 \times 14 + 21}{6} = 14.33 \text{ days}$$

$$(t_E)_B = \frac{7 + 4 \times 21 + 32}{6} = 20.5 \text{ days}$$

$$(t_E)_C = \frac{10 + 4 \times 20 + 30}{6} = 20 \text{ days}$$

$$(t_E)_D = \frac{25 + 4 \times 35 + 42}{6} = 34.5 \text{ days}$$

Therefore expected time of project,

$$t_p = 14.33 + 20.5 + 20 + 34.5 = 89.33 \text{ days}$$

(ii) Standard deviation $\sigma = \dfrac{t_p - t_0}{6}$

$$\sigma_A = \frac{21 - 9}{6} = 2$$

$$\sigma_B = \frac{32 - 7}{6} = 4.17$$

$$\sigma_C = \frac{30 - 10}{6} = 3.33$$

$$\sigma_D = \frac{42 - 25}{6} = 2.83$$

Therefore standard deviation of project,

$$\sigma_P = \sqrt{\sigma_A^2 + \sigma_B^2 + \sigma_C^2 + \sigma_D^2}$$

$$\sigma_P = \sqrt{2^2 + 4.17^2 + 3.33^2 + 2.83^2} = 6.36$$

Z for 95% probability is 1.647

Z for 5% probability is –1.647

Using $z = \dfrac{T - T_P}{\sigma_P}$

For 95% probability $1.647 = \dfrac{\text{T} - 89.33}{6.36}$

$\Rightarrow$ $T = 99.805$ days

For 5% probability $-1.647 = \dfrac{T - 89.34}{6.36}$

$\Rightarrow$ $T = 78.855$ days

Standard deviation for A is least ($\sigma_A = 2$).

Therefore activity A has the most reliable time estimates.

Q.5 (d) Solution:

Given: Diameter of rivet $d = 22\text{mm}$

Gross diameter of rivet $d_g = 22 + 1.5 = 23.5\text{mm}$

For section along *abc*:

Deduction in width for hole, $= 1 \times 23.5 = 23.5\text{mm}$

For section along *abde:*

$$\text{Deduction in width for hole, } = nd_g - \frac{p^2}{4g} = 2 \times 23.5 - \frac{(50)^2}{4 \times 75} = 38.67 \text{ mm}$$

∴ Maximum deduction for hole, in 200mm leg,

$= 38.67\text{mm}$

Now, one leg (long leg) of the anlge is connected to gusset plate;

$$A_{net} = A_1 + k \times A_2$$

$$k = \frac{3A_1}{3A_1 + A_2}$$

$$A_1 = \text{Net area of connected leg} = \left(200 - 38.67 - \frac{t}{2}\right)t$$

$$A_2 = \text{Area of outstanding leg} = \left(100 - \frac{t}{2}\right)t$$

$$P = \sigma_{at} \times A_{net}$$

$$\Rightarrow 350 \times 10^3 = 150 \times \left\{\left(161.33 - \frac{t}{2}\right)t + \frac{3\left(161.33 - \frac{t}{2}\right)t}{3 \times \left(161.33 - \frac{t}{2}\right)t + \left(100 - \frac{t}{2}\right)t} \times \left(100 - \frac{t}{2}\right)t\right\}$$

After solving, we get $t = 9.91 \text{ mm} \simeq 10\text{mm}$

Thus, provide 200mm × 100mm × 10mm angle section.

Q.5 (e) Solution:

$$d' = 50 + \frac{35}{2} = 67.5 \text{ mm}$$

$$D = 450\text{mm}$$

$$\frac{d'}{D} = \frac{67.5}{450} = 0.15$$

$$\frac{P_u}{f_{ck} \times bD} = \frac{2500 \times 10^3}{25 \times 450 \times 450} = 0.494$$

$$\frac{M_u}{f_{ck}\,bD^2} = \frac{180\times10^6}{25\times450\times450^2} = 0.079$$

From chart given, $\frac{P_t(\%)}{f_{ck}} = 0.10$

Percentage of reinforcement $= 0.10 \times 25 = 2.5\%$

$$A_{sc} = \frac{2.5}{100}\times450\times450 = 5062.5\ \text{mm}^2$$

Q.6 (a) Solution:

(i) Types of loss of prestress:

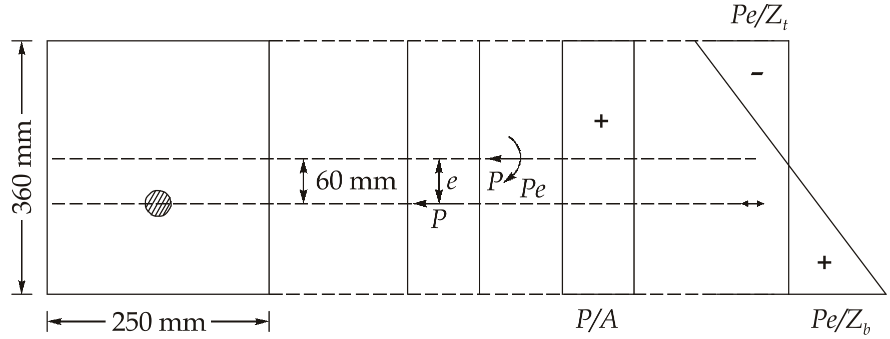
(a) In pretensioned member:

S.No	Loss due to
1.	Elastic deformation of concrete
2.	Relaxation of stress in steel
3.	Shrinkage of concrete
4.	Creep of concrete

(b) In post tensioned member:

S.No	Loss due to
1.	No losses due to elastic deformation if all the wires are simultaneously tensioned. If the wires are successively tensioned than there will be loss of prestress due to elastic deformation of concrete.
2.	Relaxation of stress in steel
3.	Shrinkage of concrete
4.	Creep of concrete
5.	Friction
6.	Anchorage slip

(ii)



Given, Initial prestress = $1250\ \text{N/mm}^2$

Area of steel wires = $350\ \text{mm}^2$

∴ Prestressing force,

$$P = \frac{1250 \times 350}{1000}\ \text{kN} = 437.5\ \text{kN}$$

Stress in concrete at the level of steel,

$$= \frac{P}{A} + \frac{Pe^2}{I}$$

$$= \frac{437.5 \times 10^3}{250 \times 360} + \frac{437.5 \times 10^3 \times (60)^2}{\frac{250 \times (360)^3}{12}} = 6.48\ \text{N/mm}^2$$

Modular ratio $m = \frac{E_s}{E_c} = \frac{210}{35} = 6$

(a) Loss of stress for the pretensioned beam:

(i) Loss of stress due to elastic shortening of concrete

$$= m \times f_c = 6 \times 6.48 = 38.88\ \text{N/mm}^2$$

(ii) Loss of stress due to creep of concrete

$$= 45 \times 10^{-6} \times 6.48 \times 210 \times 10^3 = 61.24\ \text{N/mm}^2$$

(iii) Loss of stress due to shrinkage of concrete

$$= 300 \times 10^{-6} \times 210 \times 10^3 = 63\ \text{N/mm}^2$$

(iv) Loss of stress due to relaxation of steel stress

$$= \left(\frac{5}{100} \times 1250\right) = 62.50\ \text{N/mm}^2$$

∴ Total loss of stress = $225.62\ \text{N/mm}^2$

∴ Percentage loss of stress

$$= \frac{225.62}{1250} \times 100 = 18.05\%$$

(b) Loss of stress for the post tensioned beam:

(i) Loss of stress due to elastic shortening of concrete = 0

(ii) Loss of stress due to creep of concrete

$$= 22 \times 10^{-6} \times 210 \times 10^3 \times 6.48 = 29.94\ \text{N/mm}^2$$

(iii) Loss of stress due to shrinkage of concrete

$$= 215 \times 10^{-6} \times 210 \times 10^3 = 45.15\ \text{N/mm}^2$$

(iv) Loss due to relaxation of stress in steel

$$= \frac{5}{100} \times 1250 = 62.50 \text{ N/mm}^2$$

(v) Loss of stress due to a anchorage slip

$$= \frac{1.25}{12 \times 1000} \times 210 \times 10^3 = 21.88 \text{ N/mm}^2 \quad (\because \alpha = 0)$$

(vi) Loss of stress due to friction effect

$$= f_0 (kx) = 1250 \times 0.0015 \times 12 = 22.5 \text{ N/mm}^2$$

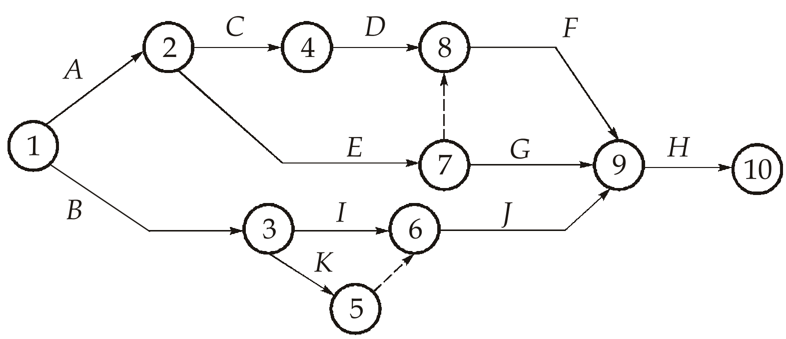
∴ Total Loss of stress = 181.97 N/mm^2

∴ Percentage loss of stress

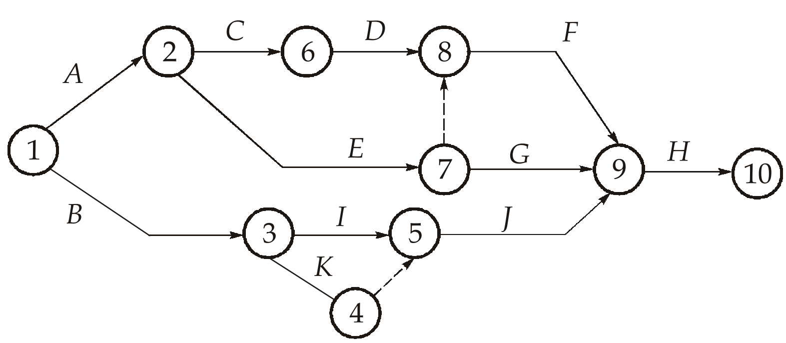
$$= \frac{181.97}{1250} \times 100 = 14.56\%$$

Q.6 (b) Solution:

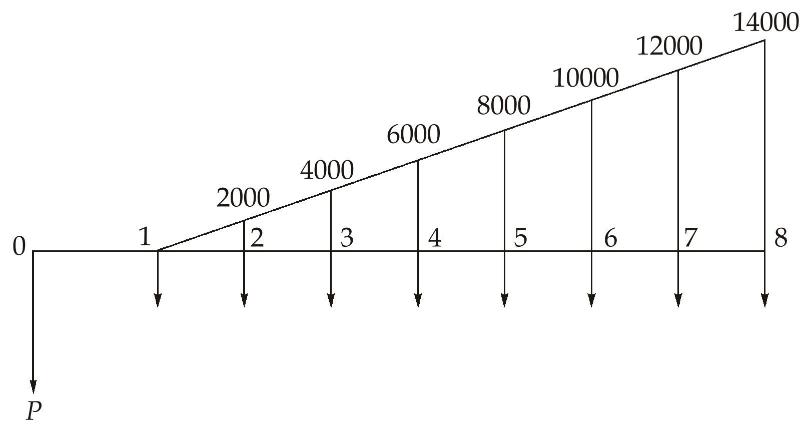
(i) Case: (i)



Case: (ii)



(ii) Cash flow diagram



Given n = 0.8, $i = \frac{20}{2} = 10\%$ (compounded semi-annually)

Using equation $A = g\left(\frac{A}{G}, i, n\right)$

where g = Rs.2000

$$A = g\left[\frac{1}{i} - \frac{n}{i} \times \frac{i}{(1+i)^n - 1}\right]$$

$\Rightarrow$
$$A = 2000\left[\frac{1}{0.1} - \frac{8}{0.1} \times \frac{0.1}{(1+0.1)^8 - 1}\right]$$

= 2000 × 3.0045 = Rs. 6009

$$P = 6009 \times \left(\frac{P}{A}, 10\%, 8\right)$$

$$= 6009 \times \left[\frac{(1+i)^n - 1}{i(1+i)^n}\right] = 6009 \times \left[\frac{(1+0.1)^8 - 1}{0.1(1+0.1)^8}\right]$$

= 6009 × 5.3349 $\simeq$ Rs. 32058

Q.6 (c) Solution:

Total factored load = 1.5 × 40 = 60 kN

Maximum bending moment $= \frac{WL}{8} = \frac{60 \times 4}{8} = 30$ kNm

Maximum shear force $= \frac{W}{2} = \frac{60}{2} = 30$ kN

Plastic section modulus required,

$$Z_{P,\text{required}} = \frac{M \times \gamma_{m0}}{f_y} = \frac{30 \times 10^6 \times 1.1}{250} = 132 \times 10^3 \text{mm}^3$$

Try ISMB 175 @ 191 N/m

$$Z_P = 166.1 \times 10^3 \text{ mm}^3 > 132 \times 10^3 \text{ mm}^3 \quad \text{(OK)}$$

Section classification

$$\frac{b}{t_f} = \frac{b_f/2}{t_f} = \frac{90/2}{8.6} = 5.23 < 9.4\,\epsilon$$

where

$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{d}{t_w} = \frac{h - 2(t_f + R_1)}{t_w} = \frac{175 - 2(8.6 + 10)}{5.5} = 25.05 < 84\,\epsilon$$

So section is plastic.

Check for shear capacity:

Design shear strength of section,

$$V_d = \frac{f_y}{\sqrt{3} \times 1.1} \times h \times t_w = \frac{250}{\sqrt{3} \times 1.1} \times 175 \times 5.5 \times 10^{-3}$$

$$= 126.29 \text{ kN} > 30 \text{ kN (safe)}$$

$$0.6V_d = 0.6 \times 126.29 = 75.77 \text{ kN} > 30 \text{ kN}$$

Hence it is a low shear case.

Check for shear buckling of web:

$$\frac{d}{t_w} = \frac{175 - 2(8.6 + 10)}{5.5} = 25.05 < 67\epsilon \quad \text{(safe)}$$

Check for design bending strength

∴ Section is plastic and thus $\beta_b = 1$

$$M_d = \frac{\beta_b f_y Z_p}{\gamma_{m0}} = \frac{1 \times 166.1 \times 10^3 \times 250}{1.1 \times 10^6} \text{ kNm}$$

$$= 37.75 \text{ kNm} > 30 \text{ kNm} \quad \text{(Safe)}$$

Check for deflection

$$\delta = \frac{5wL^4}{384EI} = \frac{5 \times 40 \times 10^3 \times (4000)^3}{384 \times 2 \times 10^5 \times 1272 \times 10^4} = 13.1 \text{ mm}$$

$$\delta_{\max} = \frac{L}{300} = \frac{4000}{300} = 13.33 \text{ mm} > 13.1 \text{ mm} \quad \text{(Safe)}$$

Q.7 (a) Solution:

(i) Scraper is a machine which can scrap the ground and load it simultaneously, transport it over the required distance, dump at the desired place and then roll the dumped material over the required area in required level and return to the pit for the next cycle. So, the scraper is self-sufficient and self operating construction equipment designed to dig, load, dump and spread the earth. But it is not suited for (i) hard rock (ii) certain sands which will not pile up into a scraper (iii) and wet or muddy material which make discharging of a scraper difficult.

Basic parts of a scaper:

1. **Bowl:** It is a pan to hold scraped material and is capable of tilting down for digging or ejecting. The size of the bowl describes the size of the scraper.
2. **Cutting Edge:** The bowl has a cutting edge attached at the bottom to make shallow cut.
3. **Apron:** This is a wall in front of the bowl which opens and closes to regulate the flow of earth
4. **Tail gate or Ejector:** It is the rear of the pan which is capable of forward and backward movement inside the bowl

Types of Scrapers

Depending upon the type of the tractor used, scrapers are classified as:

(a) Crawler tractor scraper: Used for short and difficult haul

(b) Wheel tractor scraper: Used for long and easy haul

(c) Motor scraper: Having its own engine and motoring arrangement

Size of a scraper:The size of a scraper may be specified by the struck or heaped capacity of the bowl, expressed in cubic meter.

Output of a scraper: The output of a scraper is the bank measure volume per hour (cubic meter/hr)

Output = Optimum loose volume per trip × S × $60/t$ × efficiency

where S = Swell factor depending upon type of soil

t = Cycle time per trip in minutes

t = Fixed time (Loading + Dumping and turning + Accelerating + declaration) + Haul time + Return time in minutes

Factors affecting output of scrapers:

The output of scraper depends upon the following main factors:

(1) Size and mechanical conditions of the scraper

(2) Characteristics of the soil to be handled by the scraper and work area

(3) Size and conditions of borrow pit or cut

(4) Slope of loading zone

(5) Extent of loosening material, prior to loading for hard rock.

(ii) Grouting is a corrective step by applying a suitable binding material made for unifying the rock mass into a solid one.

Purpose of grouting"

The purposes of grouting area enumerated as follows:

1. To solidify and strengthen the foundation in order to increase its capacity to support a load.
2. To reduce or eliminate the flow of water through a formation particularly in the case of dam or tunnel
3. To reduce the hydrostatic uplift under a dam.

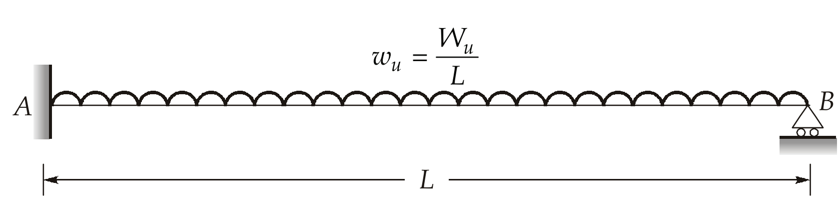
The most satisfactory method to explore the possibility of whether a foundation should be grouted or not, is the physical verification of the core sample.

The core samples may be obtained with diamond or shoot drills, usually diamond for the smaller sizes and shoot for the larger sizes.

S.No.	Physical nature of core sample	Remarks
1.	A core sample shows long and continuous pieces, with little loss in length compared with the depth of the hole.	It indicates a reasonably solid formation needing little or no grouting.
2.	A core sample is badly broken showing discontinuous pieces and the measured length of the core sample is small in proportion to the depth of the hole.	It indicates a broken and bad foundation conditions and must require a good quality of grouting.

Q.7 (b) Solution:

(i)



To determine collapse load (W_u):

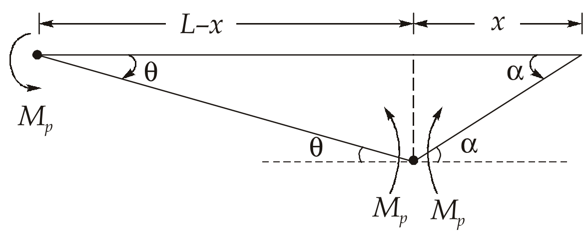
Degree of static indeterminacy $D_s = 1$.

$\therefore$ Number of plastic hinges required for collapse,

$$n = D_s + 1 = 1 + 1 = 2$$

Let us assume that one plastic hinge gets formed at A and other in between AB at a distance 'x' from B.

Mechanism:



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

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

By the principle of virtual work,

$$W_i = W_E$$

$$\Rightarrow \quad 2M_p\theta + M_p\alpha = w_u \times \left[\frac{1}{2} \times L \times x\alpha\right]$$

$$\Rightarrow \quad 2M_P\theta + M_P \times \left(\frac{L-x}{x}\right)\theta = w_u \times \left[\frac{1}{2} \times L \times x \times \frac{L-x}{x}\theta\right]$$

$$\Rightarrow \quad M_P\left(2 + \frac{L-x}{x}\right) = w_u \times \left[\frac{1}{2} \times L \times (L-x)\right]$$

$$\Rightarrow \quad w_u = \frac{2M_P \times (L+x)}{Lx \times (L-x)} \qquad \text{...(i)}$$

For collapse load w_u to be minimum,

$$\frac{dw_u}{d_x} = 0$$

$$\Rightarrow \quad x \times (L - x) \times 1 - (L + x)(L - 2x) = 0$$

$$\Rightarrow \quad Lx - x^2 - \{L^2 + Lx - 2xL - 2x^2\} = 0$$

$$\Rightarrow \quad x^2 + 2xL - L^2 = 0$$

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

$$= \frac{-2L \pm 2\sqrt{2}\,L}{2} = (\sqrt{2} - 1)L = 0.414\,L$$

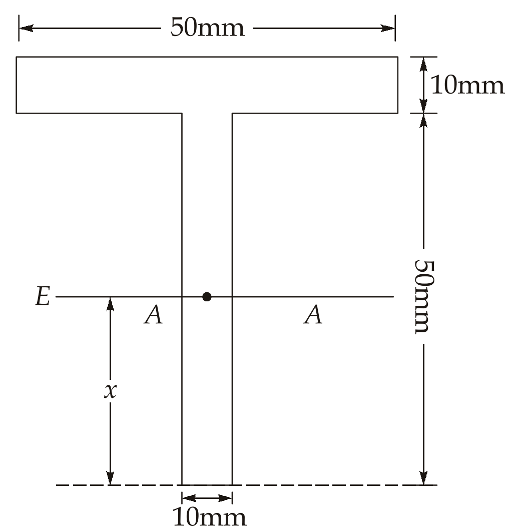
$\therefore$ From equation (i) $$w_u = \frac{2M_P \times (L + 0.414\,L)}{L \times 0.414L \times (L - 0.414\,L)} = 11.656\,M_P / L^2$$

$\Rightarrow$ $$\frac{W_u}{L} = 11.656\,M_p/L^2$$

$\Rightarrow$ $$W_u = 11.656\,M_p/L$$

(ii) $$M_P = f_y\,z_p$$

z_p = Plastic modulus of the section



Let equal area axis lies at a distance of x mm from bottom of web,

$$x \times 10 = 10 \times (50 - x) + 500$$

$\Rightarrow$ $$20x = 1000$$

$\Rightarrow$ $$x = 50 \text{ mm}$$

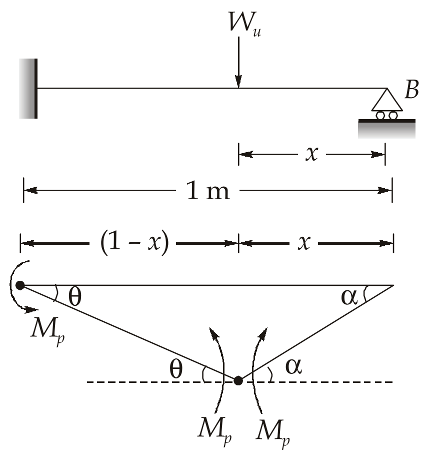
Hence, equal area axis lies at the junction of flange and web,

$\therefore$ Plastic section modulus,

$$z_p = \frac{A}{2} \times [\bar{y}_1 + \bar{y}_2]$$

$$= \frac{500 + 500}{2} \times [5 + 25] = 500 \times 30 = 1500 \text{ mm}^3$$

$\therefore \quad M_P = f_y \times z_p = 250 \times 15000 \times 10^{-6} = 3.75 \text{ kNm}$



$$x\alpha = (1-x)\theta$$

$$\Rightarrow \quad \alpha = \left(\frac{1-x}{x}\right)\theta$$

By principle of virtual work, $W_i = W_E$

$$\Rightarrow \quad 2M_p\theta + M_p\alpha = W_u \times x\alpha$$

$$\Rightarrow \quad 2M_P\theta + M_P \times \left(\frac{1-x}{x}\right) \times \theta = W_u \times x \times \left(\frac{1-x}{x}\right)\theta$$

$$\Rightarrow \quad M_P \times \left(\frac{2x+1-x}{x}\right) = W_u \times (1-x)$$

$$\Rightarrow \quad W_u = M_P \times \frac{(1+x)}{x(1-x)}$$

For W_u to be minimum, $\frac{dW_u}{dx} = 0$

$$\Rightarrow \frac{x(1-x)(1)-(1+x)(1-2x)}{\{x(1-x)\}^2} = 0$$

$$\Rightarrow \quad x - x^2 - (1 - 2x + x - 2x^2) = 0$$

$$\Rightarrow \quad x^2 + 2x - 1 = 0$$

$$\therefore \quad x = \frac{-2 \pm \sqrt{(2)^2 - 4 \times 1(-1)}}{2 \times 1} = \frac{-2 \pm \sqrt{8}}{2} = -1 \pm \sqrt{2}$$

Taking $x = \sqrt{2} - 1 = 0.414$ m

$$\therefore \quad W_u = \frac{M_P(1+0.414)}{0.414(1-0.414)} = 5.83\, M_P$$

$$= 5.83 \times 3.75 = 21.86 \text{ kN.}$$

Q.7 (c) Solution:

Given: $\dfrac{\text{Span}}{\text{Effective depth}} = 23$

Modification factor = 1.43

∴ Effective depth of slab required

$$= \frac{\text{Span}}{23 \times 1.43} = \frac{3000}{23 \times 1.43} = 91.21 \text{ mm} \simeq 92 \text{ mm (say)}$$

Providing 10 mm ϕ bars at a clear cover of 15 mm,

Effective cover = 15 + 5 = 20 mm

∴ Overall depth of slab required = 92 + 20 = 112 mm

∴ Provide an overall depth of slab as 115 mm

∴ Actual effective depth, d = 115 – 20 = 95 mm

Load calculation:

DL of the slab = 0.115 × 25 = 2.875 kN/m^2

Floor finish = 0.04 × 24 = 0.96 kN/m^2

Total dead load = 3.835 kN/m^2

∴ Factored dead load, w_d = 1.5 × 3.835 = 5.7525 kN/m^2

Live load = 3 kN/m^2

∴ Factored live load, w_l = 1.5 × 3 = 4.5 kN/m^2

Consider 1 m wide strip of the slab

B.M for end span:

B.M at the centre of the end span

$$= \frac{w_d L_1^2}{12} + \frac{w_l \times L_1^2}{10}$$

$$= \frac{5.7525 \times 3^2}{12} + \frac{4.5 \times 3^2}{10} = 8.364 \text{ kNm}$$

B.M on penultimate support $= -\left(\frac{w_d \times L_1^2}{10} + \frac{w_l \times L_1^2}{9}\right)$

$$= -\left(\frac{5.7525 \times 3^2}{10} + \frac{4.5 \times 3^2}{9}\right) = -9.677 \text{ kNm}$$

B.M for intermediate span:

B.M at the centre of the span $= \frac{w_d \times L^2}{16} + \frac{w_l \times L^2}{12}$

$$= \frac{5.7525 \times 3^2}{16} + \frac{4.5 \times 3^2}{12} = 6.61 \text{ kNm}$$

B.M on interior support: $= -\left(\frac{w_d \times L^2}{10} + \frac{w_l \times L^2}{9}\right)$

$$= -\left(\frac{5.7525 \times 3^2}{12} + \frac{4.5 \times 3^2}{9}\right) = -8.814 \text{ kNm}$$

Check for depth:

During bending moment, M_u = Greatest of the above bending moment

$= 9.677$ kNm

$BM_u = M_{u,\lim}$ $(= 0.138 f_{ck} bd^2)$ (for Fe415)

$\Rightarrow$ $9.677 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$\Rightarrow$ $d = 59.21 \text{ mm} < 95 \text{ mm}$

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

Flexural steel:

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

$$= \frac{0.5 \times 20 \times 1000 \times 95}{415}\left(1 - \sqrt{1 - \frac{4.6 \times 9.677 \times 10^6}{20 \times 1000 \times 95^2}}\right)$$

$= 302.187 \text{ mm}^2$

$\therefore$ Spacing of 10 mm ϕ bars $= \dfrac{1000 \times \frac{\pi}{4}(10)^2}{302.187} = 259.90$ mm c/c

$\therefore$ Provide 10 mm ϕ bars @ 240 mm c/c

$\therefore$ Actual area of steel provided $= \dfrac{1000 \times \frac{\pi}{4}(10)^2}{240} = 327.25 \text{ mm}^2$

Distribution steel $= \dfrac{0.12}{100} \times 1000 \times 115 = 138 \text{ mm}^2$

Using 8 mm ϕ bars as distribution steel ,

$$\text{Spacing} = \frac{1000 \times \frac{\pi}{4}(8)^2}{138} = 364.24 \text{ mm}$$

Provide 8mmϕ bars @ 300 mm c/c

Check for shear:

Maximum shear force occurs at the penultimate support and is equal to 0.6 times of total load on the end span.

$$\therefore \quad V_u = 0.6\,(5.7525 + 4.5) \times 3 = 18.45 \text{ kN}$$

Nominal shear stress $\tau_v = \dfrac{V_u}{bd} = \dfrac{18.45 \times 10^3}{1000 \times 95} = 0.194 \text{ N/mm}^2$

Design shear strength of concrete:

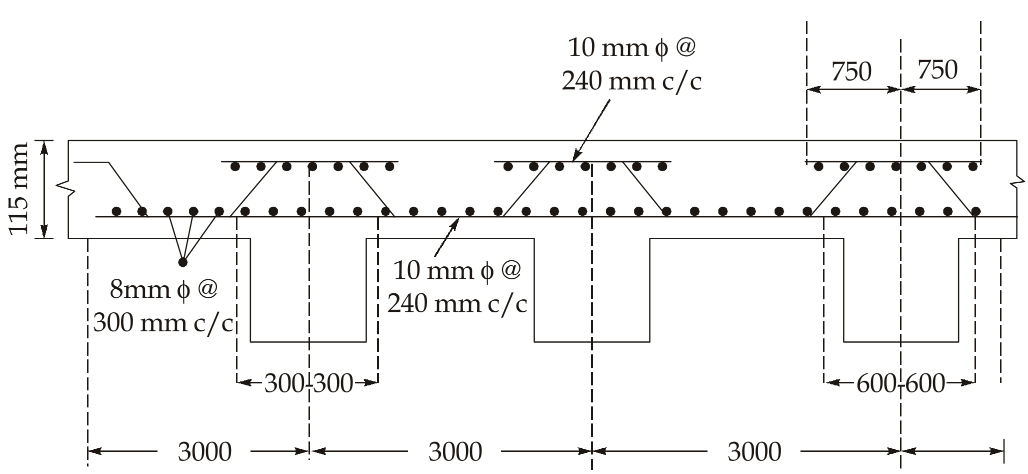
Percentage of tensile reinforcement provided.

$$P_t(\%) = \frac{A_{st}}{bd} \times 100 = \frac{327.25}{1000 \times 95} \times 100 = 0.34\%$$

From table given $\tau_c = 0.36 + \dfrac{0.48 - 0.36}{0.50 - 0.25}(0.34 - 0.25) = 0.403 \text{ N/mm}^2$

Since $\tau_v < \tau_c$ and thus slab is safe in shear

Reinforcement details:



(All dimensions in mm)

Q.8 (a) Solution:

Given: Steel is of grade Fe410 (E250)

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

Angle section is 50 × 50 × 6 mm

$\therefore \quad A_g = (50 + 50 - 6) \times 6 = 564 \text{ mm}^2$

$g = 28 \text{ mm}$

Given M-12 bolts,

$\therefore \quad d = 12 \text{ mm}, \quad d_0 = 13 \text{ mm}$

Design tensile strength based on yielding,

$$T_{dg} = f_y/\gamma_m \times A_g$$

$$= \frac{250}{1.1} \times 564 \times 10^{-3} \text{ kN} = 128.2 \text{ kN}$$

Design tensile strength based on rupture,

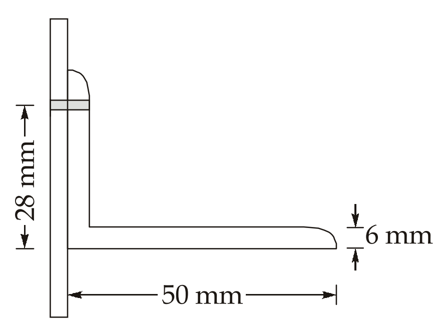
$$T_{dn} = 0.9 \frac{f_u}{\gamma_{m1}} A_{nc} + \beta \times \frac{f_y}{\gamma_{mo}} A_{go}$$

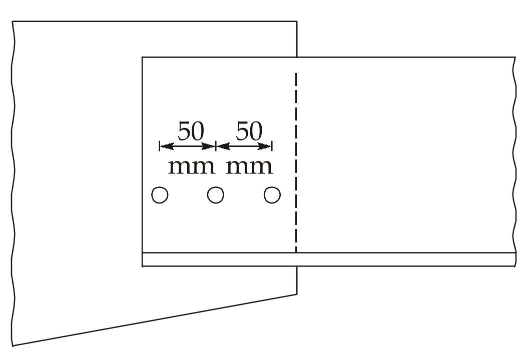
$$w = 50 \text{ mm}$$

$$b_s = w + g - t$$

$$= 50 + 28 - 6 = 72 \text{ mm}$$

$$L_c = 2 \times 50 = 100 \text{ mm}$$





$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{50}{6} \times \frac{250}{410} \times \frac{72}{100}$$

$$= 1.122 \geq 0.7 \text{ or } \leq 0.9\frac{\frac{f_u}{\gamma_{m1}}}{\frac{f_y}{\gamma_{mo}}} = \frac{0.9 \times \frac{410}{1.25}}{\frac{250}{1.1}} = 1.299$$

$$\therefore \quad \beta = 1.122$$

Net area of connected leg:

$$A_{nc} = (50 - 13 - 6/2) \times 6 = 204 \text{ mm}^2$$

Gross area of outstanding leg,

$$A_{go} = (50 - 6/2) \times 6 = 282 \text{ mm}^2$$

$$\therefore \quad T_{dn} = \left(0.9 \times \frac{410}{1.25} \times 204 + 1.122 \times \frac{250}{1.1} \times 282\right) \times 10^{-3} \text{ kN}$$

$$= 132.13 \text{ kN}$$

∴ Design tensile strength of the member:

$$T_d = \text{minimum} \begin{cases} T_{dn} \\ T_{dg} \end{cases} = 128.2 \text{ kN}$$

Q.8 (b) Solution:

Given: Size of column : 300mm × 500mm

P_u = 1000 kN; M_{ux} = 120 kNm

q_u = 200 kN/m^2 ; M20, Fe415

Determination of size of footing

Assume weight of the footing plus backfill to constitute about 15% of P_u, resultant eccentricity of loading at footing base

$$e = \frac{120 \times 10^3}{1000 \times 1.15} = 104 \text{ mm}$$

Assuming $e < L/6$ (i.e. $L > 6 \times 104 = 624$ mm)

$$\therefore \quad q_u = \frac{1000 \times 1.15}{BL} + \frac{120}{\left(\frac{BL^2}{6}\right)} \leq (200 \times 1.5)$$

$$\Rightarrow \quad 300\,BL^2 - 1150L - 720 \leq 0 \qquad \text{... (i)}$$

An economical proportion of the base slab is generally one in which the projection beyond the face of column (or pedestal) is approximately equal in both the directions

(For effective two-way behaviour, i.e. $\frac{L-a}{2} \simeq \frac{B-b}{2}$).

$$\text{Provide } B = 2\text{m}$$

$$\therefore \quad 600\,L^2 - 1150\,L - 720 \leq 0$$

$$\therefore \quad L = 2.414\text{m}$$

Provide $L = 2.45$m

$\therefore$ Projection in short direction,

$$= \frac{2000 - 300}{2} = 850 \text{ mm}$$

Projection in longer direction $= \frac{2450 - 500}{2} = 975$ mm

Thickness of footing:

Factored (net) soil pressure,

$$q_{u,\max} = \frac{1000}{2 \times 2.45} + \frac{120}{\frac{2 \times (2.45)^2}{6}} = 264.1\,\text{kN/m}^2$$

$$q_{u,\min} = \frac{1000}{2 \times 2.45} - \frac{120}{\frac{2 \times (2.45)^2}{6}}$$

$$\Rightarrow \quad q_{u,\min} = 204.1 - 60 = 144.1 \text{ kN/m}^2$$

For one-way shear:

The critical section is located 'd' distance away from the column face,

The average pressure contributing to the factored one-way shear is:

$$q_u = 264.1 - 60 \times \left\{\frac{(975 - d)}{2}\right\} / 1225$$

$$= (240.2 + 0.02449\,d) \text{ kN/m}^2$$

Assuming $d = 600$ mm

$\therefore \quad q_u = 0.255 \text{ N/mm}^2$

$\therefore \quad V_{u_1} = 0.255 \times 2000 \times (975 - d) = (497250 - 510\, d)$

Assuming $\tau_c = 0.36$ MPa (For M20 concrete with $P_t\% = 0.25$)

$\therefore \quad V_{uc} = 0.36 \times 2000 \times d = 720d$

$V_{u_1} \leq V_{uc}$

$\Rightarrow \quad 497250 - 510d \leq 720d$

$\Rightarrow \quad d \geq 404.3 \text{ mm} \simeq 404 \text{ mm}$

For two-way shear:

The critical section is located at '$d/2$' distance from the periphery of the column all around.

The average pressure contributing to the factored two-way shear:

$$q_u = 204.1 \text{ kN/m}^2 \simeq 0.2041 \text{ N/mm}^2$$

$$V_{u_2} = 0.2041\,[2000 \times 2450 - (300 + d)\,(500 + d)]$$

Assuming $d = 404$ mm

(The minimum depth required for one way shear)

$\therefore \quad V_{u_2} = 870 \times 10^3$ N

For two way shear limiting shear stress of concrete,

$$\tau_{c_z} = k_\beta \times 0.25\sqrt{f_{ck}}$$

$$k_\beta = 0.5 + \frac{b}{a} = 0.5 + \frac{300}{500}\,;\ \ngtr 1 = 1$$

$$\therefore \quad \tau_{c_z} = 1 \times 0.25\sqrt{20} = 1.118 \text{ N/mm}^2$$

$$V_{u_c} = 1.118 \times [(300 + d) + (500 + d)] \times 2 \times d$$

$$= [1788.8\, d + 4.472\, d^2] \text{ N}$$

Substitue $d = 404$ mm

$$V_{u_c} = 1452.58 \text{ kN} > V_{u_2} = 870 \text{ kN}$$

Hence, one way shear governs the footing slab thickness and $d \geq 404$ mm,

Assuming a clear cover of 75 mm and diameter of bar as 16 mm,

$$D \geq 404 + 75 + \frac{16}{2} = 487 \text{ mm}$$

Provide $D = 500$ mm

Effective depth of footing:

Long span, $d_x = 500 - 75 - 8 = 417$ mm

Short span, $d_y = 417 - 16 = 401$ mm

Check for maximum soil pressure:

$$q_{max} = \frac{1000}{2 \times 2.45} + \{24 \times 0.5 + 18 \times (1.25 - 0.5)\} \times 1.5 + \frac{120 \times 6}{2 \times 2.45^2}$$

$$= 302 \text{ kN/m}^2 \simeq 200 \times 1.5 = 300 \text{ kN/m}^2$$

Design of flexural reinforcement:

The critical sections for moment are located at the faces of the column in both directions (*XX* and *YY*)

Along long span:

Cantilever projection = 975 mm; width = 2000 mm

d_x = 417mm ; $q_u = 0.2163$ N/mm^2 at face of column, 0.2641 N/mm^2 at footing edge.

$$M_{ux} = \left(0.2163 \times 2000 \times \frac{975^2}{2}\right) + (0.2641 - 0.2163) \times \frac{1}{2} \times 2000 \times 975^2 \times \frac{2}{3}$$

$$= (205.6 + 30.3) \times 10^6 = 236 \times 10^6 \text{ Nmm} = 236 \text{ kNm}$$

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

$$= \frac{0.5 \times 20 \times 2000 \times 417}{415}\left(1 - \sqrt{1 - \frac{4.6 \times 236 \times 10^6}{20 \times 2000 \times 417^2}}\right)$$

$$= 1634.78 \text{ mm}^2$$

$$P_t(\%) = \frac{1634.78}{2000 \times 417} \times 100 = 0.196\% < 0.25\%$$

$$\therefore \qquad (A_{st})_{req} = \frac{0.25}{100} \times 1000 \times 417 = 2085 \text{ mm}^2$$

$$\text{No. of 16 mm dia. bars} = \frac{2085}{\frac{\pi}{4}(16)^2} \simeq 11$$

$$\text{Spacing} = \frac{(2000 - 75 \times 2 - 16)}{10} = 183 \text{ mm}$$

Provide 11 nos. 16mmϕ bars in long direction

Along short span:

Cantilever projection = 850 mm

Width = 2450mm

d_y = 401 mm

q_u varies along the section YY with an average value of 0.2041 N/mm^2 at the middle. (Consider mean of value at centre and at footing edge)

$$q_u = \frac{(0.2041 + 0.2641)}{2} = 0.2341 \text{ N/mm}^2$$

$$M_{uy} = 0.2341 \times 2450 \times \frac{850^2}{2} = 207.2 \text{ kNm}$$

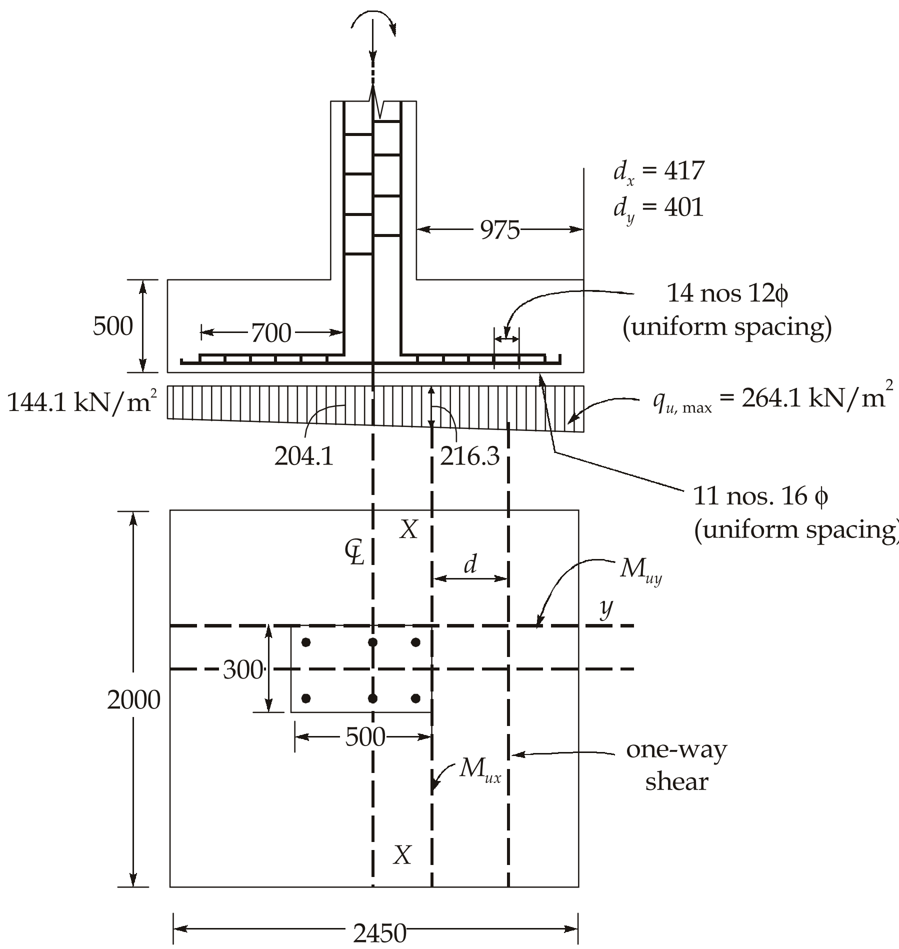
$$A_{st} = \frac{0.5 \times 20 \times 2450 \times 401}{415}\left(1 - \sqrt{1 - \frac{4.6 \times 207.2 \times 10^6}{20 \times 2450 \times 401^2}}\right)$$

$$= 1477.97 \text{ mm}^2$$

$$P_t(\%) = \frac{1477.97}{2450 \times 401} \times 100 = 0.15\%$$

Using 12ϕ bars

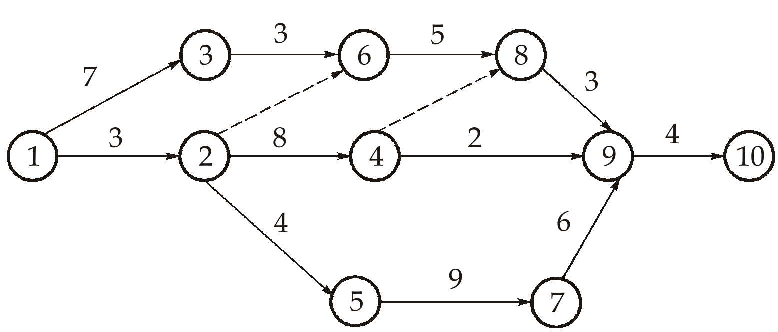
$$\text{No. of bars required} = \frac{1477.97}{\frac{\pi}{4}(12)^2} \simeq 14$$



(All dimensions in mm)

Q.8 (c) Solution:

(i) Original Network

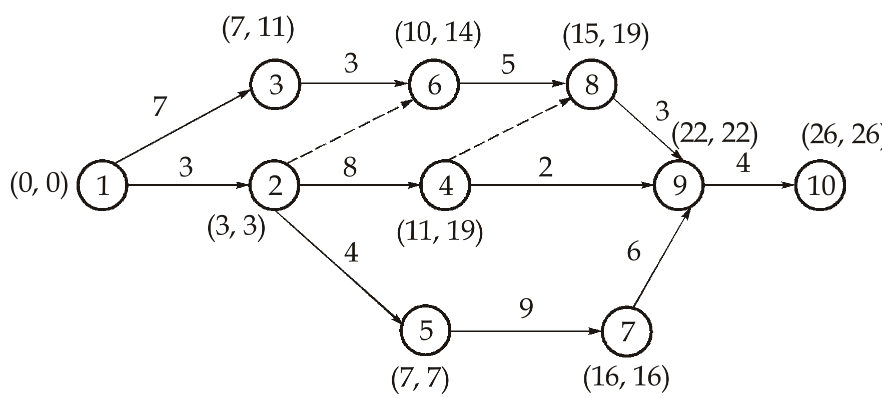


(Activity duration in days)

Let us find out the critical path of network.

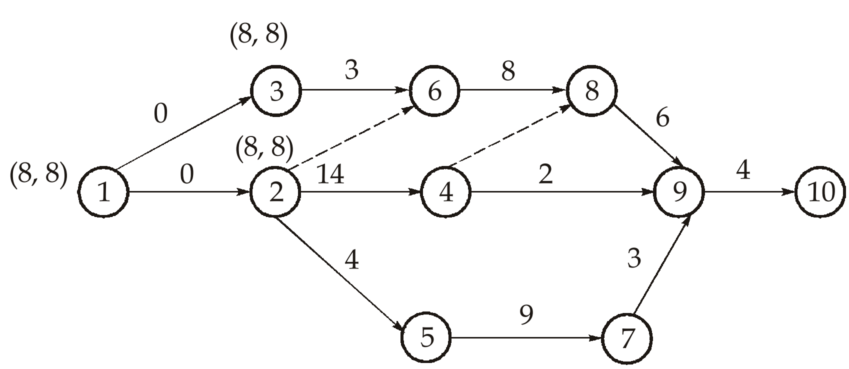
Path	**Days**
1 - 3 - 6 - 8 - 9 - 10	22
1 - 2 - 6 - 8 - 9 - 10	15
1 - 2 - 4 - 8 - 9 - 10	18
1 - 2 - 4 - 9 - 10	17
1 - 2 - 5 - 7 - 9 - 10	26 (Critical path)

$\therefore$ Critical path of project is 1 - 2 - 5 - 7 - 9 - 10 with project completion duration of 26 days



Project updation:

1. At the end of 8 days, activities 1-2 and 1 - 3 are completed
2. Activities 6 - 8, 2 - 4, 8 - 9 and 7 - 9 are rescheduled and their revised duration are 8, 14, 6, 3 days respectively. Apply project updating.



Again finding out the critical path of network.

Path	Days
1 - 3 - 6 - 8 - 9 - 10	29
1 - 2 - 6 - 8 - 9 - 10	26
1 - 2 - 4 - 8 - 9 - 10	32
1 - 2 - 4 - 9 - 10	28
1 - 2 - 5 - 7 - 9 - 10	28

Updation has led to a new critical path 1 - 2 - 4 - 8 - 9 - 10 with a project duration of 32 days. On comparing it with the original network, we find that on updating the project, the project duration has increased by 6 days.

(ii) Resource Levelling: In project management, resource levelling is defined by a guide named as Project Management Body Kf knowledge (PMBOK guide) as "A technique in which start and finish dates are adjusted based on resource constraints with the goal of balancing demand for resources with the available supply".

In resource levelling, the main constraint would be the resources and if the maximum demand on any resource is not to exceed a certain limit, the activities will then have to be rescheduled so that the total demand on the resource at any time will be within limit. The project duration gets consequently exceeded.

Resource Loading: Resource loading also known as resource allocation simply means deciding what resources each activity of the project requires. For this, resource usage profile i.e. histogram is drawn either on earliest start of an activity or latest start of each activity basis.

The process of resource loading consists of fitting activities into pattern of resource availability within stipulated time period and it is achieved by following methods:

1. Resource smoothening
2. Resource levelling

Main difference between resource loading and resource levelling is that resource levelling relates project requirements with available resources while resource loading is the process of allocation of resources to various activities.

In allocation/resource loading, we have to deal with manpower only while both time and resources are involved in resource levelling.

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