MADE EASY

India's Best Institute for IES, GATE & PSUs

Detailed Solutions

ESE-2019 Mains Test Series

Civil Engineering Test No : 14

Section A

Q.1 (a) Solution:

(i)

Brick work is superior to stone work because of following reasons:

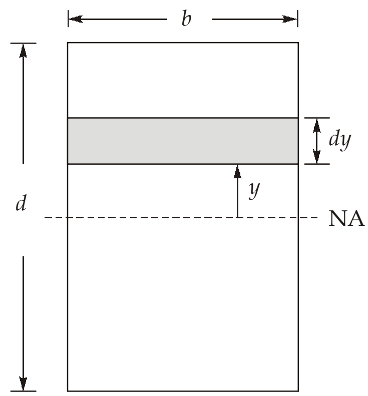
a. Bricks are more durable than stones.
b. Bricks resist fire better.
c. Bricks occupy less space due to their standard modular shape.
d. Can be used for cantilever wall construction because they are lighter than stones.
e. Cost of construction is lesser in brick work.
f. Complicated lifting machinary is not required for brick work as in stone work.

(ii)

Difference between wet process and dry process for manufacturing of cement:

S.No.	Dry Process	Wet Process
1.	In this process quite hard raw materials are used.	Here crushed raw materials are used, along with water.
2.	Economical process because less consumption of fuel in the kiln	Comparatively, process is not ecnonomical
3.	Short kiln are used.	Longer kilns are used and these are less responsive to a variable clinker demand.
4.	Control over homogeneity of the mixture is not good.	Control over homogeneity of the slurry is good.

Q.1 (b) Solution:



Consider an elementary area, dA (= bdy) located at y distance from N.A.,

$$dM = \sigma_y (dA) y$$

$$= (E \cdot \varepsilon)^{1/n} b \cdot dy \cdot y$$

$$\therefore \qquad M = 2\int_0^{d/2} (E\varepsilon)^{1/n} by\, dy$$

As the strain variation is linear,

$$\frac{\varepsilon}{y} = \frac{\varepsilon_{max}}{d/2}$$

$$\therefore \qquad \varepsilon = \frac{\varepsilon_{max}}{d/2} \cdot y = \frac{\sigma_{max}^n}{E} \cdot \frac{2y}{d} \qquad \left[\text{as } \sigma^n = \varepsilon E\right]$$

$$\Rightarrow \qquad M = 2\int_0^{d/2} \left(\sigma_{max}^n \cdot \frac{2y}{d}\right)^{1/n} by\, dy$$

$$\Rightarrow \qquad M = 2\left(\frac{d}{2}\right)^{-1/n} \cdot b \cdot \sigma_{max} \left[\frac{y^{2+\frac{1}{n}}}{2+\frac{1}{n}}\right]_0^{d/2}$$

$$\Rightarrow \qquad M = 2\sigma_{max} \cdot b \cdot \left(\frac{d}{2}\right)^2 \left(\frac{n}{2n+1}\right)$$

$$\Rightarrow \qquad \sigma_{max} = \frac{2n+1}{n} \cdot \frac{M}{2b\left(\frac{d}{2}\right)^2} = \frac{2n+1}{n} \cdot \frac{M}{bd^2/2}$$

But $$I = \frac{bd^3}{12}$$

$$\Rightarrow \quad \frac{bd^2}{2} = \frac{6I}{d}$$

$$\Rightarrow \quad \sigma_{max} = \frac{2n+1}{n} \cdot \frac{M}{6I/d}$$

$$= \frac{Md}{6In}(2n+1)$$

Q.1 (c) Solution:

Let ϕ_{xy} be the shear strain in xy plane,

Normal strain is given by,

$$\epsilon'_x = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\epsilon_x - \epsilon_y}{2}\cos 2\theta + \frac{\phi_{xy}}{2}\sin 2\theta$$

Here, $\theta = 45°$, $\epsilon_x = \epsilon_0$ and $\epsilon_y = \epsilon_{90°}$

Hence, $$\epsilon_{45°} = \frac{\epsilon_{0°} + \epsilon_{90°}}{2} + \frac{\epsilon_{0°} - \epsilon_{90°}}{2}\cos 90° + \frac{\phi_{xy}}{2}\sin 90°$$

$$\Rightarrow \quad 500 \times 10^{-6} = \frac{600 \times 10^{-6} + 200 \times 10^{-6}}{2} + \frac{\phi_{xy}}{2}$$

$$\Rightarrow \quad \phi_{xy} = 200 \times 10^{-6}$$

Principal strains are calculated as,

$$\epsilon_1 / \epsilon_2 = \frac{\epsilon_{0°} + \epsilon_{90°}}{2} \pm \frac{1}{2}\sqrt{(\epsilon_{0°} - \epsilon_{90°})^2 + \phi_{xy}^2}$$

$$= \frac{(600+200)\times 10^{-6}}{2} \pm \frac{1}{2}\sqrt{\left((600-200)\times 10^{-6}\right)^2 + \left(200 \times 10^{-6}\right)^2}$$

$$= 400 \times 10^{-6} \pm 223.61 \times 10^{-6}$$

$$\Rightarrow \quad \epsilon_1 = 623.61 \times 10^{-6}$$

and $$\epsilon_2 = 176.39 \times 10^{-6}$$

Thus principal stresses will given as,

$$\sigma_1 = \frac{E}{1-\mu^2}(\epsilon_1 + \mu\,\epsilon_2)$$

$$= \frac{2 \times 10^5}{1-0.3^2}\left[623.61 \times 10^{-6} + 0.3 \times 176.39 \times 10^{-6}\right]$$

$$= 148.69 \text{ MPa}$$

$$\sigma_2 = \frac{E}{1-\mu^2}[\epsilon_2 + \mu\,\epsilon_1]$$

$$= \frac{2\times10^5}{1-0.3^2}\left[176.39\times10^{-6} + 0.3\times623.61\times10^{-6}\right]$$

$$= 79.88 \text{ MPa}$$

The direction of principal planes is given as,

$$\tan 2\theta_p = \frac{\phi_{xy}}{\epsilon_x - \epsilon_y} = \frac{200\times10^{-6}}{(600-200)\times10^{-6}}$$

$$\Rightarrow \quad \theta_{p_1} = 13.28° \quad \text{(Anticlockwise)}$$

$$\Rightarrow \quad \theta_{p_2} = 103.28° \quad \text{(Anticlockwise)}$$

Q.1 (d) Solution:

As the vertical rod is under the action of W, the extension δ is given by

$$\delta = \frac{WL}{AE} = \frac{WL}{\frac{\pi}{4}d^2E} = \frac{4WL}{\pi d^2 E}$$

$$\Rightarrow \quad E = \frac{4WL}{\pi d^2 \delta}$$

As a shaft is under the action of torque T, we have

$$\frac{T}{J} = \frac{G\theta}{L}$$

$$\Rightarrow \quad G = \frac{TL}{J\theta} = \frac{TL}{\frac{\pi}{32}d^4\theta} = \frac{32TL}{\pi d^4\theta}$$

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

$$\Rightarrow \quad \frac{4WL}{\pi d^2\delta} = \frac{2\times32TL(1+\mu)}{\pi d^4\theta}$$

$$\Rightarrow \quad 1+\mu = \frac{Wd^2\theta}{16T\delta}$$

$$\Rightarrow \quad \mu = \left(\frac{Wd^2\theta}{16T\delta} - 1\right)$$

Q.1 (e) Solution:

Self-compacting or self-consolidating concrete : Self-compacting concrete, also known as self-consolidating concrete (SCC) is characterized by high flowability that enables the concrete to spread through and around dense reinforcement under its own weight to adequately fill voids without segregation or excessive bleeding, and without the need for significant vibration. This emerging technology is made possible by the use of polycarboxylates plasticiser instead of older naphthalene based polymers, and viscosity modifiers to address aggregate segregation.

The flowability is achieved by using an advanced synthetic high-range water-reducing admixture (HRWR), while the viscosity of the paste is increased by using a viscosity-modifying admixture (VMA) or by increasing the percentage of fines incorporated into the SCC mix design.

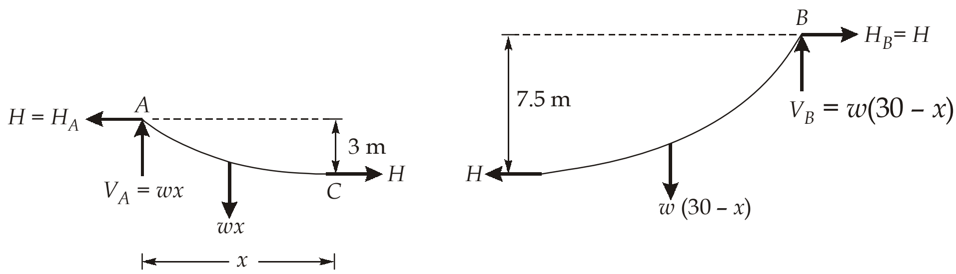
Advantages of Self-Compacting Concrete :

- SCC has very high flowability as measured by cone flow test rather than slump (height), typically the flow or spread is between 650-750 mm. This allows faster pouring rates in complicated formwork.
- There is no need for the vibrators to compact the concrete.
- There is no bleeding of water or aggregate segregation problem.
- There is significant reduction in manpower required at site.
- The post-stripping work gets reduced drastically.
- There is significant improvement in durability of concrete.
- Wear and tear on formwork is reduced.
- Good surface finish is produced.
- Thinner concrete sections can be cast.
- Noise levels reduce due to absence of vibration.
- It provides safer and cleaner working environment.

Q.2 (a) Solution:

Let the lowest point C be x meter from 'A'

Cut the cable at supports and at 'C' as shown.



Take moments at A,

$$H \times 3 = wx \times \frac{x}{2}$$

$$\Rightarrow \quad H = \frac{wx^2}{6} \qquad \text{...(i)}$$

Take moments about B,

$$H \times 7.5 = \frac{w(30-x)^2}{2}$$

$$H = \frac{w(30-x)^2}{15} \qquad \text{...(ii)}$$

Equating (i) and (ii), we get

$$\frac{wx^2}{6} = \frac{w(30-x)^2}{15}$$

$$\Rightarrow \quad 2.5x^2 = 900 - 60x + x^2$$

$$\Rightarrow \quad 1.5x^2 + 60x - 900 = 0$$

$$\Rightarrow \quad x^2 + 40x - 600 = 0$$

$$\therefore \quad x = 11.62 \text{ m}$$

$\therefore$ Minimum tension will be at lowest point

$$\therefore \quad T_{min} = H = \frac{6.6 \times 11.62^2}{6} = 148.5 \text{ kN}$$

Maximum tension will be at B

$$\therefore \quad T_{max} = \sqrt{H^2 + [w.(30-x)]^2}$$

$$\therefore \qquad T_{max} = \sqrt{148.5^2 + [6.6 \times 18.38]^2} = 191.7 \text{ kN}$$

$$\text{Tension at } A,\ T_A = \sqrt{148.5^2 + (6.6x)^2}$$

$$= \sqrt{148.5^2 + (6.6 \times 11.62)^2} = 167.1 \text{ kN}$$

Forces at pier A

$$\text{Vertical reaction, } V_{A,\text{ pier}} = 11.62 \times 6.6 + 167.1 \times \sin 30$$
$$= 160.45 \text{ kN}$$

$$\text{Horizontal reaction, } H_{A,\text{ pier}} = 148.5 - 167.1 \times \cos 30$$
$$= 3.9 \text{ kN}$$

Forces at pier B

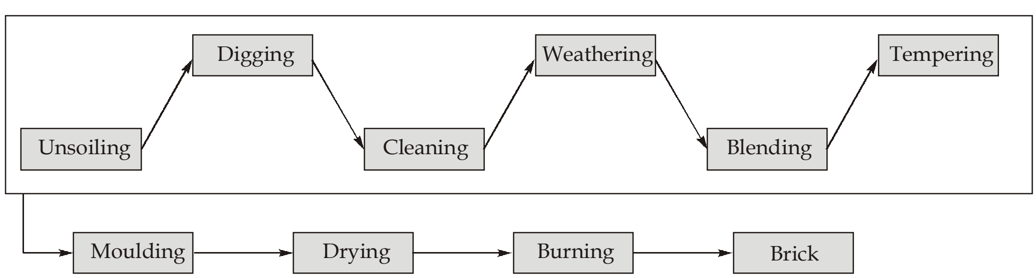
$$\text{Vertical reaction, } V_{B,\text{ pier}} = (30 - 11.62) \times 6.6 + 191.7 \times \sin 30$$
$$= 217.2 \text{ kN}$$

$$\text{Horizontal reaction, } H_{B,\text{ pier}} = 148.5 - 191.7 \cos 30$$
$$= -17.5 \text{ kN}$$

Q.2 (b) Solution:

Brick manufacturing involves four stages:

1. Brick clay/brick earth preparation.
2. Moulding of bricks.
3. Air drying of bricks.
4. Burning of bricks.



Flow chart for preparation of Brick

1. **Preparation of brick earth**

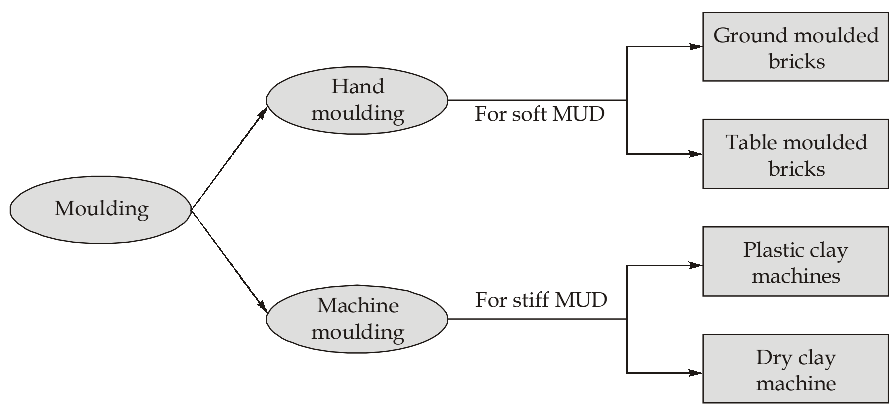
   Various steps involved in sequence are:

   a. Unsoiling
   b. Digging
   c. Cleaning

d. Weathering
e. Blending
f. Tempering

Unsoiling is removal of top soil of about 200 mm depth as it may contain impurities. **Digging** is taking out and spreading clay on a levelled ground. **Cleaning** is for removal of impurities such as stones, organic matter through washing and screening. **Weathering** is exposure to atmosphere for softening or mellowing, (exposure period ranges from few weeks to full season). **Blending** is mixing of added ingredients with the prepared loose clay. **Tempering** is adding water to clay and kneading it under the feet of men or cattle. It is done in pug mills through pugging.

2. Moulding: Moulding is filling the prepared clay into moulds of definite size and pattern.



Hand Moulding: Are of two types:

(a) **Ground moulded** is done where large levelled land is available. To avoid moulded bricks from sticking to the side of the mould, either sand is sprinkled on the inner sides of the mould or the mould may be dipped in water every time before moulding is done.

Sand moulded bricks are better because they have rough surface necessary for having good bond between bricks and mortar.

(b) **Table moulded** is done on tables and send for further process of drying. Here as time passes efficiency of the moulder decreases and cost of moulding increases.

Machine Moulding

(a) Plastic clay machine method: It is adopted for stiffer clay.

(b) Dry clay machine method (pressed bricks): The strong clay is first converted into powder form. Then after adding water a stiff plastic paste is prepared, which is used for moulding.

3. **Drying:** Moulded bricks are dried to remove moisture far controlling of shrinkage and saving of fuel. Green bricks have moisture content about 7-30%, which is dried upto 5 to 7%. Drying period ranges from 7-14 days.

There are two methods of drying.

(a) Natural drying

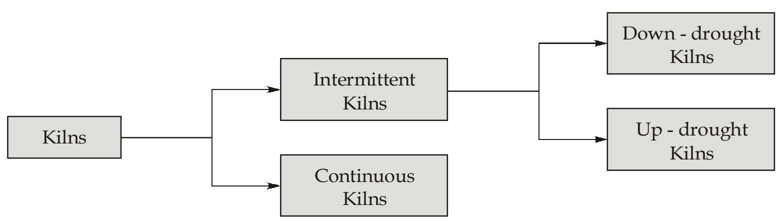
(b) Artificial drying

4. **Burning:** Proper uniform burning results into dense and durable brick. Bricks are burnt to a temperature of about 900 - 1200°C.

Burning is done in kilns or damps. Kiln is a permanent structure far large scale manufacturing and clamp is a temporary structure far small scale manufacturing.

Kiln is of two types:

1. **Intermittent Kilns:** Where only single process takes place at a time. Processes are loading, unloading, cooling and burning of moulded brick earth.
2. **Continuous Kiln:** Multiple processes takes place at a time.



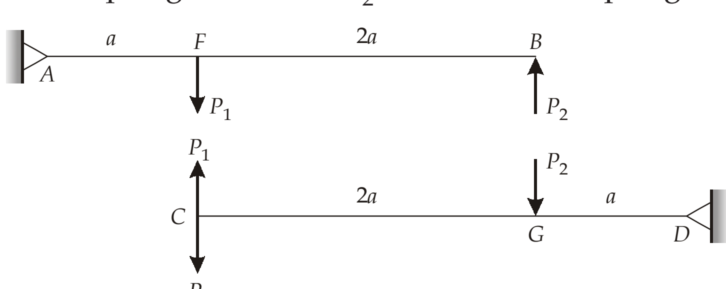
As far as efficiency is concerned down-drought kilns are better compared to up-drought kilns because of evenly burnt bricks are produced.

Types of continuous kilns are:

1. Bull's trench Kiln (popular in India)
2. Hoffman's Kiln, and
3. Tunnel Kiln.

Q.2 (c) Solution:

Let P_1 be the force in spring 1 at C and P_2 the be force in spring 2 at G.



Free body diagram

Taking moments about A,

$$P_1(a) - P_2(3a) = 0$$

$$\Rightarrow \quad P_1 = 3P_2 \quad \text{...(i)}$$

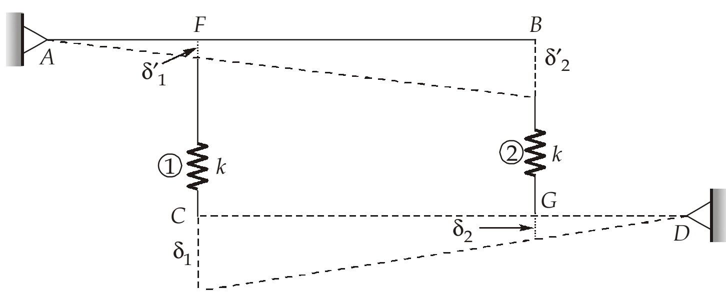
Taking moments about D,

$$(P_1 - P)\,3a = P_2 a$$

$$\Rightarrow \quad P_1 - P = \frac{P_2}{3} \quad \text{...(ii)}$$

From (i) and (ii),

$$P_1 = \frac{9}{8}P \text{ and } P_2 = \frac{3}{8}P$$



Deformation diagram

$$\text{Net deformation of spring 1} = \delta_1 - \delta_1' = \Delta_1 = \frac{P_1}{k}\text{(extension)}$$

$$\text{Net deformation of spring 2} = \delta_2' - \delta_2 = \Delta_2 = \frac{P_2}{k}\text{(compression)}$$

But, $$\frac{\delta_1}{\delta_2} = \frac{3a}{a}$$

$$\Rightarrow \quad \delta_1 = 3\delta_2$$

and, $$\frac{\delta_2'}{\delta_1'} = \frac{3a}{a}$$

$$\Rightarrow \quad \delta'_2 = 3\delta'_1$$

$$\therefore \quad \frac{P_1}{k} = \delta_1 - \delta_1' = \delta_1 - \frac{\delta_2'}{3}$$

$$\Rightarrow \quad \frac{3P_1}{k} = 3\delta_1 - \delta_2' \quad \text{...(iii)}$$

Also, $$\frac{P_2}{k} = \delta_2' - \delta_2 = \delta_2' - \frac{\delta_1}{3} \quad \text{...(iv)}$$

Solving (iii) and (iv), we get

$$\delta_1 = \frac{3}{8k}(3P_1 + P_2)$$

$$= \frac{3}{8k}\left(3 \times \frac{9}{8}P + \frac{3}{8}P\right) = \frac{45}{32}\frac{P}{k}$$

$\therefore$ Deflection at C = $\delta_1 = \frac{45}{32}\frac{P}{k}$

Q.3 (a) Solution:

(i) Factors affecting selection of flooring material:

Each type of flooring material has its own merits & demerits and the factors affecting selection of flooring material are as follows:

1. **Initial cost:** The cost of construction is very important in the selection of a type of flooring material. A floor consisting of marbles, granite, special clay tiles etc. is considered to be very expensive whereas a flooring of cork, slate, vinyl tile etc. is moderately expensive. The floors made of concrete and bricks offer the cheapest type of floor construction. It should be ensured during comparison of costs for different flooring materials that cost of covering the sub-floor has been accounted for.
2. **Appearance:** Flooring should produce the desired colour effect and architectural beauty in conformity with its use in the building. Generally, flooring of terrazzo, tiles, marble and cement mortar provide a good appearance.
3. **Cleanliness:** The flooring material should be non-absorbent and should be capable of being easily and effectively cleaned. All joints in flooring should be such as to offer a watertight surface. Moreover, greasy and oily substances should neither spoil the appearance nor have a destroying effect on flooring materials.
4. **Durability :** The flooring material should offer sufficient resistance to wear and tear, temperature, chemical action etc. so as to provide long life to the floors. From the durability point of view, flooring of marble, terrazzo, tiles and concrete is considered to be of the best type.
5. **Damp resistance :** All the floors, should offer sufficient resistance against dampness in buildings to ensure a healthy environment.
6. **Fire resistance :** Flooming material must be fire resistant so that fire cannot spread from one floor level to another.
7. **Maintenance :** The flooming material should require least maintenance.

(ii) 1. Drawing halls of residential building: PVC flooring is used in drawing halls of residential buildings. It is made of plastic material called polyvinyl chloride and is fabricated in the form of tiles of different sizes and different colour shades. PVC tile flooring is chemical resistant, smooth, good and can be easily cleaned.

2. **Dancing halls:** Timber flooring is used for dancing halls because it provides a non-slippery platform and is easy to repair. This flooring is not commonly used for residential building due to its cost.
3. **Warehouses:** Brick flooring is used in ware houses. Bricks are laid either flat or on edge, arranged in herring bone fashion or set at right angles to the walls or set at any other good looking pattern. The floors of warehouses are subjected to heavy load with significant wear and tear.

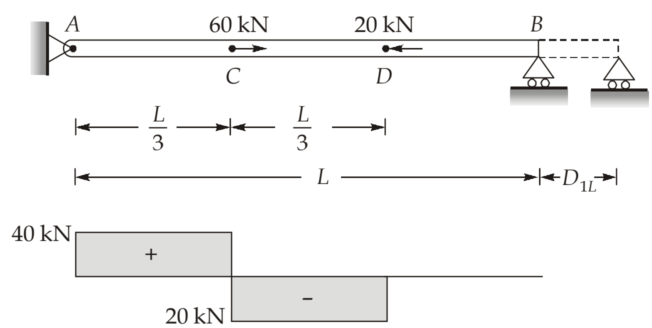
Q.3 (b) Solution:

(i)

The structure is statically indeterminate to first degree.

Let the horizontal reaction H_B ($\rightarrow$) at support B be treated as the redundant X.

Now remove the redundant force and thus the support B will act as a roller.



Axial force diagram

Fig (a) Effect of external loads

Now apply redundant force (X = 1 kN $\rightarrow$)

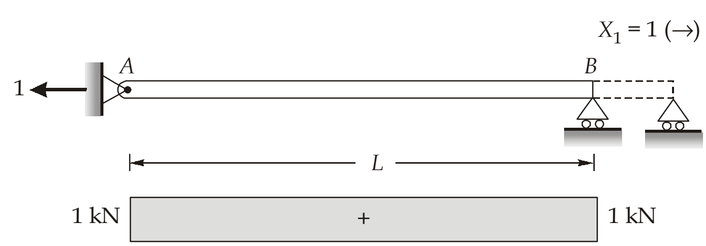


Fig (b) Effect of redundant force (X = 1 kN)

$\therefore$ From compatibility equation

$$D_{1L} + F_{11} X_1 = D_1$$

where, D_{1L} = Displacement due to external loads

$$= \frac{1}{AE}\left[40\times\frac{L}{3}+(-20)\times\frac{L}{3}\right]=\frac{20L}{3AE}$$

F_{11} = Displacement due to unit force in direction of redundant

$$= \frac{1}{AE}[(1\times L)]=\frac{L}{AE}$$

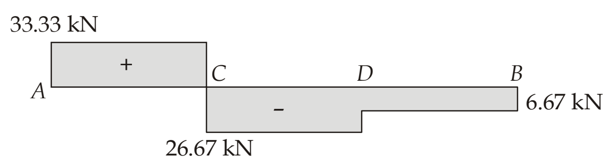
$$\therefore \quad \frac{20L}{3AE}+\frac{L}{AE}X_1 = 0$$

$$\Rightarrow \quad X_1 = \frac{-20}{3} = -6.67 \text{ kN}$$

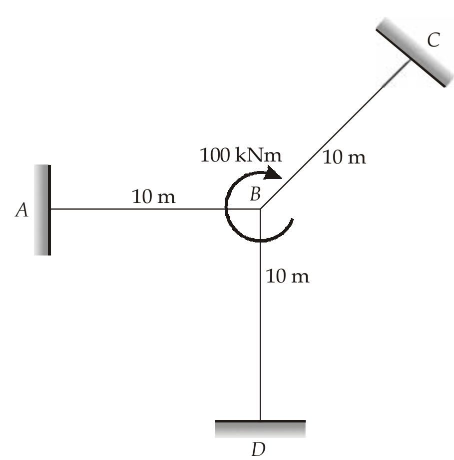
$\therefore$ $H_B = X_1 = -6.67$ kN i.e. 6.67 kN (←)

$\therefore$ $H_A = 60 - 20 - 6.67 = 33.33$ kN (←)

$\therefore$ Axial force diagrams can be drawn as



(ii)



The distribution factors are given below:

Joint	Member	Relative stiffness	Total relative stiffness	Distribution factors
B	BA	$I/10$		1/3 = 0.333
	BC	$I/10$	$3I/10$	1/3 = 0.333
	BD	$I/10$		1/3 = 0.333

Given, $M_B = M_{BA} + M_{BC} + M_{BD}$

As per moment distribution method,

$$\frac{M_{BA}}{(I/10)} = \frac{M_{BC}}{(I/10)} = \frac{M_{BD}}{(I/10)}$$

$\Rightarrow$ $$M_{BA} = M_{BC} = M_{BD}$$

$\therefore$ $$M_{BA} = M_{BC} = M_{BD} = \frac{M_B}{3} = \frac{100}{3} = 33.33 \text{ kNm}$$

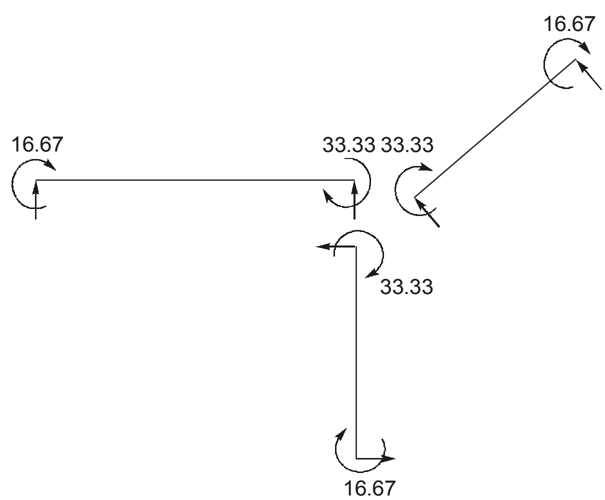
$\because$ A supports, A, C and D are fixed, $\frac{1}{2} \times M_{BA} = M_{AB}$ (Carryover factor)

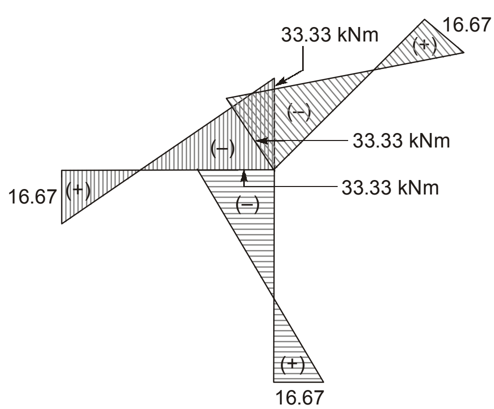
$\therefore$ $$M_{AB} = \frac{M_{BA}}{2} = \frac{33.33}{2} = +16.67 \text{ kNm}$$

$$M_{CB} = \frac{33.33}{2} = +16.67 \text{ kNm}$$

$$M_{DB} = \frac{M_{BD}}{2} = \frac{33.33}{2} = 16.67 \text{ kNm}$$

Hence **free body diagram** are:

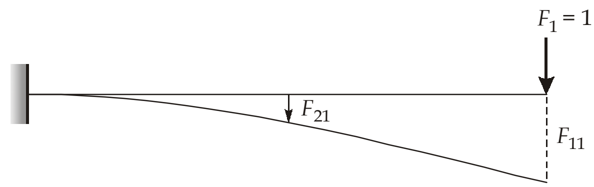




Bending moment diagram

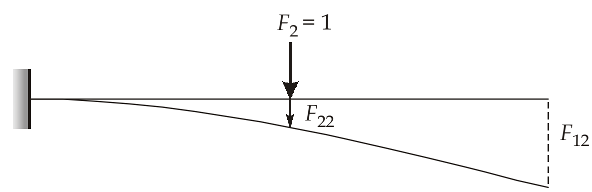
Q.3 (c) Solution:

It is easier to calculate the flexibility matrix and invert the same for the stiffness matrix.



$$F_{11} = \frac{F_1 L^3}{3EI} = \frac{L^3}{3EI}$$

$$F_{21} = \frac{5L^3}{48EI}$$



$$F_{22} = \frac{F_2\left(\frac{L}{2}\right)^3}{3EI} = \frac{L^3}{24EI}$$

$$F_{12} = \frac{F_2\left(\frac{L}{2}\right)^3}{3EI} + \frac{F_2\left(\frac{L}{2}\right)^2}{2EI} \times \frac{L}{2}c$$

$$= \frac{L^3}{24EI} + \frac{L^3}{16EI} = \frac{5L^3}{48EI}$$

$$\text{Flexibility matrix will be, } [F] = \frac{L^3}{48EI}\begin{bmatrix} 16 & 5 \\ 5 & 2 \end{bmatrix}$$

$$\therefore \quad \text{Stiffness matrix} = [F]^{-1}$$

$$= \frac{48EI}{7L^3}\begin{bmatrix} 2 & -5 \\ -5 & 16 \end{bmatrix}$$

The slope (θ) and deflection (δ) of the cantilever beam subjected to the point load F at the end are

$$\theta = \frac{FL^2}{2EI} \text{ and } \delta = \frac{FL^3}{3EI}$$

∴ The equation of motion are

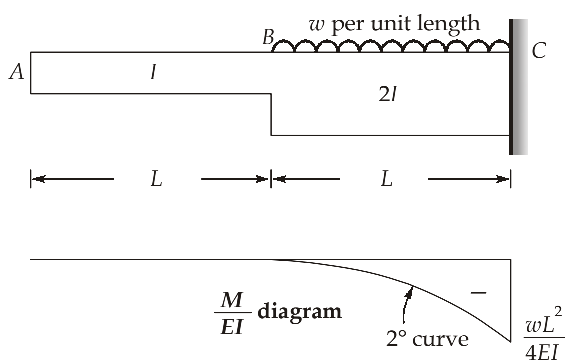
$$[M]\{\ddot{x}\}+[K]\{x\} = \{0\}$$

$$\Rightarrow \begin{bmatrix} \frac{ML}{4} & 0 \\ 0 & \frac{ML}{2} \end{bmatrix}\begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \frac{48EI}{7L^3}\begin{bmatrix} 2 & -5 \\ -5 & 16 \end{bmatrix}\begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix}$$

Q.4 (a) Solution:

Let the reaction at A be 'R'

(i) Effect of UDL,



Downward deflection at A due to UDL is given by,

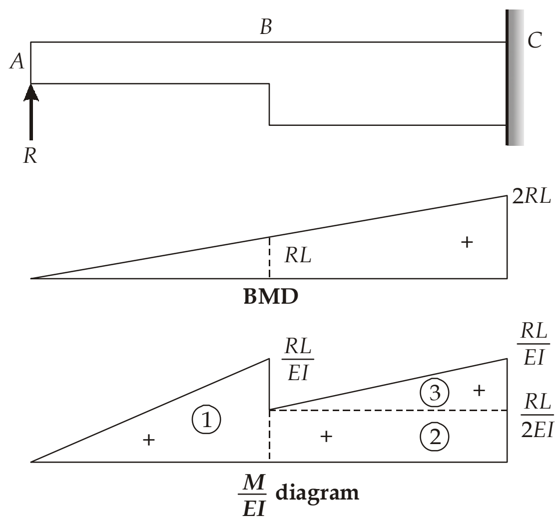
$(\delta_A)_1$ = Moment of area of $\frac{M}{EI}$ diagram between A and C about $A = A\bar{x}$,

$$A_1 = -\frac{1}{3}L\times\frac{wL^2}{4EI} = \frac{-wL^3}{12EI}$$

$$\bar{x}_1 = L+\frac{3}{4}L = \frac{7}{4}L \qquad \text{(From A)}$$

$$\therefore \quad (\delta_A)_1 = \frac{-wL^3}{12EI}\times\frac{7}{4}L = \frac{-7}{48}\frac{wL^4}{EI} \text{(Downward)}$$

(ii) Effect of propped reaction R



$$(\delta_A)_2 = \text{Moment of } \frac{M}{EI} \text{ diagram between A and C about A}$$

$$= A_1\bar{x}_1 + A_2\bar{x}_2 + A_3\bar{x}_3$$

$$= \left(\frac{1}{2}\times L\times\frac{RL}{EI}\right)\times\left(\frac{2L}{3}\right)+\left(\frac{RL}{2EI}\times L\right)\times\left(\frac{3L}{2}\right)+\left(\frac{1}{2}\times L\times\frac{RL}{2EI}\right)\times\frac{5L}{3}$$

$$= \frac{RL^3}{3EI}+\frac{3RL^3}{4EI}+\frac{5}{12}\frac{RL^3}{EI}$$

$$(\delta_A)_2 = \frac{3}{2}\frac{RL^3}{EI}$$

As, $(\delta_A)_1 = (\delta_A)_2$

$\Rightarrow$ $$\frac{7}{48}\frac{wL^4}{EI} = \frac{3}{2}\frac{RL^3}{EI}$$

$\Rightarrow$ $$R = \frac{7}{72}wL(\uparrow)$$

Q.4 (b) Solution:

Torsional moment diagram:

Portion CB,

Taking clockwise torsional moment as positive,

$$T_x \text{ (from C)} = +5 \text{ kNm} \qquad [0 \le x \le 3 \text{ m}]$$

$\therefore$ $T_C = +5$ kNm

and $T_B = +5$ kNm

$$\textbf{Portion BA,} \quad T_x \text{ (from B)} = +5 - 10 + 2x$$
$$= -5 + 2x$$

At $x = 0$, T_B(just right of B) $= -5$ kNm

At $x = 5$, $T_A = -5 + 2 \times 5 = +5$ kNm

Torsional moment diagram changes sign, at a point where,

$$T_x = -5 + 2x = 0$$

$\Rightarrow$ $x = 2.5$ m from B

Angle of twist diagram

Portion AB,

$$\theta = \frac{TL}{GJ}$$

where GJ = Torsion rigidity (which is constant throughout the rod)

Consider a section of thickness dx at a distance x from A,

$$T_x \text{ (from } A) = 5 - 2x$$

$$\therefore \quad \theta_{Ax} = \int_0^x \frac{T_x dx}{GJ} = \frac{1}{GJ}\int_0^x (5-2x)\,dx = \frac{1}{GJ}\left[5x - x^2\right]$$

$$\Rightarrow \quad GJ\theta_{Ax} = 5x - x^2$$

Since, $\theta_{Ax} = \theta_x - \theta_A$

$\Rightarrow$ $\theta_x = \theta_{Ax} + \theta_A$ [As $\theta_A = 0$]

$$\Rightarrow \quad \theta_x = \frac{1}{GJ}\left(-x^2 + 5x\right)$$

At $x = 0$ m, $\theta_A = 0$

At $x = 5$ m, $\theta_B = 0$

Maximum value of θ_x will be at $x = 2.5$ m

$$\therefore \quad \theta_{max} = \frac{1}{GJ}\left(-2.5^2 + 5 \times 2.5\right) = \frac{6.25}{GJ}$$

Portion BC

Consider a section at a distance x from B,

$$T_x \text{ (} x \text{ distance from } B) = 5 \text{ kNm}$$

$$\therefore \qquad \theta_{Bx} = \int_0^x \frac{T_x dx}{GJ} \qquad [0 \le x \le 3]$$

$$= \int_0^x \frac{5dx}{GJ} = \frac{5x}{GJ}$$

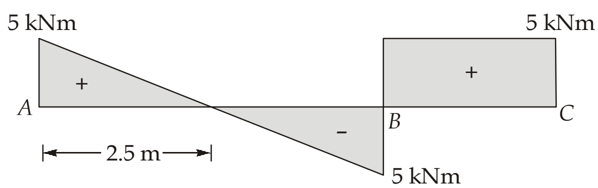
$$\theta_{Bx} = \theta_B - \theta_x = \frac{5x}{GJ}$$

$$\Rightarrow \qquad \theta_x = \theta_B + \frac{5x}{GJ}$$

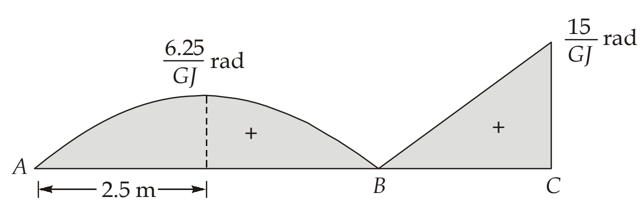
$$\Rightarrow \qquad \theta_x = \frac{5x}{GJ} \qquad [\text{As } \theta_B = 0]$$

At $x = 0$, $\qquad \theta_B = 0$

At $x = 3$ m, $\qquad \theta_C = \dfrac{15}{GJ}$



Torsional moment diagram



Twist angle variation diagram

Q.4 (c) Solution:

(i)

Varnish is a transparent, hard protective covering or film primarily used in wood finishing and also on other materials. It is a solution or resin or resinous substance prepared either in alcohol or turpentine. Varnish finishes are usually glossy but may be designed to produce satin or semi-gloss sheens by the addition of flatting agents. Varnish is transparent, has little or no colour and has no added pigment, as opposed to paints or wood stains, which contain pigment and generally range from opaque to translucent.

Varnish contains the following ingredients:

- **Resin :** Resins that are used in varnishes include amber, kauri gum, dammar, copal, rosin (pine resin), sandarac, balsam, elemi and others.

- **Drying oil or drier :** There are different types of drying oils including linseed oil, tung oil and walnut oil which are added for accelerating the drying process. They contain high levels of polysaturated fatty acids.
- **Thinner or solvent :** Traditionally, natural (organic) turpentine was used as the thinner or solvent, but has been replaced by several mineral based turpentine substitutes such as white spirit or paint thinner, also known as mineral spirit. The type of solvet of solvet to be used depends on the nature and type of resin.

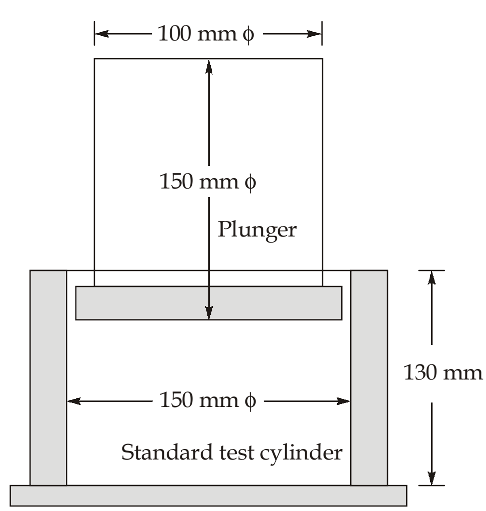
Classification of Varshishes:

Depending on the solvent used, varnishes are classified into four categories:

- **Oil varnishes :** Oil varnish dries slowly but forms hard and durable surface. Examples - Copal in linseed oil.
- **Spirit varnishes :** It dries quickly. Examples - Shellacs, etc., in methylated spirit.
- **Turpentine varnish :** It dries quickly and is light coloured. Examples - Rosia, gum, dammar, mastic in turpentine.
- **Water varnish :** It is used for delicate internal work. Examples - Shellac in hot water (with borax or potash soda added to dissolve the shellac).

(ii)

1. **Ten percent fines value test :** In this test, about 6.5 kg material passing 12.5 mm and retained on 10 mm sieves is filled in a standard test cylinder with base plate as shown in figure below in three layers, each layer compacted with 25 strokes of an iron tamping rod. The top layer is leveled off and the weight of the sample is recorded.



Apparatus for ten per cent fines value for coarse aggregate

The plunger is placed on the cylinder and the unit is set up in a compression testing machine. The load is gradually applied at a uniform rate so that plunger penetrates a specified distance in 10 minutes. This penetration is 15 mm for rounded aggregates such as natural gravel, 20 mm for normal crushed stone and 24 mm for honey combed aggregates like shale and slag, etc.

After specified penetration is reached, the load is released and material is sieved through 2.36 mm sieve. The weight of fines passing the sieve is expressed as a percentage of the weight of test sample. This should be in the range of 7.5 to 12.6 per cent, i.e., about 10 percent. The test is repeated with the same weight of test sample as taken in the preceding test till the load for the fines in above range is attained.

The load for 10 percent fines is calculated as follows:

$$\text{Load for 10 percent fines} = \left(\frac{14}{y+4}\right)x$$

where, x = load in tonnes for causing 7.5 to 12.6 percent fines, and y = mean of the percentage of fines from two test at x tonnes load.

The value is reported to nearest 0.5 tonnes. The values generally recommended are :

- For normal concrete, not less than 5 tonnes.
- For wearing surface, not less than 10 tonnes.
- For monolithic concrete in buildings, not less than 15 tonnes.

2. **Aggregate impact value test :** This test is for the aggregates to be used in the concrete which is to undergo impact such as runways in airports. In this test, material passing 12.5 mm and retained on 10 mm sieves is filled in a standard cylinder in three layers, each layer tampered with 25 strokes of an iron rod. A hammer weighing 14 kg is dropped from a height of 380 mm for 15 times and the resulting material is sieved through 2.36 mm IS sieve. The percentage fine of the material is the aggregate impact test value. It should not be more than 45 for aggregate used for ordinary concrete and not more than 30 for aggregates used for concrete in runways and pavements. This value for Indian aggregates generally ranges from 15 to 30 percent.
3. **Aggregate crushing value test :** In this test the prepared specimen set in compression testing machine is loaded to 40 tonnes in 10 minutes through the plunger. The load is released and the materials is sieved through 2.36 mm sieve to obtain the aggregate crushing value or percentage of fines. It is usually limited to 45 for aggregates to be used for concrete other than that for wearing surfaces and 30 for the aggregate for wearing surface concrete. The value generally varies from 18 to 27 per cent for Indian aggregates.

Section B

Q.5 (a) Solution:

For a fixed ended beam and loaded partly for its span as shown in figure below, the plastic hinges will be at A and C, and at a point D at a distance x from support C.

From the mechanism,

$$\Delta = (L-x)\theta = x\theta_1$$

$$\Rightarrow \quad \theta_1 = \frac{L-x}{x}\theta$$

External work done = Intensity of load × Area of collapse mechanism diagram under the load

$$= \frac{W_u}{L/2} \times \left[\left(\frac{x\theta_1 + \frac{L\theta}{2}}{2}\right)\left(\frac{L}{2} - x\right) + \left(\frac{x\theta_1 + 0}{2}\right)x\right]$$

$$= \frac{2W_u}{L} \times \left[\frac{1}{2}\left(x\frac{L-x}{x} + \frac{L}{2}\right)\left(\frac{L}{2} - x\right)\theta + \frac{x^2}{2} \times \frac{L-x}{x}\theta\right]$$

$$= \frac{W_u}{L} \times \left[\left(L - x + \frac{L}{2}\right)\left(\frac{L}{2} - x\right) + x(L-x)\right]\theta$$

$$= \frac{W_u}{L} \times \left[\frac{3}{4}L^2 - \frac{3}{2}Lx - \frac{Lx}{2} + x^2 + Lx - x^2\right]\theta$$

$$= \frac{W_u}{L} \times \left[\frac{3}{4}L^2 - Lx\right]\theta = W_u\left(\frac{3}{4}L - x\right)\theta$$

$$= W_u\left(\frac{3}{4}L - x\right)\theta$$

Internal work done $= M_P\theta + M_P(\theta + \theta_1) + M_P\theta_1$

$$= 2M_P(\theta + \theta_1)$$

$$= 2M_P\left(\theta + \frac{L-x}{x}\theta\right)$$

$$= 2M_P\left(1 + \frac{L-x}{x}\right)\theta = 2M_P\left(\frac{L}{x}\right)\theta$$

By the principle of virtual work

External work done = Internal work done

$$W_u\left(\frac{3}{4}L - x\right)\theta = 2M_P\left(\frac{L}{x}\right)\theta$$

$$\Rightarrow \quad M_p = \frac{W_u}{2L}\left(\frac{3}{4}Lx - x^2\right)$$

For the maximum value of M_p

$$\frac{dM_P}{dx} = 0 = \frac{W_u}{2L}\left(\frac{3}{4}L - 2x\right)$$

$$\Rightarrow \quad x = \frac{3}{8}L = 0.375L$$

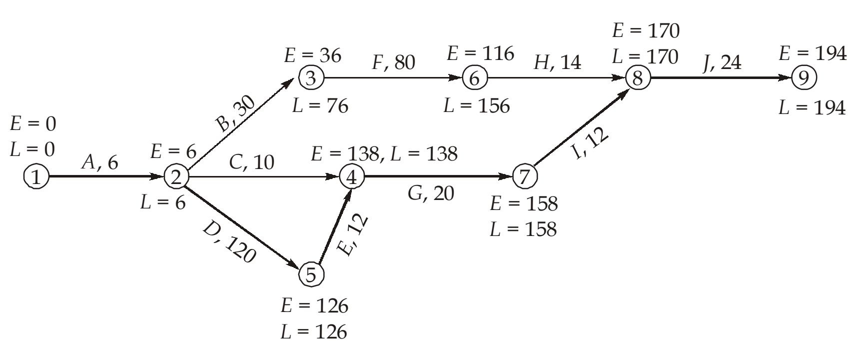
Hence,
$$M_p = \frac{W_u}{2L}\left[\frac{3}{4}L \times 0.375L - 2\times(0.375L)^2\right]$$

$$= \frac{0.07031W_u}{L}$$

$$W_u = 14.22\frac{M_P}{L}$$

Q.5 (b) Solution:

Activity	Duration (Days)	Preceding activities
A	6	NONE
B	30	A
C	10	A
D	120	A
E	12	D
F	80	B
G	20	C, E
H	14	F
I	12	G
J	24	H, I



Activity	t_E	EST	EFT	LST	LFT	Total Float
A, 1–2	6	0	6	0	6	0
B, 2–3	30	6	36	46	76	40
C, 2–4	10	6	16	128	138	122
D, 2–5	120	6	126	6	126	0
E, 5–4	12	126	138	126	138	0
F, 3–6	80	36	116	76	156	40
G, 4–7	20	138	158	138	158	0
H, 6–8	14	116	130	156	170	40
I, 7–8	12	158	170	158	170	0
J, 8–9	24	170	194	170	194	0

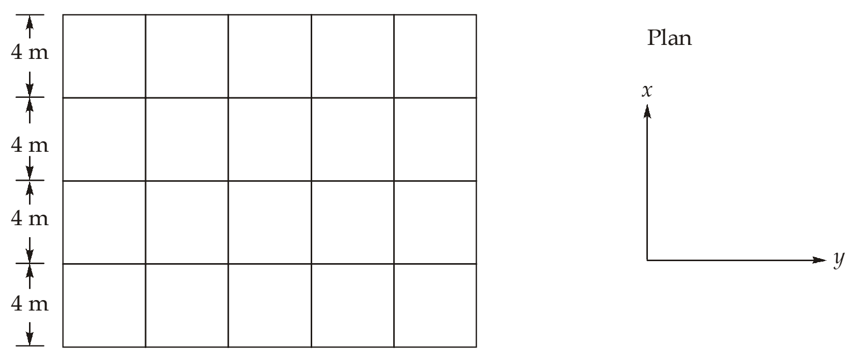
For critical activities, total float = 0

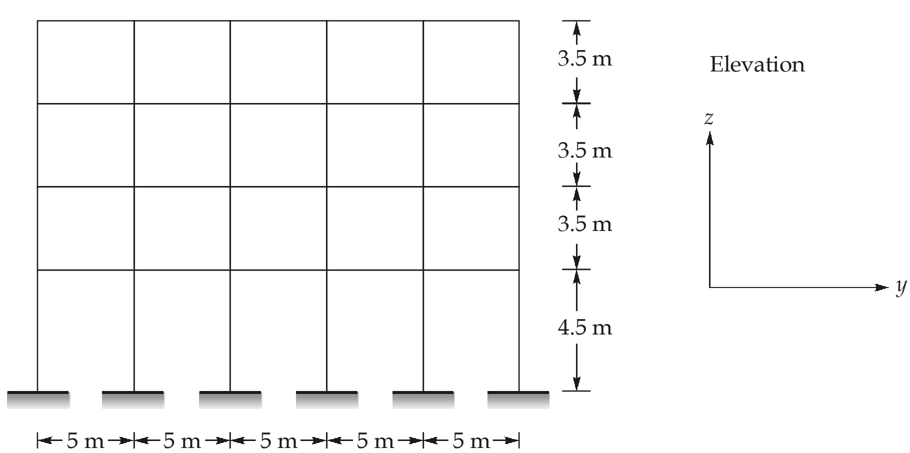
Therefore critical path is :

1 – 2 – 5 – 4 – 7 – 8 – 9 i.e. $A \rightarrow D \rightarrow E \rightarrow G \rightarrow I \rightarrow J$

Total duration = 194 days

Q.5 (c) Solution:





Dead load on floor = 12 kN/m^2

Dead load on roof = $6\ kN/m^2$

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

Live load on roof = $1.5\ kN/m^2$

Floor area = $16 \times 25 = 400\ m^2$

Since the live load is $3\ kN/m^2$, only 25% of the live load is taken for seismic weight of floors.

Seismic weight of the floors = $W_2 = W_3 = W_4 = (12 + 0.25 \times 3) \times 400 = 5100$ kN

At roof no live load is taken for seismic weight calculation

∴ Seismic weight of roof,

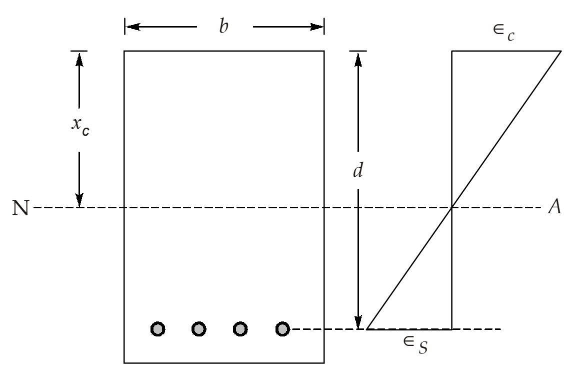
$$W_1 = 6 \times 400 = 2400 \text{ kN}$$

∴ Total seismic weight of the structure, $W = \sum Wi$

$$= 3 \times 5100 + 2400$$

$$= 17700 \text{ kN}$$

Q.5 (d) Solution:



Under-reinforced section : In case of under-reinforced section, the extreme strain in concrete $\in_c < 0.0035$ but tension steel yields i.e. strain in tension steel reaches $\in_y$ where

$$\in_s = \in_y = \frac{f_y}{1.15 \in_s} + 0.002$$

A RC section becomes under-reinforced when,

$$x_u < x_{ulim} \quad \text{i.e when}$$

$$A_{st} < A_{st\ lim} \quad \text{where}$$

$$A_{st\ lim} = \frac{p_{t\lim}}{100}(bd)$$

$$p_{t\,\lim} = 41.61\left(\frac{f_{ck}}{f_y}\right)\left(\frac{x_{u\,\lim}}{d}\right)$$

Failure in this type of section is ductile as steel yield before the concrete fails in compression.

Over-reinforced section : In case of over-reinforced section, strain in tension steel $\in_s \left(< \frac{f_y}{1.15 \in_s} 0.002\right)$ but strain in concrete in extreme compression fibre reaches $\in_s$ (=0.0035) i.e.

$$\in_c = 0.0035$$

and

$$\in_s < \frac{f_y}{1.15 \in_s} 0.002$$

Failure in this type of section is brittle due to sudden compression failure of concrete.

A section becomes over-reinforced when

$$x_u > x_{u\lim} \text{ i.e. when}$$

$$A_{st} > A_{st\lim}$$

Limit State: For a structure, limit state is a state of impending failure, beyond which the structure cannot perform its intended function satisfactorily.

Beyond limit state, the structure cannot be used either due to collapse of structure or its unserviceability.

There are two types of limit state:

1. **Ultimate limit state (limit state of collapse):**

   It is due to failure of structure in any of the following modes

   (a) flexure (b) compression

   (c) torsion (d) shear

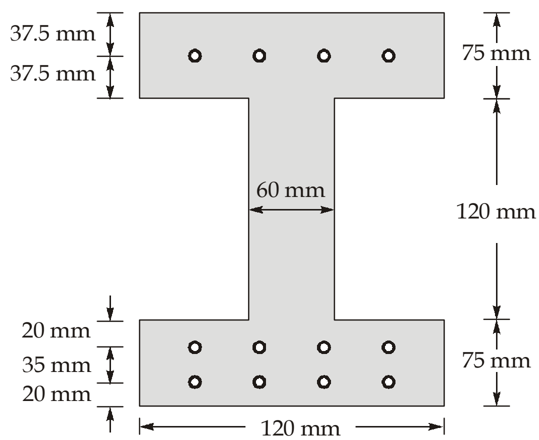
2. **Serviceability Limit State:**

In this structure becomes unserviceable due to attainment of the limit state in any of the following:

(a) excessive deflection (b) excessive cracking

(c) vibration (d) corrosion

Q.5 (e) Solution:



Area of steel wires in top flange = $4 \times \frac{\pi}{4} \times 3^2 = 28.274 \text{ mm}^2$

Area of steel wires in bottom flange = $2 \times 28.274 = 56.548 \text{ mm}^2$

∴ Total area of steel wires = $28.274 + 56.548 = 84.822 \text{ mm}^2$

∴ Prestressing force, P = $84.822 \times 1000 = 84822$ N

Area of beam section, A = $2(120 \times 75) + 60 \times 120 = 25200 \text{ mm}^2$

Moment of inertia of beam section,

$$I = \frac{120 \times 270^3}{12} - \frac{60 \times 120^3}{12} = 1.8819 \times 10^8 \text{ mm}^4$$

Direct compressive stress, $\sigma = \frac{P}{A} = \frac{84822}{25200} = 3.37 \text{ N/mm}^2$

Net/equivalent eccentricity about the CG of beam cross-section,

$$e = \frac{28.274 \times (60 + 37.5) - 56.548 \times (60 + 20 + 17.5)}{28.274 + 56.548}$$

e = –32.5 mm (–ve sign means below (CG))

So, extreme stress due to eccentricity of P-force

$$= \frac{P \times e}{I} \times y = \frac{84822 \times 32.5}{1.8819 \times 10^8} \times (60 + 75) = 1.98 \text{ N/mm}^2$$

Let the maximum sagging moment be M

(i) Limiting the extreme compressive stress at to top at 14 N/mm².

$$3.37 - 1.98 + \frac{M}{I} \times y = 14$$

$$\Rightarrow \quad M = 12.61 \times \frac{1.8819 \times 10^8}{135} \text{ N.mm}$$

$$M = 17.58 \text{ kNm}$$

(ii) Limiting the extreme tensile stress at bottom at 1.40 N/mm^2

$$3.37 + 1.98 - \frac{M}{I} \times y = -1.4$$

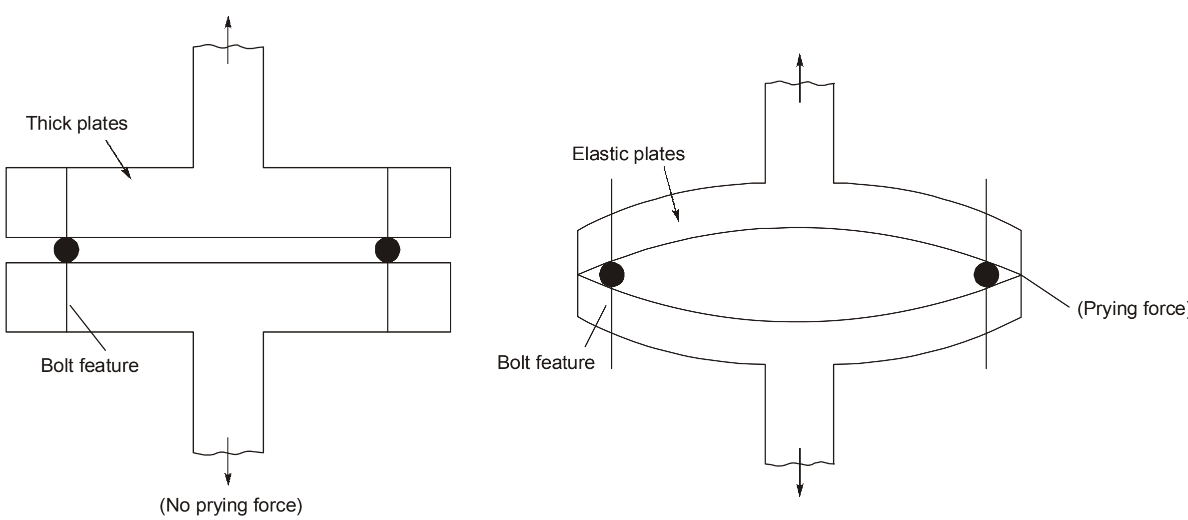
$$\Rightarrow \quad M = 6.75 \times \frac{1.8819 \times 10^8}{135} \text{ Nmm}$$

$$\Rightarrow \quad M = 9.41 \text{ kNm}$$

So maximum sagging moment that can be applied = 9.41 kNm.

Q.6 (a) Solution:

(i) Prying forces: Moment resisting beam to column connections often contain regions in which the bolts will be required to transfer load by direct tension, such as the upper bolts in the end plate connection. In the design of such connections, we should consider an additional force induced in the bolts as a result of so called 'prying action'. These additional prying forces induced in the bolts are mainly due to the flexibility of connected plates. Thus in a simple T-stub connection as shown in figure, the prying force will develop only when the ends of the flanges are in contact due to the external load as shown in figure.



(ii)

Bolts: A bolt is a metal pin with head formed at one end and the shank threaded at the other in order to receive a nut.

Advantages of bolted connection:

1. No noise creation in preparation of bolted connection unlike in case of revetting (because hammering is done in revetting).
2. The bolted connection is be done quickly in comparison to the riveting.
3. Bolted connection is economical to use as less labour is required for installation and work proceeds quickly.
4. The bolted connections facilitates the erection because of ease with which these connections can be done.
5. There is no risk of fire in bolted connection (whereas in riveting there is a risk of fire as rivets are made red hot for making the connection.

Disadvantages of bolted connection:

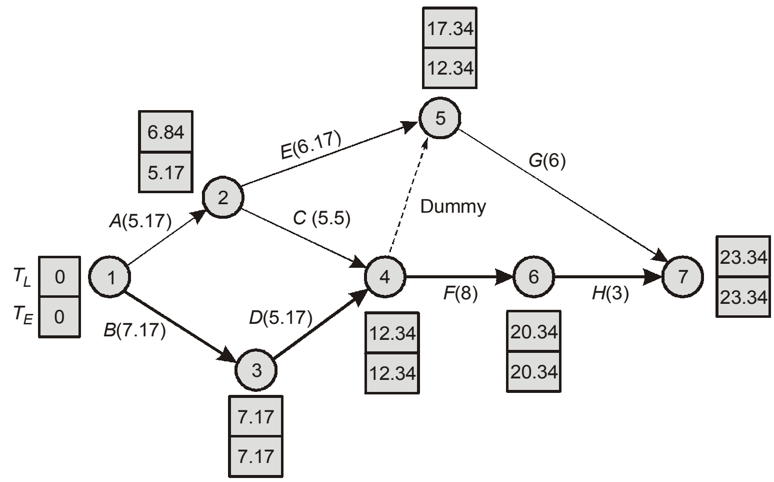
1. If bolted connection becomes loose, the strength reduces considerably.
2. The bolted connections have less strength when they are subjected to axial tension, because area at root of thread is less as compared to shank area.
3. Generally, the diameter of holes is kept 2 mm more than nominal diameter of bolts. The bolt does not fill the hole and there remains a clearance in bolted connections.

Q.6 (b) Solution:

Activity	Expected time (t_e)	Variance (σ^2)
A	5.17	0.69
B	7.17	0.25
C	5.5	0.69
D	5.17	0.69
E	6.17	0.69
F	8	1
G	6	1
H	3	1.78

As $$t_e = \frac{t_0 + t_p + 4t_m}{6} \text{ and } \sigma^2 = \frac{(t_P - t_o)^2}{36}$$

(i)



(ii) Expected duration and variance have been computed in the above table.

(iii) Critical path: $1 \rightarrow 3 \rightarrow 4 \rightarrow 6 \rightarrow 7$

i.e. B, D, F and H are critical activities.

Expected project completion time = 23.34 weeks

(iv)
$$t_{cp} = 23.34 \text{ weeks}$$
$$\sigma^2_{cp} = \sigma^2_B + \sigma^2_D + \sigma^2_F + \sigma^2_H = 3.72$$
$$\therefore \quad \sigma_{cp} = 1.93$$
$$\therefore \quad z = \frac{T - t_{cp}}{\sigma_{cp}} = \frac{25 - 23.34}{1.93} = 0.86$$

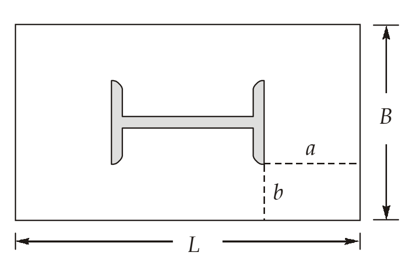
$\therefore$ By Interpolation, $P(Z = 0.86) = 80.21\%$

Q.6 (c) Solution:

$$\text{Load on column, } P = 1500 \text{ kN}$$
$$\therefore \quad \text{Factored load, } P_u = 2250 \text{ kN}$$
$$\text{Bearing strength of concrete} = 0.45 f_{ck} = 11.25 \text{ MPa}$$
$$\therefore \quad \text{Area of slab base required} = \frac{2250 \times 1000}{11.25} = 20 \times 10^4 \text{mm}^2$$

For minimum thickness of slab base, overhang in both directions should be equal i.e.

$$a = b$$



Now, $L \times B = 20 \times 10^4$

$\Rightarrow (400 + 2a)(250 + 2b) = 20 \times 10^4$

$\Rightarrow (400 + 2a)(250 + 2a) = 20 \times 10^4$

$\Rightarrow 10 \times 10^4 + 800a + 500a + 4a^2 = 20 \times 10^4$

$\Rightarrow 4a^2 + 1300a - 10 \times 10^4 = 0$

$a = 64.23$ mm

So, take $a \simeq 65 \text{ mm} = b$

So area of slab base $= (400 + 2 \times 65)(250 + 2 \times 65)$

$= (530 \times 380)\ \text{mm}^2$

$= 201400\ \text{mm}^2$

Now, bearing stress on concrete

$$w_u = \frac{1500 \times 1.5 \times 1000}{530 \times 380} = 11.17 \text{ MPa} < 11.25 \text{ MPa} \quad \text{(OK)}$$

So, thickness of slab base required,

$$t \geq \sqrt{\frac{2.5 w_u \left(a^2 - 0.3b^2\right)}{f_y / 1.1}}$$

$$\Rightarrow \quad t \geq \sqrt{\frac{2.5 \times 11.17 \left(65^2 - 0.3 \times 65^2\right)}{250 / 1.1}}$$

$\Rightarrow t \geq 19.06$ mm

So, provide $t = 20$ mm

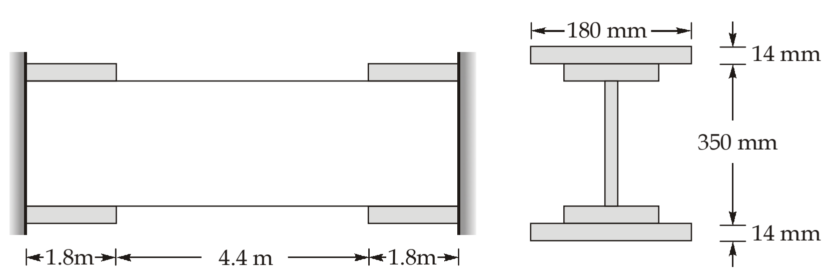
$\therefore$ Provide a slab base of size 530 mm × 380 mm × 20 mm

Q.7 (a) Solution:

For ISMB 350

Section modulus, $Z_{xx} = 778.9 \times 10^{-6}\ \text{m}^3$

Take shape factor for I-section $= 1.12$



As cover plates are welded at top and bottom of ISMB at the ends, the plastic moment carrying capacity of the end part will be M_{P1}.

If M_p is the plastic moment carrying capacity of central part, then

$$M_P = f_y \times Z_p$$
$$= 250 \times 1.12 \times 778.9 \times 10^{-6} \times 10^6$$
$$= 218092 \text{ Nm} = 218.092 \text{ kNm}$$

($\because$ Z_p = shape factor × Z_{xx})

For end zone

$$Z_{P_1} = Z_p \text{ of plate} + Z_p \text{ of } I\text{-section}$$

$$M_p \text{ of one plate} = f_y\, Z_{P,plate} = f_y \cdot A\bar{y} = f_y\left(180\times 14\times 182\right)$$

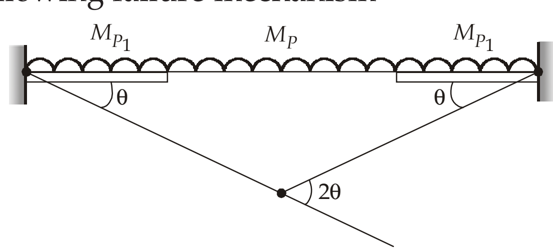
$$Z_p \text{ of plate} = \frac{A}{2}\left(\bar{y}_1+\bar{y}_2\right)$$

$$= \frac{2 \times 180 \times 14}{2}\left(182+182\right) = 917280 \text{ mm}^3$$

So, $$Z_{P_1} = 1.12 \times 778.9 \times 10^3 + 917280 = 1789648 \text{ mm}^3$$

$\therefore$ $$M_{P_1} = f_y Z_{P_1} = 250 \times 1789648 \text{ Nmm} = 447.412 \text{ kNm}$$

Now consider the following failure mechanism



$\because$ Degree of static indeterminacy = 2

$\therefore$ Number of plastic hinges required for collapse = 2 + 1 = 3

$\therefore$ $$M_{P_1}\theta + M_P\left(2\theta\right) + M_{P_1}\theta = \frac{1}{2}\times W_u \times L \times \frac{L}{2}\theta$$

$$2M_{P_1} + 2M_P = \frac{1}{4}W_u L^2$$

$\Rightarrow$ $$2 \times 447.412 + 2 \times 218.092 = \frac{1}{4}\times W_u \times 8^2$$

$\Rightarrow$ $$W_u = 83.188 \text{ kN/m}$$

Q.7 (b) Solution:

$$b_w = 150 \text{ mm}, \quad f_{ck} = 40 \text{ MPa}$$
$$D = 300 \text{ mm}, \quad f_{cp} = 5 \text{ N/mm}^2$$
$$d = 300 - 50 = 250 \text{ mm}, \quad f_y = 415 \text{ N/mm}^2$$
$$V = 130 \text{ kN}$$
$$f_t = 0.24\sqrt{f_{ck}} = 0.24\sqrt{40} = 1.518 \text{ N/mm}^2$$

Now, ultimate shear strength of section

$$V_{cw} = 0.67 b_w D\sqrt{f_t^2 + 0.8 f_{cp} f_t}$$
$$= 0.67 \times 150 \times 300\sqrt{1.518^2 + 0.8 \times 5 \times 1.518}$$
$$\simeq 87260 \text{ N} = 87.26 \text{ kN}$$

Hence balance of shear force to be resisted by stirrups = $V - V_{cw}$

$$= 130 - 87.26 = 42.74 \text{ kN}$$

Using 8 mm diameter two legged stirrups, the spacing of stirrups is

$$S_v = \frac{0.87 f_y A_{sv} d}{V - V_{cw}}$$
$$= \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times 250}{42.74 \times 10^3} = 212.31 \text{ mm}$$

Maximum permissible spacing of vertical stirrups = $0.75d$

$$= 0.75 \times 250$$
$$= 187.5 \text{ mm} < 212.31 \text{ mm}$$

So, provide 8 mm diameter two legged stirrups at 180 mm c/c.

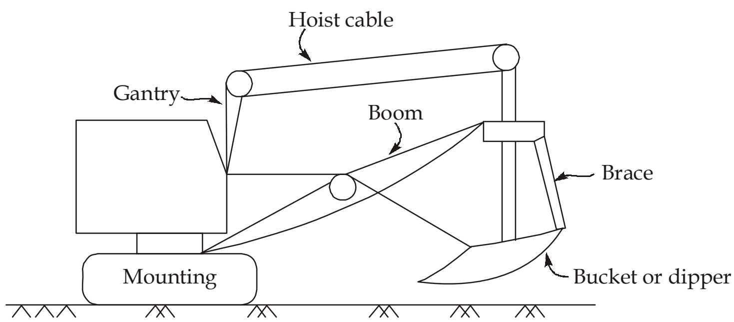
Q.7 (c) Solution:

(i)

Hoe is an excavating equipment of the power shovel group and is used primarily to excavate below the natural surface of the ground on which the machine rests. A hoe is sometimes referred to by other names such as back hoe or back shovel.

Basic parts of a hoe:

The basic parts of a hoe are illustrated in the figure shown.



Operation of a hoe :

The machine is placed in operation by setting the boom at the desired angle. The dipper is moved to the desired position. The free end of the boom is lowered down by releasing the tension in the hoist cable until the dipper teeth engages the material to be dug. As the cable is pulled in, the dipper gets filled up. Then the boom is raised and swing to the dumping position.

Applications of hoe:

1. It can be used to excavate below the natural surface of the ground on which the machine rests.
2. It can dig trenches, excavations for footings or basements and general grading work which requires precise control of depths.
3. It can operate on close range work and dump into trucks.
4. It can penetrate easily into toughest materials to be dug. However, it is not the production excavation machine. It is designed for mobility and general purpose work.

(ii)

1. **Muster roll :** The categories of skilled and unskilled workers employed on daily basis whose daily attendance and out turn are recorded in a form for the purpose of payment is called muster roll.
2. **Liquidated damage :** It is a fixed stipulated sum payable by the contractor on account of penalty for delays and does not bear any relationship with the real damage to the owner. It is generally high and fixed on a per day basis for the excess period over the specified period in the tender.
3. **Earnest money :** Earnest money is an assurance or guarantee on the part of the contractor to keep open the offer for consideration and to confirm his intention to take up the work if accepted in his favour for execution as per terms and conditions in the tender.

While submitting a tender, the contractor is to deposit a certain amount usually 1% to 2% of the total estimated cost of the work, with the department, as earnest money.

4. **Security money :** The contractor whose tender has been accepted is required to deposit 10% of the tendered amount as security money with the department inclusive of the earnest money already deposited. This amount is kept as a check so that the contractor fulfils all the terms and conditions and carries out the work satisfactorily within the prescribed time limit. If the contractor fails on this part, his whole or part of the security money is forfeited by the department as a punishment. The security money also serves as a security against the materials or the tools and machineries provided by the department to the contractor on loan.

Q.8 (a) Solution:

(i) Size of foundation

Load on column = 1500 kN

Dead load of column and footing = 0.15 × 1500 = 225 kN

∴ Total load = 1500 + 225 = 1725 kN

$$\text{Area of footing} = \frac{1725}{140} = 12.32 \text{ m}^2$$

$$\text{So, size of foundation, } B = \sqrt{12.32}$$

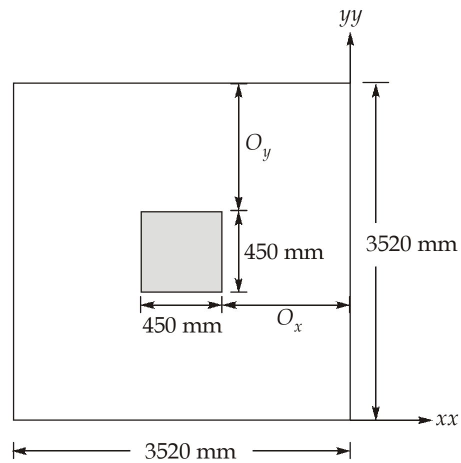
$$\Rightarrow \quad B = 3.51 \simeq 3.52 \text{ m}$$

So provide a square footing of size 3.52 m

$$\therefore \quad \text{Area of footing provided} = 3.52^2 = 12.3904 \text{ m}^2$$

$$\text{Design upward soil pressure} = \frac{1.5 \times 1500}{12.3904} = 181.5922 \simeq 181.60 \text{ kN/m}^2$$

(ii) Design of depth as per bending moment



$$\text{Overhang} = \frac{(3520 - 450)}{2}$$

$\Rightarrow$ $$O_x = O_y = 1535 \text{ mm}$$

$\therefore$ $$M_{xx} = M_{yy} = 181.60 \times 3.520 \times \frac{1.535^2}{2} = 753.087 \text{ kNm}$$

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

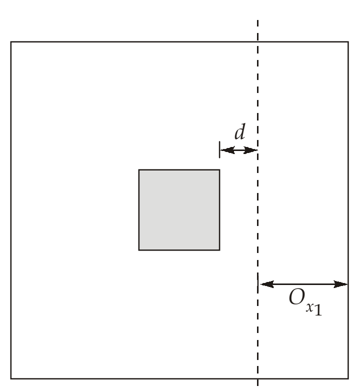
$\Rightarrow$ $$753.087 \times 10^6 = 0.138 \times 20 \times 3520 \times d^2$$

$\Rightarrow$ $$d = 278.42 \text{ mm}$$

$\therefore$ $$\text{Total depth, } D = 278.42 + 75 + \frac{16}{2} = 361.42 \text{ mm} \simeq 375 \text{ mm (say)}$$

$$d = 375 - 75 - \frac{16}{2} = 292 \text{ mm}$$

(iii) Check for one-way shear



Critical section for shear

$$\text{Overhang} = O_{x_1} = O_{y_1} = \frac{3520 - 450}{2} - 292$$
$$= 1243 \text{ mm}$$
$$= 1.243 \text{ m}$$

One-way shear force, $V_{ux_1} = V_{uy_1} = 181.60 \times 3.52 \times 1.243 = 794.6$ kN

$$\tau_{ux_1} = \tau_{vy_1} = \frac{794.6 \times 10^3}{3520 \times 292} = 0.773 \text{ N/mm}^2$$

$\tau_{c,\ min} = 0.28 \text{ N/mm}^2$ for M20 as per Table 19 of IS 456:2000 $\not> \tau_{ux_1}$

Now, $\tau_{ux_1} = \tau_{uy_1} = 0.773 \text{ N/mm}^2$ to too high.

∴ Depth of footing needs to be increased

Let $D = 750$ mm

∴ $$d = 750 - 75 - \frac{16}{2} = 667 \text{ mm}$$

∴ $$\text{Overhang, } O_{x_1} = O_{y_1} = \frac{3520 - 450}{2} - 667 = 868 \text{ mm}$$

∴ $$V_{ux_1} = V_{uy_1} = 181.6 \times 3.52 \times 0.868 \text{ kN}$$
$$= 554.85 \text{ kN}$$

∴ $$\tau_{ux_1} = \tau_{uy_1} = \frac{554.85 \times 10^3}{3520 \times 667}$$
$$= 0.24 \text{ N/mm}^2 < \tau_{min} (= 0.28 \text{ N/mm}^2) \qquad OK$$

(iv) Check for punching or two-way shear

Net punching shear force, $P_u = 1.5 \times 1500 - w_u (a + d)^2$
$$= 1.5 \times 1500 - 181.60 \times (0.45 + 0.667)^2$$
$$= 2023.42 \text{ kN}$$

$$\text{Punching shear stress} = \frac{2023.42 \times 10^3}{2[a + d + a + d] \times d}$$
$$= \frac{2023.42 \times 10^3}{4(450 + 667)667} = 0.68 \text{ N/mm}^2$$

Maximum permissible punching shear stress

$$\tau_{c,max} = k_s 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} \qquad (\text{where } k_s = 1)$$

$$= 1.118 \text{ N/mm}^2 > 0.68 \text{ N/mm}^2 \qquad \text{So OK}$$

(v) Area of steel

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

$$= \frac{0.5(20)}{415}\left[1-\sqrt{1-\frac{4.6\times 753.087\times 10^6}{20\times 3520\times 667^2}}\right]3520\times 667$$

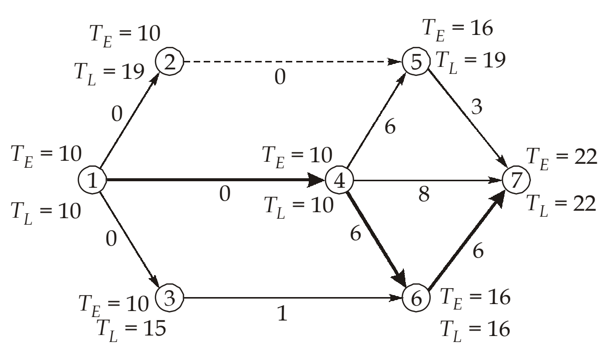
$$= 3220.4 \text{ mm}^2$$

$\therefore$ Number of 16 mm diameter bars required $= \dfrac{3220.4}{\frac{\pi}{4}\times 16^2} = 16.02 \simeq 17$ bars

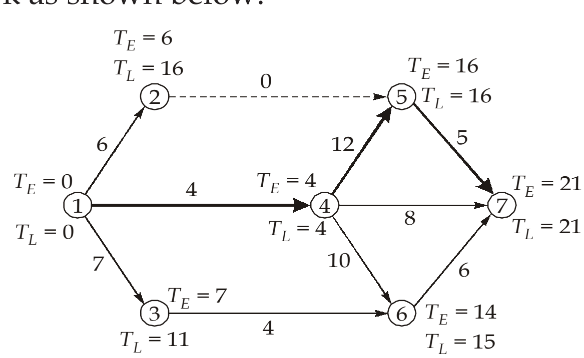
Q.8 (b) Solution:

(i)

Updated network is shown in figure below.



The original network as shown below:



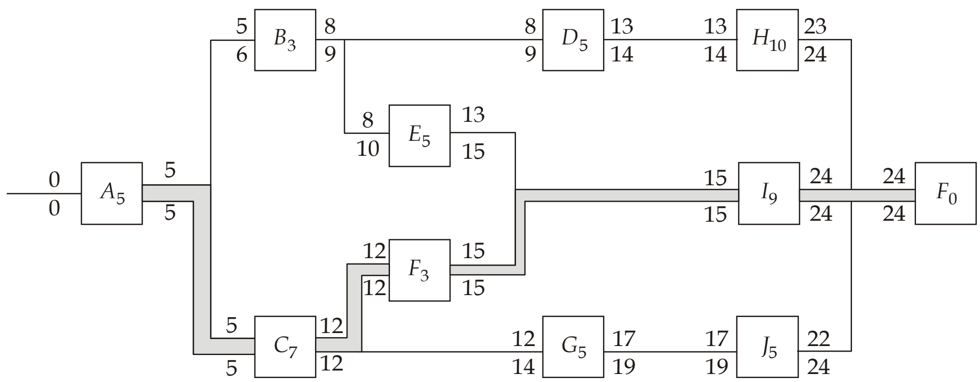
∴ Original project duration = 21 days

∴ Increase in project duration = 22 – 21 = 1 day

Critical path of updated network is : 1 – 4 - 6 – 7.

(ii)

AON diagram for the given A-O-A diagram is shown in figure below.



F_0 = End activity of zero duration

Critical path = A – C – F – I, project duration = 24 units

EST, EFT, LST and LFT values have been indicated on the A-O-N diagram. However, the values are tabulated as below.

Activity	Duration	EST	EFT	LST	LFT	TF	Remarks
A	5	0	5	0	5	0	Critical
B	3	5	8	6	9	1	
C	7	5	12	5	12	0	Critical
D	5	8	13	9	14	1	
E	5	8	13	10	15	2	
F	3	12	15	12	15	0	
G	5	12	17	14	19	2	Critical
H	10	13	23	14	24	1	
I	9	15	24	15	24	0	Critical
J	5	17	22	19	24	2	

Q.8 (c) Solution:

Principle moment of inertia,

$$I_{uu}/I_{vv} = \frac{I_{xx}+I_{yy}}{2} \pm \sqrt{\left(\frac{I_{xx}-I_{yy}}{2}\right)^2 + I_{xy}^2}$$

$$= \frac{2\times10^7 + 0.97\times10^7}{2} \pm \sqrt{\left(\frac{2-0.97}{2}\right)^2 \times 10^{14} + \left(-0.83\times10^7\right)^2}$$

$$= 1.485 \times 10^7 \pm 0.977 \times 10^7$$

$$I_{uu} = 2.462 \times 10^7 \text{ mm}^4$$

$$I_{vv} = 0.508 \times 10^7 \text{ mm}^4$$

$\therefore$ $$I_{min} = I_{vv} = 0.508 \times 10^7 \text{ mm}^4$$

Minimum radius of gyration,

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{0.508\times10^7}{5025}} = 31.8 \text{ mm}$$

Given, $l_{eff} = 4.5$ m

So, slenderness ratio, $$\lambda = \frac{l_{eff}}{r_{min}} = \frac{4.5\times10^3}{31.8} = 141.51$$

From table, $$f_{cd} = 51 - \frac{51-45}{150-140}\times(141.51-140)$$

$$= 50.094 \text{ MPa}$$

Safe load $= f_{cd} \times A$

$\Rightarrow$ $P = 50.094 \times 5025 \times 10^{-3}$ kN

$\Rightarrow$ $P = 251.72$ kN

❍❍❍❍