MADE EASY
Leading Institute for ESE, GATE & PSUs

Detailed Solutions

ESE-2024 Mains Test Series

Civil Engineering Test No : 10

Section - A

Q.1 (a) Solution:

Quarrying the stone:

The process of extracting the stone from the natural rock bed is known as quarrying of stone. The term quarry is used to indicate exposed surface of natural rock. Thus, quarrying of stone is carried out almost at or near the ground level in an exposed condition. The site from where stone is extracted is called the quarry site. The selection of quarry site for building stone is generally based on the following factors:

1. Geological information regarding the formation of rock and availability of sufficient quantity of stone at or near the ground level of the site should be available. The quality of stone at the site should not vary significantly with the depth.
2. The quarry site should be easily approachable so that quarrying equipment can be carried to and installed there. For the ease and economy in transportation of stone, the quarry should be near a motorable road.
3. The local manpower and electricity should be available at the site in sufficient quantity for quarrying operations.
4. Space for disposal of refuse and quarry waste should be available near the quarry site. Source of clean and clear water, and facilities for providing drainage from quarry pits should also be available.

5. There should not be any populated areas nearby in case of quarrying by blasting.
6. Quarrying should not cause any health hazards.

Methods of quarrying: Following methods are generally used for quarrying the building stone: (i)quarrying with hand tools, (ii) quarrying with channelling machine, and (iii) quarrying with blasting. Small scale extraction of rocks is possible with simple hand tools such as drills, wedges and hammers, but skill and experience is essential to ensure accurate cuts. Harder rocks, such as granite require more sophisticated mechanised equipment. Blasting is the most commonly used method for medium to large scale extraction of rocks. The main operations are:

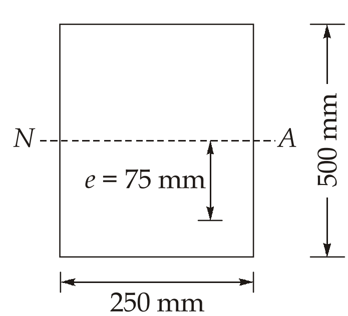
1. **Boring:** The blast holes of appropriate diameter and depth are made with a jumper. However, in case of bigger diameter hole or harder rock, the holes may be drilled with pneumatic or percussion drilling machine.
2. **Charging:** The charge of gun powder or dynamite (gun cotton) is placed into the dried blast hole with fuse chord placed in its position. About one metre of the fuse chord is kept projecting out the blast hole.
3. **Tamping:** The blast hole is filled up with damp clay and rammed hard with tamping rod. Detonators are sometimes used in place of fuse to explode the dynamite.
4. **Firing**: The charge placed inside the blast holes is fired either with the help of match stick or electric spark. However, in case of dynamite the detonators act on electric spark.

Q.1 (b) Solution:

Given data; B = 250 mm, D = 500 mm, e = 75 mm, A_{st} = 150 mm^2

σ_i = 1200 N/mm^2, w_L = 8 kN/m, L = 10 m, E_s= 2.1 × 10^5 N/mm^2

E_c = 3 × 10^4 N/mm^2, γ_c = 24 kN/m^3



Load calculations

$$\text{Dead load, } w_d = 0.25 \times 0.5 \times 1 \times 24 = 3 \text{ kN/m}$$

$$\text{Live load, } w_L = 8 \text{ kN/m}$$

$$\text{Total load, } w_T = 3 + 8 = 11 \text{ kN/m}$$

$$\text{Moment of inertia, } I = \frac{B.D^3}{12} = \frac{250\times500^3}{12} = 26.04 \times 10^8 \text{ mm}^4$$

$$\text{Prestressing force, } P = \sigma_i \times A_{st} = 1200 \times 150 \times 10^{-3} \text{ kN} = 180 \text{ kN}$$

where σ_i is initial stress in wire

Now, rotation due to prestress, $\theta_p = \dfrac{PeL}{2EI}$

$$\Rightarrow \qquad \theta_p = \frac{180\times10^3\times75\times10\times10^3}{2\times3\times10^4\times26.04\times10^8} = 0.000864 \text{ radians}$$

$$\text{Rotation due to load,} \qquad \theta_c = \frac{wL^3}{24EI} = \frac{11\times(10,000)^3}{24\times3\times10^4\times26.04\times10^8} = 0.005867$$

$$\therefore \qquad \text{Net rotation, } (\theta) = 0.005867 - 0.000864 = 0.005003 \text{ radian}$$

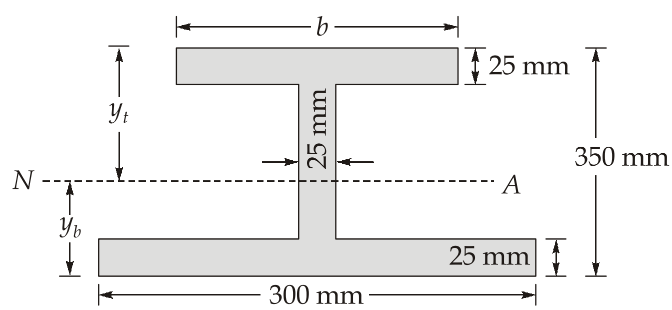
$$\text{Now} \qquad \text{elongation of the cable} = 2 \times e \times \theta = 2 \times 75 \times 0.05003 = 0.75 \text{ mm}$$

$$\therefore \qquad \text{Increase in stress} = \frac{0.75}{10,000}\times2.1\times10^5 = 15.75 \text{ N/mm}^2$$

$$\text{and percentage increase in stress} = \frac{15.75}{1200}\times100 = 1.3125\%$$

Q.1 (c) Solution:

Let, y_t and y_b are distances of neutral axis from top and bottom fibre respectively.



$$\text{Now,} \qquad y_t = \frac{b\times25\times\left(\frac{25}{2}\right)+25\times(350-50)\times175+300\times25\times337.5}{b\times25+25\times(350-50)+300\times25}$$

$$\Rightarrow \qquad y_t = \frac{312.5b+3843750}{25b+15000} \qquad \text{...(i)}$$

$$\text{Now,} \qquad \frac{\sigma_t}{\sigma_b} = \frac{4}{3} \qquad \text{...(ii)}$$

where σ_t and σ_b are maximum tensile and compressive stresses at top and bottom fibre respectively.

Also, $$\frac{\sigma_t}{\sigma_b} = \frac{y_t}{y_b} \quad \text{...(iii)}$$

Comparing (ii) and (iii), we get

$$\frac{y_t}{y_b} = \frac{4}{3} \quad \text{...(iv)}$$

Also, $$y_t + y_b = 350 \text{ mm} \quad \text{...(v)}$$

From (iv) and (v), we get

$$y_t = 200 \text{ mm and } y_b = 150 \text{ mm}$$

Putting value of y_t in (i), we get

$$200 = \frac{312.5b + 3843750}{25b + 15000}$$

$$\Rightarrow \quad 5000b + 3000000 = 312.5b + 3843750$$

$$\Rightarrow \quad 4687.5b = 843750$$

$$\Rightarrow \quad b = 180 \text{ mm}$$

Q.1 (d) Solution:

(i) Desirable Properties of a good brick:

The properties of good brick for construction are as follows:

1. It should be table moulded well burnt in kiln to a uniform bright red or copper colour.
2. It should be reasonably hard such that finger nail does not leave a mark on scratching the bricks surface.
3. It should be of uniform size and shape with sharp edges and well-defined corners.
4. It should be free from cracks, flaws or stone grit and lime nodules. The surface should be clean and even but not very smooth.
5. It should produce clear ringing metallic sound when struck against with another brick without breaking.
6. Brick when broken should reveal a homogeneous and compact texture on the interior broken face.
7. The brick should not absorb water more than 15 per cent of its dry weight when immersed in water for 24 hours.
8. The brick soaked in water for 24 hours should not show any deposits of white salts on drying in shade, i.e., there should not be any efflorescence.
9. No brick should have crushing strength less than 5.5 MPa. The brick should have sufficient strength from structural considerations.

10. The brick should not break into pieces when dropped on hard ground from a height of about one metre.
11. The brick should have low thermal conductivity and high sound insulation property.

However, since there is wide variation in the properties of bricks, the bricks are generally classified into different groups to facilitate the selection of bricks for a particular job and to maintain quality standards.

(ii) High-performance concrete (HPC): A performance enhanced concrete or high performance concrete (HPC) is a specialised series of concrete designed to provide several benefits in the construction of concrete structures that cannot always be achieved routinely using conventional ingredients, normal mixing and curing practices. In other words a high performance concrete is a concrete in which certain characteristics are developed for a particular application and environment, so that it will give excellent performance in the structure in which it will be placed, in the environment to which it will be exposed, and with the loads to which it will be subjected during its design life. It includes concrete that provides either substantially improved resistance to environmental influences (durability in service) or substantially increased structural capacity while maintaining adequate durability. It may also include concrete, which significantly reduces construction time without compromising long term serviceability. It is therefore, not possible to provide a unique definition of HPC without considering the performance requirements of the intended use of the concrete. Examples of characteristics that may be considered critical in an application requiring performance enhancement are: Ease of placement and compaction without segregation, early-age strength, long-term mechanical properties, permeability, density, heat of hydration, toughness, volume stability, and long life in severe environments, i.e., durability. Concretes possessing many of these characteristics often achieve higher strength. Therefore, HPC is often of high strength but high strength concrete may not necessarily be of high performance.

Q.1 (e) Solution:

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

$$\delta = \frac{WL}{AE} = \frac{WL}{\left(\frac{\pi}{4}d^2E\right)} = \frac{4WL}{\pi d^2 E}$$

$$\Rightarrow \qquad E = \frac{4WL}{\pi d^2 \delta} \qquad \text{...(i)}$$

As the vertical rod is under the action of torque T, we have

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

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

[where J = polar moment of inertia = $\frac{\pi d^4}{32}$]

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

$$\Rightarrow \quad 1+\mu = \frac{E}{2G} = \frac{4WL}{\pi d^2\delta}\times\frac{1}{2}\times\frac{\pi d^4\theta}{32TL} = \frac{Wd^2\theta}{16T\delta}$$

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

Q.2 (a) Solution:

Given,

$\varepsilon_x = -0.00075$, $\varepsilon_y = 0.00125$, $\phi_{xy} = 0.001$

Modulus of elasticity, $E = 73 \times 10^3\ \text{N/mm}^2$

Poisson's ratio, $v = 0.3$

(1) Now,

$$\sigma_x = \frac{E}{1-\mu^2}\left(\varepsilon_x + \mu\varepsilon_y\right) = \frac{73\times10^3}{1-0.33^2}(-0.00075+0.33\times0.00125)$$

$$= -27.65\ \text{N/mm}^2$$

$$\sigma_y = \frac{E}{1-\mu^2}\left(\varepsilon_y + \mu\varepsilon_x\right)$$

$$= \frac{73\times10^3}{1-0.33^2}\left(0.00125+0.33\times(-0.00075)\right)$$

$$= 82.13\ \text{N/mm}^2$$

Now, strain in z-direction, ε_z is given as

$$\varepsilon_z = \frac{-\mu}{E}\left(\sigma_x+\sigma_y\right) = \frac{-0.33}{73\times10^3}(-27.65+82.13) = -2.46\times10^{-4}$$

Therefore, change in thickness (Δt) = ε_z × thickness of plate = $-2.46 \times 10^{-4} \times 10$

$$= -2.46 \times 10^{-3}\ \text{mm}$$

Here, (–ve) sign indicates that there is decrease in thickness of plate.

(2) Volumetric strain, $\varepsilon_v = \frac{(1-2\mu)}{E}\left(\sigma_x + \sigma_y\right) = \frac{(1-2\times0.33)}{73\times10^3}(-27.65+82.13) = 2.54 \times 10^{-4}$

Now, volume of plate, $V = 200 \times 180 \times 10 = 3.6 \times 10^5 \text{ mm}^3$

So, change in volume of plate

$$\Delta V = \varepsilon_V \times V = 2.54 \times 10^{-4} \times 3.6 \times 10^5 = 91.44 \text{ mm}^3$$

(3) Principal stresses, $\sigma_1/\sigma_2 = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^{\ 2}}$

where, τ_{xy} is shear stress and is given as $\tau_{xy} = G\phi_{xy}$

where G is shear modulus and is given as

$$G = \frac{E}{2(1+\mu)} = \frac{73\times10^3}{2\times(1+0.33)} = 2.744 \times 10^4 \text{ N/mm}^2$$

So, $\tau_{xy} = 2.744 \times 10^4 \times 0.001 = 27.44 \text{ N/mm}^2$

Now, $\sigma_1/\sigma_2 = \frac{-27.65+82.13}{2} \pm \sqrt{\left(\frac{-27.65-82.13}{2}\right)^2 + 27.44^2}$

$$= 27.24 \pm \sqrt{54.89^2 + 27.44^2} = 27.24 \pm 61.37$$

$$= 88.61 \text{ MPa}, -34.13 \text{ MPa}$$

Now, strain in direction of σ_1, $\varepsilon_1 = \frac{\sigma_1}{E} - \mu \times \frac{\sigma_2}{E} = \frac{1}{73\times10^3}(88.61 - 0.33\times(-34.13))$

$$= 1.368\times10^{-3}$$

Strain in direction of σ_2, $\varepsilon_2 = \frac{\sigma_2}{E} - \frac{\mu\sigma_1}{E} = \frac{1}{73\times10^3}(-34.13 - 0.33\times88.61)$

$$= -8.68 \times 10^{-4}$$

(4) If there is no change in volume, then $\varepsilon_V = 0$

$$\Rightarrow \left(\frac{1-2\mu}{E}\right)\left(\sigma_x + \sigma_y + \sigma_z\right) = 0$$

$$\Rightarrow \sigma_x + \sigma_y + \sigma_z = 0$$

$$\Rightarrow -27.65 + 82.13 + \sigma_z = 0$$

$$\Rightarrow \sigma_z = -54.48 \text{ N/mm}^2$$

$\therefore$ Axial force, $P = 180 \times 10 \times \sigma_z = 180 \times 10 \times (-54.48)$

$$= -98.06 \times 10^3 \text{ N} = -98.06 \text{ kN}$$

Q.2 (b) Solution:

Given: B = 500 mm, D = 700 mm

- $P_u = 1.5 \times 1500 = 2250$ kN
- $M_{ux} = 1.5 \times 120 = 180$ kN-m
- $M_{uy} = 1.5 \times 100 = 150$ kN-m

Unsupported length

- $L_{ox} = 15 - 0.8 = 14.2$ m
- $L_{oy} = 15 - 0.7 = 14.3$ m

Effective length

As the column is fixed at both ends, so unsupported lengths will be multiplied by 0.65.

- $L_{ex} = 0.65 \times 14.2 = 9.23$ m
- $L_{dy} = 0.65 \times 14.3 = 9.295$ m
- $d' = 70$ mm

Check

$$\frac{L_{ex}}{D} = \frac{9.23}{0.7} = 13.186 > 12$$

$$\frac{L_{ey}}{B} = \frac{9.295}{0.5} = 18.59 > 12$$

$\therefore$ It is a long column

$$\text{Minimum eccentricity, } e_{min,x} = \frac{L_{ox}}{500}+\frac{D}{30} = \frac{14200}{500}+\frac{700}{30} = 51.73 \text{ mm}$$

$$e_{min,y} = \frac{L_{oy}}{500}+\frac{B}{30} = \frac{14300}{500}+\frac{500}{30} = 45.267 \text{ mm}$$

$$M_{ux,\,\min} = P_{ux}.e_{min\ x} = 2250 \times 51.73 \times 10^{-3} \text{ kN-m}$$

$\Rightarrow$ $= 116.39$ kN-m

$\therefore$ $M_{ux,\,\min} < M_{ux}$ (= 180 kN-m)

Hence, $M_{ux} = 180$ kN-m ok

$$M_{uy,\,\min} = P_{ux}.e_{min,\,y} = 2250 \times 45.267 \times 10^{-3} \text{ kN-m}$$

$\Rightarrow$ $= 101.85$ kN-m

$\therefore$ $M_{uy,\,\min} < M_{uy}$ (150 kN-m)

Hence, $M_{uy} = 150$ kN-m

Now additional moment:

$$M_{ax} = \frac{P_u.D}{2000}\left[\frac{L_{ex}}{D}\right]^2 = \frac{2250\times0.7}{2000}\left[\frac{9.23}{0.7}\right]^2 = 136.92 \text{ kN-m}$$

$$M_{ay} = \frac{P_u.B}{2000}\left[\frac{L_{ey}}{B}\right]^2 = \frac{2250\times0.5}{2000}\left[\frac{9.295}{0.5}\right]^2 = 194.39 \text{ kN-m}$$

Check $\left(\frac{M_{ux}+M_{ay}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}+M_{ay}}{M_{uy_1}}\right)^{\alpha_n} \le 1$

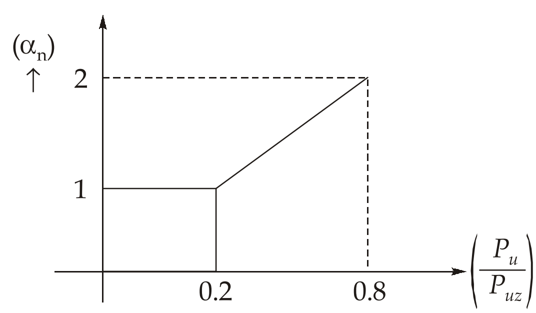
- Assume 2% area of steel.
- $P_{uz} = 0.45 f_{ck} . A_g + (0.75 f_y - 0.45 f_{ck}). A_{sc}$

$= 0.45 \times 30 \times 500 \times 700 + (0.75 \times 415 - 0.45 \times 30) \times 0.02 \times 500 \times 700$

$= 6809250 \text{ N} = 6809.25 \text{ kN}$

Now, $\frac{P_u}{P_{uz}} = \frac{2250 \text{ kN}}{6809.25 \text{ kN}} = 0.33$

Now α_n is calculated as below:



$$\frac{2-1}{0.8-0.2} = \frac{\alpha_n - 1}{0.33-0.2}$$

$\Rightarrow$ $\alpha_n = 1.217$

M_{ux1} Calculation: $\frac{d'}{D} = \frac{70}{700} = 0.1$ and Fe415 steel

Now from the Chart 44, $\frac{P_u}{f_{ck}.BD} = \frac{2250\times10^3}{30\times500\times700} = 0.21$

$$\frac{p_t}{f_{ck}} = \frac{2}{30} = 0.067$$

$\therefore$ $\frac{M_{ux_1}}{f_{ck}.BD^2} = 0.11$

$\Rightarrow$ $M_{ux1} = 0.11 \times 30 \times 500 \times 700^2 \times 10^{-6}$ kN-m

$= 808.5$ kN-m

M_{uy1} calculation: $\frac{d'}{B} = \frac{70}{500} = 0.14$ and Fe415 steel.

$$\frac{p\%}{f_{ck}} = \frac{2}{30} = 0.067$$

$$\therefore \quad \frac{M_{uy_1}}{f_{ck}.BD^2} = 0.10$$

$$\Rightarrow \quad M_{uy1} = 0.10 \times 30 \times 500 \times 700^2 \times 10^{-6} \text{ kn-m}$$

$$= 735 \text{ kN-m}$$

Check, $\left(\frac{M_{ux}+M_{ax}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}+M_{ay}}{M_{uy_1}}\right)^{\alpha_n} \le 1$

$$\Rightarrow \quad \left(\frac{180+136.92}{808.5}\right)^{1.217} + \left(\frac{150+194.39}{735}\right)^{1.217} \le 1$$

$$\Rightarrow \quad 0.717 \le 1$$

Now, so, $A_{sc} = 0.02 \times 500 \times 700 = 7000 \text{ mm}^2$

28 mm dia. bars NOT commercially available.

Provide, 12 – 28 mm diameter bars.

Design of lateral ties:

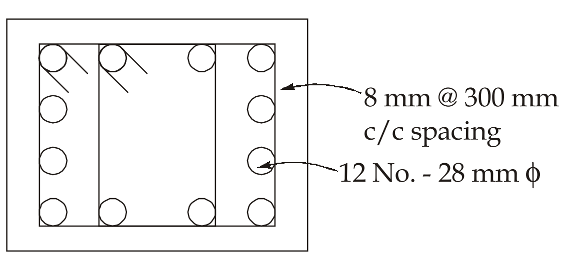
(i) Diameter $= \max.\begin{cases} \frac{\phi_{main}}{4} = \frac{28}{4} = 7 \text{ mm} \\ 6 \text{ mm} \end{cases}$

Provide 8 mm diameter ties.

(ii) Spacing $= \min.\begin{cases} B = 500 \text{ mm} \\ 16\,\phi = 16 \times 28 = 448 \text{ mm} \\ 300 \text{ mm} \end{cases}$

$\therefore$ Provide 300 mm spacing c/c.

8 mm ties @



Q.2 (c) Solution:

(i) Ferrocement: Ferrocement may be considered as a type of thin reinforced concrete construction where cement mortar matrix is reinforced with many layers of continuous and relatively small diameter wire meshes. While the mortar provides the mass, the wire mesh imparts tensile strenth and ductility to material. In terms of structural behaviour, ferrocement exhibits very high tensile strength to weight ratio, superior cracking performance, high toughness, fatigue resistance, impermeability etc.

Materials:

1. **Cement mortar matrix:** The ferrocement composite is a rich cement-sand mortar matrix of 10 to 60 mm thickness with reinforcement volume of five to eight percent in the form of one or more layers of very thin wire mesh and a skeleton reinforcement consisting of either welded-mesh or mild steel bars.

   Normally, Portland cement and fine aggregate matrix used in ferrocement constitutes about 95 per cent of the ferrocement and governs the behaviour of the final product. The fine aggregate (sand) which is the inert material occupying 60 to 75 per cent of the volume of mortar must be hard, strong, non-porous and chemically inert, and should be free from impurities. The fine aggregates conforming to grading zones II and III with particle size greater than 2.36 mm and smaller than 150 mm removed are suitable for ferrocement. The water used for making mortar should be free from impurities.

   Plasticisers and other admixtures may also be added for achieving:

   (i) an improved workability,

   (ii) water reduction for increase in strength and reduction in permeability,

   (iii) water-proofing,

   (iv) increase in durability. Pozzolanas such as flyash may be added as cement replacement materials (up to 30 per cent) to increase the durability.

2. **Mix proportions:** The mix proportions in terms of sand-cement ratio (by mass) normally recommended are 1.5 to 2.5. The water-cement ratio may vary between 0.35 and 0.6. In order to reduce permeability, the water-cement ratio must be kept below 0.4.

3. **Skeleton steel:** The skeleton steel comprises relatively large-diameter (about 3 to 8 mm) steel rods typically spaced at 70 to 100 mm. It may be tied-reinforcement or welded wire fabric.

4. **Wire mesh:** The wire mesh consisting of galvanised wire of diameter 0.5 to 1.5 mm spaced at 6 to 20 mm centre-to-centre, is formed by welding, twisting or

weaving. Meshes with hexagonal openings are sometimes referred to as efficient than the mesh with square openings.

The placing of the mortar is termed as impregnation of meshes with matrix. A sufficient quantity of mortar is impregnated through mesh layers so that the mortar reaches the other side and there are no voids left in the surface.

Properties of ferrocement:

1. Since ferrocement is a system of construction using layers of closely spaced wire mesh separated by skeleton bars and filled with cement-sand mortar presents all the mechanical characteristics of a homogeneous material.
2. Due to the very high percentage of well distributed and continuously running steel reinforcement, the ferrocement behaves as steel plates. Ferrocement combines easy modulability of concrete to any desired shape, lightness, tenacity and toughness of steel plates. Due to very high tensile strength- to-weight ratio and superior cracking behaviour, the ferrocement is an attractive material for light and watertight structure and other portable structures such as mobile homes. The other specialised applications include water tanks, pipes, folded plates and shell roofs, floor units, kiosks, service core units, modular housing, and permanent forms of concrete columns. Saving in steel consumption of the order of 10 per cent in roof cost has been estimated in USSR.
3. Ferrocement is suitable for manufacturing the precast units which can be easily transported.
4. The construction technique is simple and hence does not require highly skilled labour, even for complicated forms.
5. Partial or complete elimination of formwork is possible.
6. Ferrocement construction is easily amenable to repairs in case of local damage due to abnormal loads (such as impact).

(ii) Fineness modulus (FM)

- Fineness modulus is defined as sum of cumulative percentage retained on the sieves of the standard sizes, divided by 100. Sieves of standard size are 150 μm, 300 μm, 600 μm, 1.18 mm, 2.36 mm, 4.75 mm, 10 mm, 12.5 mm, 20 mm, 40 mm and 80 mm.
- It is a numerical index of fineness, giving an idea about the mean size of particles in aggregate.
- FM varies as 2 to 3.5 for fine aggregate, 5.5 to 8.0 for coarse aggregate and 3.5 to 7.5 for all in aggregates.

- Aggregate whose FM is required, is placed on a standard set of sieves and the set vibrated. Material retained on each sieve after sieving represents the fraction of aggregate coarser than the sieve in question but finer than the sieve above.
- Sum of the cumulative percentages retained on the sieves divided by 100 gives the FM.
- Coarse aggregate have higher fineness modulus.
- Fineness modulus of 3 represents third sieve i.e. 600μm, is the average size. Similarly FM = 4 represents 1.18 mm as average size of aggregate.
- Purpose of finding FM is to grade the given aggregate for the required strength and workability of concrete mix with minimum cement.
- Higher FM aggregate results in harsh concrete mixes and lower FM results in uneconomical concrete mixes.

Q.3 (a) Solution:

Width of section A-A = 300 mm

Overall depth at section A-A,

$$D = 300 + \frac{600-300}{3} \times 2 = 500 \text{ mm}$$

Effective depth, $d = 500 - 50 = 450$ mm

$$\text{Area of tension steel} = 2\left[\frac{\pi}{4} \times 25^2 + \frac{\pi}{4} \times 32^2\right] = 2590.24 \text{ mm}^2$$

$$\text{Percentage of tension steel} = \frac{2590.24 \times 100}{300 \times 500} = 1.73\%$$

∴ Shear strength of concrete (from table),

$$\frac{0.75-0.72}{1.75-1.50} = \frac{\tau_c - 0.72}{1.73-1.50}$$

$$\tau_c = 0.7476 \text{ N/mm}^2$$

Maximum shear stress, $\tau_{c\,max} = 0.625\sqrt{f_{ck}} = 0.625\sqrt{20} = 2.8 \text{ N/mm}^2$

Nominal shear stress, $$\tau_v = \frac{V_u - \left(\frac{M_u}{d}\right)\tan\beta}{B.d}$$

where $V_u = 1.5 \times 150 = 225$ kN

$M_u = 1.5 \times 200 = 300$ kN-m

$$\tan\beta = \frac{600-300}{3000} = 0.1$$

Now, $\tau_v = \dfrac{225\times10^3 - \left(\dfrac{300\times10^6}{500}\right)\times0.1}{300\times500} = 1.1\ \text{N/mm}^2$

$\therefore \quad \tau_v = 1.1\ \text{MPa} < \tau_{c\,max}\ (2.8\ \text{MPa})$ (OK)

But $\tau_v > \tau_c$, hence shear reinforcement is provided.

Design shear force, $V_{us} = V_u - V_c = (\tau_v - \tau_c)B.d$

$= (1.1 - 0.7476) \times 300 \times 500 \times 10^{-3}\ \text{kN} = 52.86\ \text{kN}$

Using 2 legged 8 mm ϕ stirrups:

Spacing of stirrups, $S_v = \dfrac{0.87 f_y A_{sv} d}{V_{us}} = \dfrac{0.87\times415\times2\times\dfrac{\pi}{4}\times8^2\times450}{52.86\times10^3}$

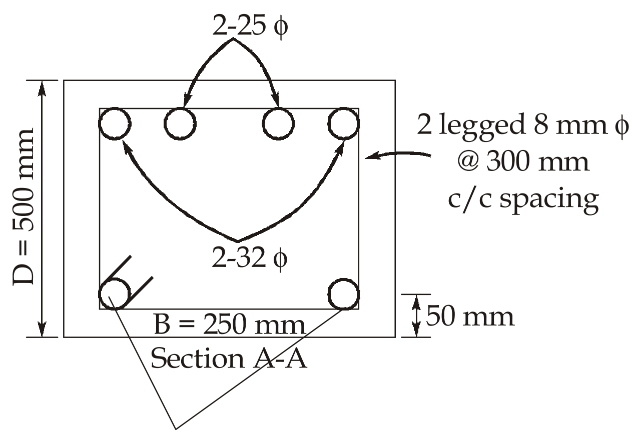
$\simeq 309$ mm

Check,

Spcaing, $S_v = \text{Minimum}\begin{cases}0.75d\\300\ \text{mm}\end{cases} = \text{Minimum}\begin{cases}0.75\times450 = 337.5\ \text{mm}\\300\,\text{mm}\end{cases}$

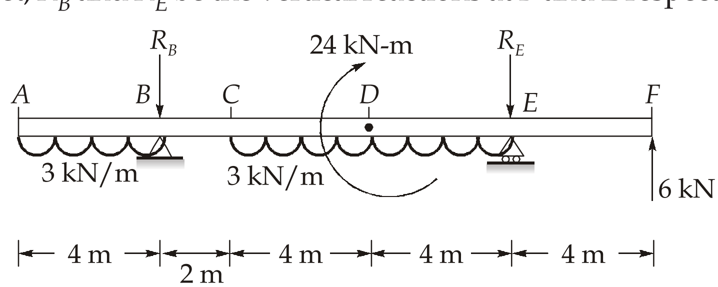
Hence, provide 2 legged 8 mm stirrups @ of 300 mm c/c.

The shear reinforcement is shown below:



Q.3 (b) Solution:

- **Reactions:** Let, R_B and R_E be the vertical reactions at B and E respectively as shown below.



Now, $\sum F_y = 0$

$\Rightarrow$ $3 \times 4 - R_B + 3 \times 8 - R_E + 6 = 0$

$\Rightarrow$ $R_B + R_E = 42$ kN

Also, $\sum M_E = 0$

$\Rightarrow$ $3 \times 4 \times 12 - R_B \times 10 + 3 \times 8 \times 4 + 24 - 6 \times 4 = 0$

$\Rightarrow$ $R_B = 24$ kN

$\Rightarrow$ $R_E = 42 - 24 = 18$ kN

(i) Shear force diagrams:

Take a section $x - x$ at distance x from A

Portion *AB*

$$S_x(x \text{ from } A) = 3x \qquad [0 \le x < 4]$$

At $x = 0$, $S_x = 0$

At $x = 4$ m $S_x = 3 \times 4 = 12$ kN

Portion *BC*

$$S_x(x \text{ from } A) = 3 \times 4 - R_B \qquad [4 \le x < 6]$$

At $x = 4$ m, $S_x = 3 \times 4 - 24 = 12 - 24 = -12$ kN

At $x = 6$ m, $S_x = 3 \times 4 - 24 = -12$ kN

Portion *CE*

$$S_x(x \text{ from } A) = 3 \times 4 - R_B + 3(x - 6) \qquad [6 \le x < 14]$$

At $x = 6$ m, $S_x = 3 \times 4 - 24 + 3 \times (6 - 6) = 12 - 24 + 0 = -12$ kN

At $x = 14$ m, $S_x = 3 \times 4 - 24 + 3\,(14 - 6) = 12$ kN

Portion *EF*

$$S_x(x \text{ from } A) = 3 \times 4 - R_B + 3 \times 8 - R_E \qquad [14 \le x < 18]$$

At $x = 14$ m, $S_x = 12 - 24 + 24 - 18 = -6$ kN

At $x = 18$ m, $S_x = 12 - 24 + 24 - 18 = -6$ kN

At F, there is in upward load of 6 kN. So, shear force will be zero just right of F.

Bending moment diagram,

Take a section $x - x$ at distance x from A

Portion AB

$$M_x(x \text{ from } A) = \frac{3x^2}{2} \qquad [0 \le x < 4]$$

At $x = 0$ $\quad M_x = 0$

At $x = 4$ m $\quad M_x = 3\times\frac{4^2}{2} = 24\text{ kN-m}$

Portion BC

$$M_x(x \text{ from } A) = 3\times4\times(x-2) - R_B\times(x-4) \qquad [4 \le x < 6]$$

At $x = 4$m, $\quad M_x = 3\times4\times2 - 24\times(4-4)$

$= 24 - 0 = 24$ kN-m

At $x = 6$m, $\quad M_x = 3\times4\times(6-2) - 24\times(6-4) = 48 - 48 = 0$

Portion CD

$$M_x(x \text{ from } A) = 3\times4\times(x-2) - R_B\times(x-4) + \frac{3\times(x-6)\times(x-6)}{2} \qquad [6 \le x < 10]$$

At $x = 6$ m, $\quad M_x = 3\times4\times4 - 24\times(6-4) + 3\times0 = 48 - 48 = 0$

At $x = 10$ m, $\quad M_x = 3\times4\times(10-2) - 24\times(10-4) + \frac{3}{2}\times(10-6)^2$

$= 96 - 144 + 24 = -24$ kN-m

Portion DE

$$M_x(x \text{ from } A) = 3\times4\times(x-2) - R_B(x-4) + \frac{3\times(x-6)\times(x-6)}{2} + 24 \qquad [10 \le x < 14]$$

At $x = 10$ m, $\quad M_x = 3\times4\times8 - 24\times(10-4) + \frac{3\times4\times4}{2} + 24$

$= 96 - 144 + 24 + 24 = 0$

At $x = 14$ m, $\quad M_x = 3\times4\times12 - 24\times10 + \frac{3\times8\times8}{2} + 24$

$= 144 - 240 + 96 + 24 = 24$ kN-m

Portion EF

$$M_x(x \text{ from } A) = 3\times4\times(x-2) - R_B(x-4) + 3\times8\times(x-10) - R_E\times(x-14) + 24 \qquad [14 \le x < 18]$$

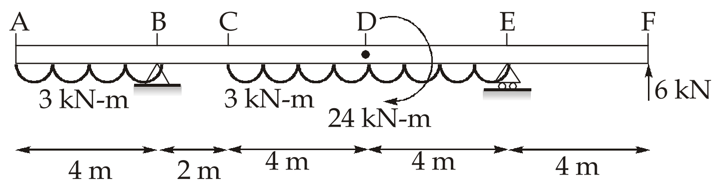
At $x = 14$ m, $\quad M_x = 3\times4\times12 - 24\times10 + 3\times8\times4 - 18\times(14-14) + 24$

$= 24$ kN-m

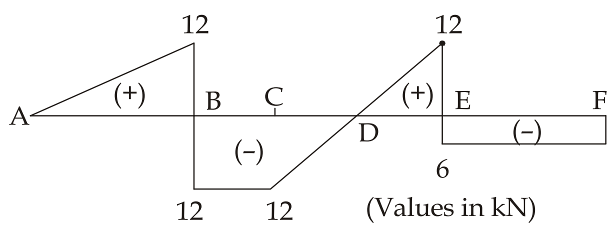
At $x = 18$ m, $\quad M_x = 3\times4\times16 - 24\times14 + 3\times8\times8 - 18\times4 + 24$

$= 0$

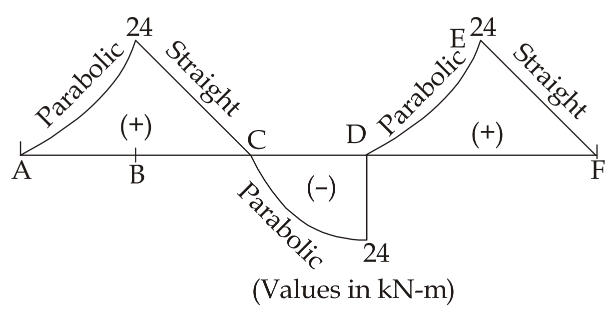
Loading diagram



Shear force diagram



Bending moment diagram,



Q.3 (c) Solution:

(i) Defects in Bricks Masonary

Brick masonary may develop defects due to following reasons:

1. Sulphate attack

- It is common defect, especially at locations where the brick work is either exposed (such as in boundary walls, unplastered external walls etc.) or where brick work is likely to come in contact with moisture.
- Sulphate salts present in brick react with hydraulic lime in the case of lime mortar and with alumina of cement in the case of cement mortar.
- Due to this reaction, the increase in the volume of mortar takes place resulting in chipping and spalling of bricks. Cracks are formed in joints and rendering.

2. Crystallization of salts from bricks

- If the bricks are manufactured from earth containing excessive soluble salts, entry of moisture, either due to dampness or due to rains etc, dissolves the soluble salts.

- Salts, after getting dissolved in water, appear in the form of fine whitish crystals on the exposed brick surface. This is known as efflorescence.
- This situation can be improved by brushing and washing on the affected surface from time to time.

3. **Corrosion of embedded fixtures**
   - Iron or steel fixtures, such as the pipes or holdfasts of doors windows etc. embedded in brick masonry gets corroded with time especially when lime mortar is used.
   - Corrosion results in the increase in the volume, resulting in cracks in brick masonry. Therefore, these figures should be well-embedded in cement mortar.

4. **Drying shrinkage**
   - When moisture penetrates the brick work, it swells.
   - On evaporation of moisture during the drying, due to atmospheric heat etc., the bricks shrink, resulting in the development of cracks in the masonry joints.

(ii) Heat treatment: Heat treatment is given to steel to derive following benefits:

(i) Increase in the heat and cold resistance
(ii) Increase in the surface hardness and strength
(iii Increase in the workability
(iv) To bring any desired change in the structure
(v) To alter magnetic and electrical properties of steel

Different methods for heat treatment of steel are as follows:

(1) Annealing: This process refers to making the steel soft so as to increase its workability upon machines. Tensile strength gets reduced whereas ductility gets increased. Toughness of steel is also enhanced against sudden stresses. Annealing temperature range depends upon the carbon content. As the carbon content increases, the required temperature for annealing decreases.

(2) Case hardening: By this process, the core of specimen is kept tough and ductile while the surface is made hard by increasing the carbon content near surface.

(3) Cementing: This process refers to saturating the skin of steel with carbon. It is achieved by heating the steel at a temperature between 880°C to 950°C in a carbon rich medium.

(4) Cyaniding: This process is used to make a hard case on the surface of low or medium carbon steel. Carbon and nitrogen are added to the surface to increase its hardness, wear resistance and fatigue limit.

(5) Hardening: In this process, the steel is made hard. This is just reverse to process of annealing of steel where steel is made soft. In both the process steel is heated, but in case of hardening the rate of cooling is fast and controlled. This process involves quenching. In case of annealing, the rate of cooling is slow.

(6) Normalizing: This process refers to the restoring the steel structure back to its normal condition.

Q.4 (a) Solution:

(i) Brick masonry have following advantages over stone masonry:

- Since the shape and size of bricks are uniform, it does not need highly skilled labour in construction.
- As bricks are light, they can be handled easily.
- Thinner walls can be constructed with bricks.
- In brick masonry, it is easy to form openings for doors and windows.
- Dead load of brick masonry is less.
- Brick masonry has good fire and weather resistance.
- It is possible to use all types of mortar in brick masonry.
- Bricks are easily available around cities and their transportation cost is less. The transportation cost of stones is higher as they are heavy and have to be brought from quarries, which are located far away from cities.

There are also some disadvantages with brick masonry:

- The strength of brick masonry is lower than that of stone masonry.
- Brick masonry is less durable.
- Brick masonry needs plastering and plastered surface needs colour washing/ painting. Hence, maintenance cost of brick masonry is high.
- Stone masonry gives massive appearance and hence monumental buildings are built with stone masonry.

(ii) Properties of timber for the use of timber as a structural element of a buildings:

I. Compressive strength:

- When subjected to compressive force acting parallel to the axis of growth, wood is found to be one of the strongest structural material. However, compressive strength perpendicular to fibres of wood is much lower than that parallel to fibres of wood. Compressive strength parallel to fibres, at 15 percent moisture content, varies from 30.0 to 77.5 N/mm^2.

- Furthermore, a knowledge of the compressive strength is of value in estimating strength in bending since experiments have demonstrated that the yield point of wooden beams is determined by the compressive strength of the wood. When wood is subjected to compression parallel to the grain, it may fail through collapsing of the cell walls or through lateral bending of the cells and fibres. In wet wood and in the hardwoods, which are composed of thick-walled fibres and vessels, incipient failure is due to bending of the individual fibres.
- In cross-grained pieces, the failure is likely to take place through shear parallel to the grain. The strength of timber compressed across the grain is brought into play whenever a concentrated load is imposed on a beam. Since the compressive strength across the grain is only a small fraction of the compressive strength parallel to the grain, proper allowance for this discrepancy must be provided with a footing to distribute the pressure.

II. Tensile Strength:

- When a properly shaped wooden stick is subjected to tensile forces acting parallel to the grain it is found to have greater strength that can be developed under any other kind of stress. Indeed, the tensile strength of wood parallel to the grain is so great that much difficulty is encountered in designing end connections so that the tensile strength of a piece can be developed. Therefore, wood tension members are rarely used. Tensile strength parallel to the fibres is of the order 80 to 190 N/mm^2.
- However, wooden parts restrained at their ends suffer from shearing stresses and crushing which wood resists poorly, and cannot be extensively used in structure working under tension. Moreover, since the tensile strength parallel to the grain is two to four times the compressive strength, the latter governs the strength of beams. The tensile strength parallel to the grain is influenced to some extent by the nature of the wood elements and their arrangement, but principally by the straightness of the grain and the thickness of the walls of the longitudinal elements.
- When failure occurs, these elements are ruptured transversely. Knots greatly reduce the tensile strength parallel to the grain. The tensile strength is less affected by moisture than are other mechanical properties. Across the grain, the tensile strength of wood is low. It is a property closely related to cleavability, and it often determines the strength of a beam which has cross-grain or spiral-grain in its tension fibres. Failure in tension across the grain occurs through separation of the cells and fibres in longitudinal planes.

Q.4 (b) Solution:

(i) Let, the compressive force in bar be 'R' when it is subjected to temperature rise.

Now, free extension of copper bar + contraction due to 'R' + contraction due to 'R' in spring = 0.2 mm.

$$\Rightarrow (L\alpha\Delta T)_{copper} - \frac{4RL}{\pi D_1 D_2 E} - \frac{R}{K} = 0.2$$

where D_1 and D_2 are diameters of bars at its ends.

$$\Rightarrow 600 \times 17.5 \times 10^{-6} \times 27° - \frac{4 \times R \times 600}{\pi \times 25 \times 50 \times 110 \times 10^3} - \frac{R}{210 \times 10^3} = 0.2$$

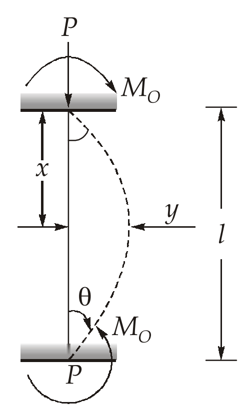
$$\Rightarrow \quad R = 8092.76 \text{ N}$$

Now, maximum compressive stress in bar, $\sigma_C = \dfrac{R}{\frac{\pi}{4} \times d_1^2} = \dfrac{8092.76}{\frac{\pi}{4} \times 25^2} = 16.49 \text{ N/mm}^2$

Also, Force in spring, $R = 8092.76$ N

Stiffness of spring, $k = 210 \times 10^3$ N/mm

Contraction in spring, $\Delta = \dfrac{R}{k} = \dfrac{8092.76}{210 \times 10^3} = 0.0385$ mm



(ii) Let, 'M_0' be restraining couple at end and 'θ' be the slope.

Now, $M_0 = K\theta$

Moment at distance x, $M_x = -Py + M_0$

$$\Rightarrow \quad EI\frac{d^2y}{dx^2} = -Py + M_0 \quad \left[\because M_x = EI\frac{d^2y}{dx^2}\right]$$

$$\Rightarrow \quad \frac{d^2y}{dx^2} + \frac{Py}{EI} = \frac{M_0}{EI} \quad \ldots(i)$$

Let $\dfrac{P}{EI} = \alpha^2$

$$\Rightarrow \quad EI = \frac{P}{\alpha^2}$$

Substituting in EI (i), we get

$$\frac{d^2y}{dx^2} + \alpha^2 y = \frac{M_0\alpha^2}{P} \quad \ldots(ii)$$

Solution of differential equation (ii) is given as

$$y = A \cos \alpha x + B \sin \alpha x + \frac{M_0}{P}$$

At $x = 0, y = 0$

$$\therefore \quad 0 = A + \frac{M_o}{P} \Rightarrow A = -\frac{M_o}{P}$$

At $x = l, y = 0$ $\quad \therefore \quad 0 = A \cos \alpha l + B \sin \alpha l + \frac{M_0}{P}$

$$\Rightarrow \quad 0 = -\frac{M_0}{P}\cos\alpha l + B\sin\alpha l + \frac{M_0}{P}$$

$$\Rightarrow \quad B = -\frac{M_0(1-\cos\alpha l)}{P\sin\alpha l} = \frac{-M_0}{P}\tan\alpha\frac{l}{2}$$

So,
$$y = \frac{-M_0}{P}\cos\alpha x - \frac{M_0}{P}\tan\alpha\frac{l}{2}\sin\alpha x + \frac{M_0}{P}$$

$$\therefore \quad \frac{dy}{dx} = \frac{M_0}{P}\alpha\sin\alpha x - \frac{M_0}{P}\tan\frac{\alpha l}{2}\alpha\cos\alpha x$$

At $x = 0$, $\frac{dy}{dx} = \theta$

$$\Rightarrow \quad \theta = \frac{-M_0\alpha}{P}\tan\frac{\alpha l}{2}$$

But
$$M_0 = K\theta = \frac{-kM_0\alpha}{P}\tan\frac{\alpha l}{2}$$

$$\Rightarrow \quad \theta = \frac{-K\theta}{P}\alpha\tan\frac{\alpha l}{2}$$

$$\Rightarrow \quad \tan\frac{\alpha l}{2} = \frac{-P}{K\alpha} \quad \text{where} \quad \alpha^2 = \frac{P}{EI}$$

Q.4 (c) Solution:

(i) **1.** Unbraced building without any masonry infill.

As per Cl. 7.6.2 (a) of IS : 1893 (Part - 1) - 2016 (for RC framed building)

Fundamental period of Vibration, T

$$T = 0.075 \text{ H} = 0.075\ (60)^{0.75} = 1.62 \text{ sec.}$$

2. Braced building with infilled brick masonry wall.

Fundamental period of vibration, T is given by:

$$T = \frac{0.09.H}{\sqrt{D}}$$

In short direction, $T = \frac{0.09\times60}{\sqrt{20}} = 1.21$ sec

In long direction, $T = \frac{0.09\times60}{\sqrt{40}} = 0.854$ sec.

(ii) The following information is required for the design of a concrete mix:

1. Grade of concrete to be designed i.e. that is M20, M30 etc.
2. Level of quality assurance, that is standard deviation.
3. Type of cement, that is, ordinary portland cement, portland slag cement, rapid hardening portland cement etc.
4. Type and maximum nominal size of aggregates.
5. Minimum cement content.
6. Maximum water-cement ratio by weight from durability and/or strength considerations.
7. Degree of workability.
8. Exposure condition.
9. Type of admixture, if required.
10. Maximum temperature of fresh concrete.

Section - B

Q.5 (a) Solution:

Moment of inertia of each column,

$$I = \frac{275\times325^3}{12} = 7.87\times10^8 \text{ mm}^4$$

Flexural rigidity, $EI = 2.1\times10^5\times7.87\times10^8 = 1.65\times10^{14}$ N-mm^2

Equivalent lateral stiffness of columns (in parallel) is,

$$k = \frac{2\times12EI}{l_1^3}+\frac{3EI}{l_2^3} = \frac{2\times12\times1.65\times10^{14}}{(4000)^3}+\frac{3\times1.65\times10^{14}}{(3000)^3}$$

$$= 80208.33 \text{ N/mm} = 80208.33\times10^3 \text{ N/m}$$

(i) Natural frequency, $\omega_n = \sqrt{\frac{k}{m}} = \sqrt{\frac{80208.33\times10^3}{\frac{6000\times10^3}{9.81}}} = 11.45$ rad/s

Damped Natural frequency, $\omega_D = \omega_n\sqrt{1-\xi^2}$

where ξ is damping coefficient i.e. 0.15

So, $\omega_D = 11.45\sqrt{1-0.15^2} = 11.32 \text{ rad/s}$

(ii) Amplitude,
$$A = \sqrt{y_0^2 + \left(\frac{V_0 + \omega_n \xi y_0}{w_D}\right)^2}$$
$$= \sqrt{30^2 + \left(\frac{15 + 11.45 \times 0.15 \times 30}{11.32}\right)^2} = 30.57 \text{ mm}$$

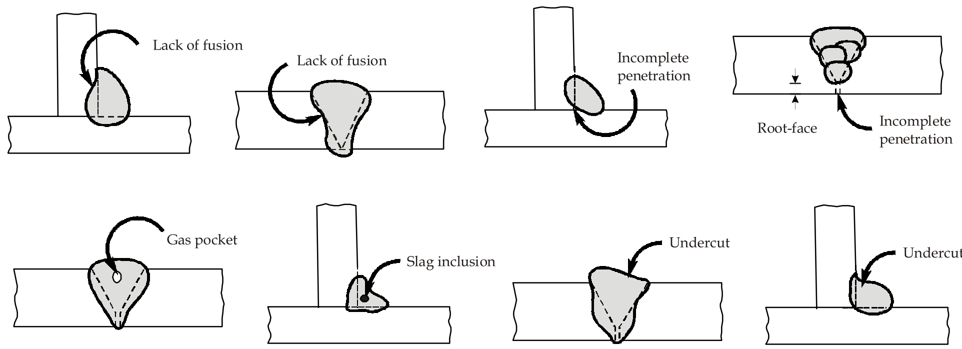
(iii) Displacement,
$$y = e^{-\omega_n \xi t}\left[y_0 \cos\omega_D t + \frac{V_0 + \omega_n \xi y_0}{\omega_D}\sin\omega_D t\right]$$
$$= e^{-11.45\times0.15t}\left[30\cos 11.32t + \frac{15 + 11.45\times0.15\times30}{11.32}\sin 11.32t\right]$$
$$= e^{-1.72t}\left[30\cos 11.32t + 5.877\sin 11.32t\right]$$

Q.5 (b) Solution:

Different types of weld defects are are below:

1. **Incomplete fusion:** It is the failure of the base metal to get completely fused with the weld metal. It is caused by rapid welding and also because of foreign materials on the surfaces to be welded.
2. **Porosity:** Porosity occurs due to voids or gas pockets entrapped in the weld while cooling. It results in stress concentration and reduced ductility of the metal. Normally porosity is not a problem because each void is spherical and not a notch. Even with a slight loss in the section because of the voids, their spherical shape may be considered to allow smooth flow of stress around the void without any measurable loss of strength.
3. **Incomplete penetration:** It is the failure of the weld metal to penetrate the complete depth of joint. It is normally found with single vee and bevel joints and also because of large size electrodes.
4. **Slag inclusions:** These are metallic oxides and other solid compounds which are sometimes found as elongated or globular inclusions. Being lighter than the molten materials these float and rise to the weld surface from where these are removed after cooling of the weld. However, excessive rapid cooling of the weld may cause them to be trapped inside the weld. These pose a problem in vertical and overhead welding.
5. **Cracks:** Cracks are divided as hot and cold. Hot cracks occur due to the presence of sulphur, carbon, silicon and hydrogen in the weld metal. Phosphorous and hydrogen trapped in the hollow spaces of the metal structure give rise to the formation of cold cracks.

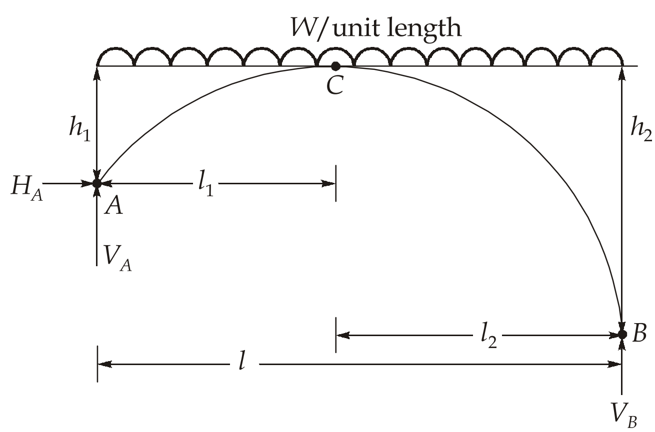
6. **Undercutting:** It is the local decrease of the thickness of the parent metal at the weld toe. This is caused by the use of excessive current or a very long arc. An under cut may result in loss of gross section and acts like a notch.



Weld defects

Q.5 (c) Solution:

Here, l_1 and l_2 are horizontal distances of upper and lower hinges respectively from the crown C.



$$\therefore \qquad l_1 + l_2 = l$$

Taking crown (C) as origin and (x, y) as coordinates of a point on centre line of arch.

$$x^2 = cy$$

$$\Rightarrow \qquad \frac{x}{\sqrt{y}} = c = \text{constant} \qquad \ldots\text{(i)}$$

Using eq. (i) for points A and B

$$\frac{l_1}{\sqrt{h_1}} = \frac{l_1 + l_2}{\sqrt{h_1} + \sqrt{h_2}} = \frac{l}{\sqrt{h_1} + \sqrt{h_2}}$$

So, $$l_1 = \frac{l\sqrt{h_1}}{\left(\sqrt{h_1}+\sqrt{h_2}\right)} \text{ and } l_2 = \frac{l\sqrt{h_2}}{\left(\sqrt{h_1}+\sqrt{h_2}\right)}$$

Now, $$\sum M_C \text{ (from left)} = 0$$

$$\Rightarrow \quad V_A \times l_1 - H \times h_1 - w\frac{l_1^2}{2} = 0$$

$$\Rightarrow \quad V_A = \frac{wl_1}{2} + \frac{Hh_1}{l_1} \quad \ldots\text{(ii)}$$

Also, $$\sum M_C \text{(from right)} = 0$$

$$\Rightarrow \quad V_B = \frac{wl_2}{2} + \frac{Hh_2}{l_2} \quad \ldots\text{(iii)}$$

Adding (ii) and (iii), we get

$$V_A + V_B = \frac{w}{2}(l_1 + l_2) + H\left(\frac{h_1}{l_1} + \frac{h_2}{l_2}\right)$$

$$\Rightarrow \quad wl = \frac{wl}{2} + H\left(\frac{h_1}{l_1} + \frac{h_2}{l_2}\right) \quad [\because l_1 + l_2 = l]$$

$$\Rightarrow \quad \frac{wl}{2} = H\left(\frac{h_1}{l_1} + \frac{h_2}{l_2}\right) \quad \ldots\text{(iv)}$$

Putting values of l_1 and l_2 in (iv), we get

$$H = \frac{\frac{wl}{2}}{h_1\left(\frac{\sqrt{h_1}+\sqrt{h_2}}{l\sqrt{h_1}}\right) + h_2\left(\frac{\sqrt{h_1}+\sqrt{h_2}}{l\sqrt{h_2}}\right)} = \frac{wl^2}{2\left[\sqrt{h_1}\left(\sqrt{h_1}+\sqrt{h_2}\right)+\sqrt{h_2}\left(\sqrt{h_1}+\sqrt{h_2}\right)\right]}$$

$$= \frac{wl^2}{2\left[\sqrt{h_1}+\sqrt{h_2}\right]^2}$$

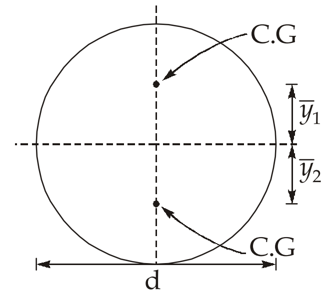
Q.5 (d) Solution:

$$\text{Shape factor} = \frac{M_P}{M_e}$$

where M_P is plastic moment carrying capacity and M_e is elastic moment carrying capacity.

$$\therefore \quad \text{Shape factor} = \frac{f_y.Z_p}{f_y.Z_e} = \frac{Z_p}{Z_e}$$

where Z_p and Z_e are plastic and elastic section modulli respectively.



(i) For circular section

$$Z_P = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

Now, $$A = \frac{\pi d^2}{4}, \bar{y}_1 = \frac{2d}{3\pi}$$

$\therefore$ $$Z_P = \frac{\pi d^2}{2\times 4}\times\left(\frac{2d}{3\pi}+\frac{2d}{3\pi}\right) = \frac{d^3}{6}$$

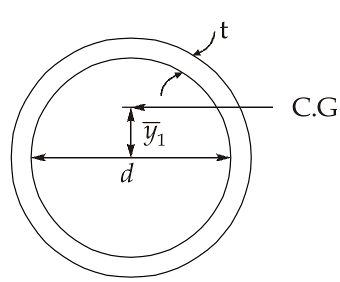
Also, $$Z_e = \frac{I_{xx}}{y} = \frac{\pi\frac{d^4}{64}}{\frac{d}{2}} = \frac{\pi d^3}{32}$$

$\therefore$ Shape factor, $$\frac{Z_P}{Z_e} = \frac{\frac{d^3}{6}}{\pi\frac{d^3}{32}} = \frac{16}{3\pi} \simeq 1.7$$

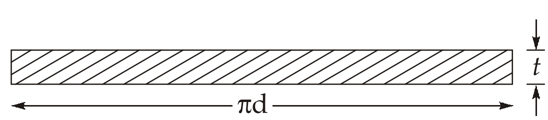
(ii) For thin hollow circular section

Let the thickness of the hollow circular section be 't' and diameter be 'd'.

For thin sections, t^2 and t^3 terms are neglected in calculations. The area is treated as line area.

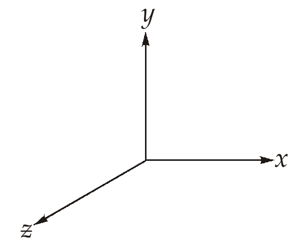


Let's idealize thin area as line area



Now, $$Z_P = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

where, $A = \pi dt, \quad \bar{y}_1 = \bar{y}_2 = \dfrac{d}{\pi}$



$\therefore \quad Z_P = \dfrac{\pi dt}{2}\left(\dfrac{d}{\pi}+\dfrac{d}{\pi}\right) = d^2 t$

Also, $Z_e = \dfrac{I_{xx}}{y}$

To find I_{xx} let's find out I_{zz} first

$$I_{zz} = \int r^2 dA = r^2.A = \pi dt \times \left(\frac{d}{2}\right)^2$$

Now $I_{xx} + I_{yy} = I_{zz}$

But $I_{xx} = I_{yy}$

$\therefore \quad I_{xx} = \dfrac{I_{zz}}{2} = \dfrac{\pi d^3 t}{8}$

$\therefore \quad \text{Shape factor} = \dfrac{Z_P}{Z_e} = \dfrac{d^2 t}{\dfrac{\pi d^3 t \times 2}{8 \times d}} = \dfrac{4}{\pi} = 1.27$

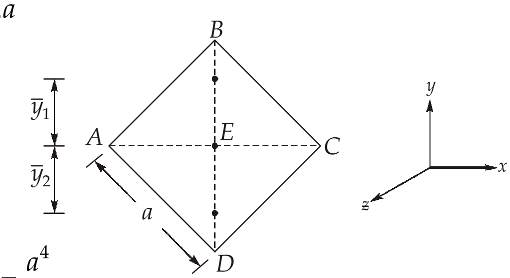
(iii) Square section

Let the side of square be 'a'

$$AC = \sqrt{a^2 + a^2} = \sqrt{2}a$$

$$BE = \frac{a}{\sqrt{2}}$$

Moment of inertia about x-axis



$$I_{xx} = \frac{2\sqrt{2}}{12}a\left(\frac{a}{\sqrt{2}}\right)^3 = \frac{a^4}{12}$$

Elastic section modulus, $Z_{ex} = \dfrac{a^4/12}{a/\sqrt{2}} = \dfrac{a^3}{6\sqrt{2}}$

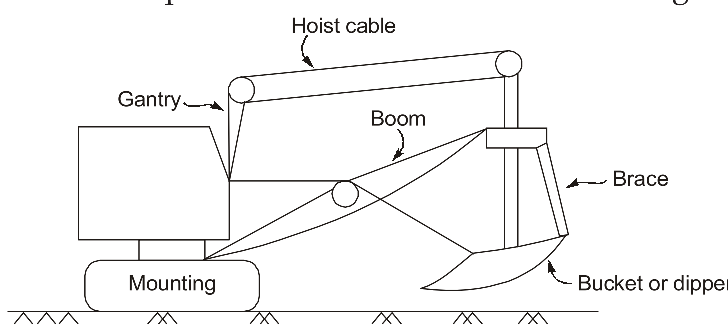
Plastic section modulus, $Z_{px} = \dfrac{A}{2}(\bar{y}_1 + \bar{y}_2) = \dfrac{a^2}{2} \times \left(\dfrac{a}{3\sqrt{2}} + \dfrac{a}{3\sqrt{2}}\right) = \dfrac{a^3}{3\sqrt{2}}$

$$\text{Shape factor, } S = \frac{Z_{px}}{Z_{ex}} = \frac{a^3/3\sqrt{2}}{a^3/6\sqrt{2}} = 2$$

Q.5 (e) Solution:

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

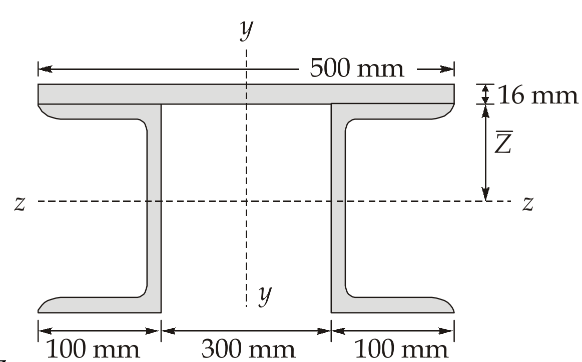


The machine is placed in operation by setting the boom at the desired angle. The dipper is moved to the desired position. The free end of the boom is lowered down by releasing the tension in the hoist cable until the dipper teeth engages the material to be dug. As the cable is pulled in, the dipper is 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, 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, these are not production excavation machines. They are designed for mobility and general purpose work.

Q.6 (a) Solution:



(i) For Fe 410 grade of steel

$f_u = 410 \text{ N/mm}^2, f_y = 250 \text{ N/mm}^2$

$l_{eff} = 5.5 \text{ m}$

Let NA be at a distance of $\overline{Z}$ from top

Now,

$$\overline{Z} = \frac{A_1 Z_1 + A_2 Z_2}{A_1 + A_2}$$

$$= \frac{500 \times 16 \times 8 + 2 \times 6293 \times 216}{500 \times 16 + 2 \times 6293} = 135.17 \text{ mm}$$

$$I_{ZZ,\text{ combined}} = 2 \times 15082.8 \times 10^4 + 500 \times \frac{16^3}{12} + 2 \times 6293 \times (216 - 135.17)^2 + 16 \times 500 \times (135.17 - 8)^2$$

$$= 513434829.2 \text{ mm}^4$$

$$I_{yy,\text{ combined}} = 2 \times 504.8 \times 10^4 + 2 \times 6293 \times (150 + 24.2)^2 + \frac{500^3}{12} \times 16$$

$$= 558692891.7 \text{ mm}^4$$

Now, $I_{\min} = I_{zz,\text{ combined}} = 513434829.2 \text{ mm}^4$

Minimum radius of gyration, $r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{513434829.2}{2 \times 6293 + 500 \times 16}} = 157.93 \text{ m}$

$\therefore$ Slenderness ratio, $\lambda = \frac{kL}{r_{\min}} = \frac{5.5 \times 1000}{157.93} = 34.826$

From table, $\frac{40 - 30}{198 - 211} = \frac{34.826 - 30}{f_{cd} - 211}$

$\Rightarrow$ $f_{cd} = 204.7262 \text{ N/mm}^2$

$\therefore$ Design compressive load $= \frac{204.7262 \times (2 \times 6293 + 500 \times 16)}{1000} \text{ kN} = 4214.49 \text{ kN}$

(ii) The yielding of the cross-section of tension member causes excessive elongation and hence the load corresponding to the yielding of gross section is taken as one limit state.

However the net section through the bolt holes at the ends of the members may be subjected to tensile stresses well in excess of the yield stress to as high as ultimate stress without the member suffering excessive elongation. Hence, the rupture strength of the net section through the bolt holes at the ends is considered another limit state.

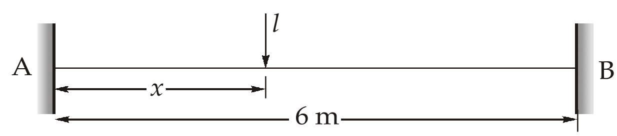
Several factors affect the rupture strength of the net section of tension members and are listed below:

1. **Effect of bolt holes:** In order to make connections, tension members are often bolted to adjacent membes directly or by using gusset plate. These bolt holes often reduce the area of cross-section available to carry the tension and hence affects the strength.
2. **Method of fabrication:** There are generally two methods of making holes to receive bolts, namely punching and drilling. Due to punching, the materials around the holes is deformed in shear beyond ultimate strength to punch out the holes. It has been found that the strength of members with punched holes may be 10-15% less than the members with drilled holes.

3. **Net area of cross-section:** The presence of a hole tends to reduce the strength of a tension member. When more than one bolt hole is present, the failure paths may occur along sections normal to the axis of the members, or they may include zig-zag sections, if the fasteners are staggered. Staggering holes improves the load carrying capacity of the member for a given row of bolts.
4. **Effect of shear lag:** The force is transferred to a tension member by a gusset or the adjacent member connected to one of the legs either by bolting or welding. The force thus transferred to one leg by the end connection locally gets transferred as tensile stress over the entire cross section by shear. Hence, the tensile stress on the section from the first bolt hole up to the last bolt hole will not be uniform. The connected leg will have higher stresses at failure even of the order of ultimate stress while the outstanding leg stresses may be even below yield stress. However, at sections away from the end connection, the stress distribution becomes more uniform.
5. **Geometry factor:** Tests on bolted joints show that the net section is more efficient if the ratio of the guage length '*g*' to the diameter '*d*' is small. The increase in efficiency due to a small g/d ratio is due to suppression of contraction at the net section.

Q.6 (b) Solution:

Consider a unit load at x distance from left support as shown below.



Now, fixed end moment at A,

$$M_A = \frac{-1\times x\times(6-x)^2}{6^2}\ \text{m} \quad \ldots\text{(i)}$$

$$M_B = \frac{1\times x^2\times(6-x)}{6^2}\ \text{m} \quad \ldots\text{(ii)}$$

Now, $\sum F_y = 0$

$\Rightarrow$ $V_A + V_B - 1 = 0$

$\Rightarrow$ $V_A + V_B = 1$

$\sum M_B = 0$

$\Rightarrow$ $V_A \times 6 - 1 \times (6 - x) + M_A + M_B = 0$

$$\Rightarrow \quad V_A \times 6 - (6-x) - \frac{x(6-x)^2}{36} + \frac{x^2(6-x)}{36} = 0$$

$$\Rightarrow \quad V_A = \frac{(6-x)}{6} + \frac{x(6-x)^2}{36\times 6} - \frac{x^2(6-x)}{36\times 6} = \frac{(6-x)}{6} + \frac{(36+x^2-12x)x}{36\times 6} - \frac{(6x^2-x^3)}{36\times 6}$$

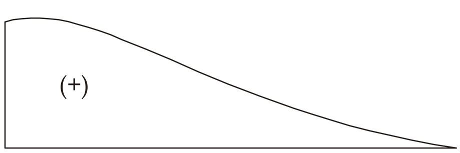
$$= \frac{216-36x+36x+x^3-12x^2-6x^2+x^3}{216} = \frac{2x^3-18x^2+216}{216} = \frac{x^3-9x^2+108}{108} \quad \ldots\text{(iii)}$$

Also,
$$V_B = 1-\left(\frac{x^3-9x^2+108}{108}\right) = \frac{-x^3+9x^2}{108} \quad \ldots\text{(iv)}$$

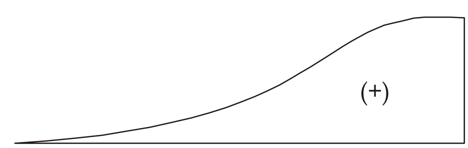
By substituting different values of x at 2 m interval, we can obtain the ILD and calculations are tabulated below.

$x(\text{m})$	$V_A = \frac{x^3-9x^2+108}{108}$	$V_B = \frac{-x^3+9x^2}{108}$	$M_A = \frac{-x(6-x)^2}{36}$	$M_B = \frac{x^2(6-x)}{36}$
0	1	0	0	0
2	0.741	0.259	–0.889	0.444
4	0.259	0.741	–0.444	0.889
6	0	1	0	0

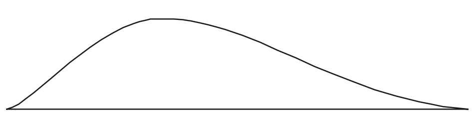
(i) ILD for V_A



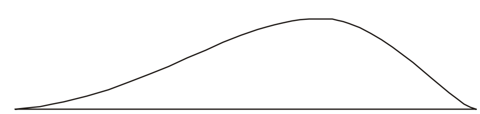
(ii) ILD for V_B



(iii) ILD for M_A

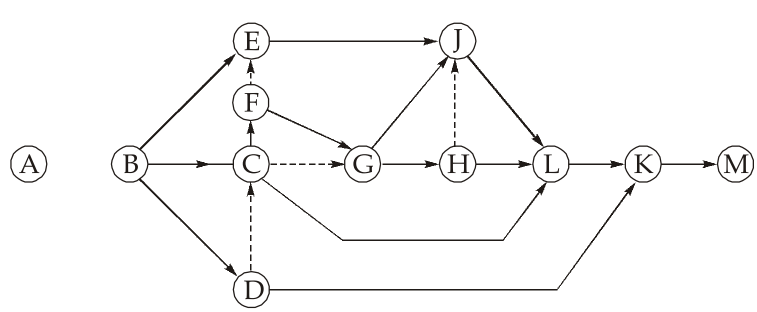


(iv) ILD for V_A



Q.6 (c) Solution:

(i)



(ii) Cycle time for a dump turck:

- Loading time = 10 minutes
- Hauling time @ 20 km/hr = $\frac{4 \times 60}{20} = 12$ minutes
- Returning time @ 30 km/hr = $\frac{4 \times 60}{30} = 8$ minutes
- Other fixed time = 4 minutes

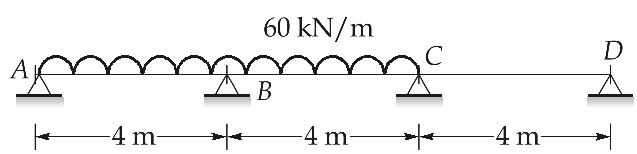
∴ Total time = 10 + 12 + 8 + 4 = 34 minutes.

- Actual working period is considered as 50 minutes per hour and 5 hours working period in a day.
- Material transported per hour = $\frac{50}{34} \times 20 = 29.41$ cum.
- Material transported per day = 29.41 × 5 = 147.05 cum.

∴ No. of dump trucks required per day = $\frac{1000}{147.05} = 6.8 \simeq 7$ (say)

∴ 7 dump trucks are required.

Q.7 (a) Solution:



1. Fixed end moment:

$$M_{FAB} = \frac{-wL^2}{12} = \frac{-60 \times 4^2}{12} = -80 \text{ kN-m}$$

$$M_{FAB} = \frac{wL^2}{12} = \frac{60 \times 4^2}{12} = 80 \text{ kN-m}$$

$$M_{FBC} = \frac{-wL^2}{12} = \frac{-60 \times 4^2}{12} = -80 \text{ kN-m}$$

$$M_{FCB} = \frac{wL^2}{12} = \frac{60 \times 4^2}{12} = 80 \text{ kN-m}$$

$$M_{FCB} = M_{FDC} = 0$$

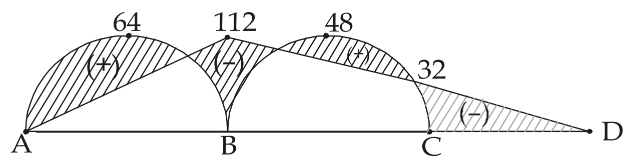
2. Distribution factors:

Joint	Member	Stiffness	Total Stiffness	Distribution factors
B	BA	$\frac{3EI}{4}$	$\frac{7EI}{4}$	$\frac{3}{7}$
	BC	$\frac{4EI}{4}$		$\frac{4}{7}$
C	CB	$\frac{4EI}{4}$	$\frac{7EI}{4}$	$\frac{4}{7}$
	CD	$\frac{3EI}{4}$		$\frac{3}{7}$

3. Moment distribution table:

Joint	A	B		C		D
D.F.		**3/7**	**4/7**	**4/7**	**3/7**	
F.E.M.	**–80**	**80**	**–80**	**80**	**0**	**0**
End correction	**80** →	**40**				
Corrected F.E.M.	0	120	–80	80	0	0
Balancing Moment		–17.14	–22.86	–45.72	–34.28	0
C.O.M.			–22.86	–11.43		
Balancing Moment		+9.80	+13.06	6.53	4.89	
C.O.M.			3.26	6.53		
B.M.		–1.40	–1.86	–3.73	–2.8	
C.O.M.			–1.86	–0.93		
B.M.		0.8	1.06	0.53	0.4	
C.O.M.			0.26	0.53		
B.M.		–0.11	–0.15	–0.30	–0.23	
C.O.M.			–0.15	–0.07		
B.M.		0.06	0.09	0.04	0.03	
C.O.M.			0.02	0.045		
	0	**112**	**–112**	**32**	**–32**	**0**

4. BMD



(Values in kN-m)

(ii)

Maximum bending moment,

$$M = 112 \text{ kN-m}$$

Moment of inertia, $I = \dfrac{100 \times 300^3}{12} = 2.25 \times 10^8 \text{ mm}^4$

Maximum bending stress,

$$\sigma_{max} = \frac{M}{I} \times y = \frac{112 \times 10^6}{2.25 \times 10^8} \times 150 = 74.67 \text{ N/mm}^2$$

So, maximum tensile and compressive stresses will be at support B at top and bottom fibre, respectively and will have value of 74.67 N/mm^2.

Q.7 (b) Solution:

Size of the weld, $S = 6 \text{ mm}$

Throat thickness, $t_t = kS = 0.7 \times 6 = 4.2 \text{ mm}$

Grade of steel is Fe410

$\therefore$ $f_u = 410 \text{ N/mm}^2, fy = 250 \text{ N/mm}^2$

Let's find distance of C.G. of the weld section from vertical weld.

$$\bar{x} = \frac{4.2 \times 300 \times 0 + 4.2 \times 170 \times \frac{170}{2} \times 2}{4.2 \times 300 + 4.2 \times 170 \times 2} = 45.156 \text{ mm} \simeq 45.16$$

$$\bar{y} = 150 \text{ mm}$$

$\therefore$ Total torsional moment, $T = P \cos 30° \times (300 + 170 - 45.16) - P \sin 30° \times 150$

$= P \cos 30° \times 424.84 - P \sin 30 \times 150 = 292.92\, P$

Let's find polar moment of inertia of weld group.

$$I_p = I_{xx} + I_{yy}$$

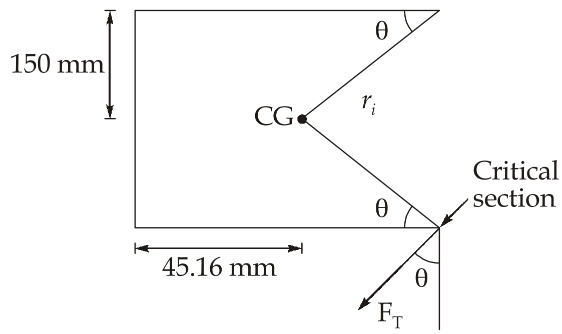
$$I_{xx} = 4.2 \times \frac{300^3}{12} + 2 \times 170 \times 4.2 \times (150)^2 = 41580000 \text{ mm}^4$$

$$= 4158 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 2\times\frac{170^3}{12}\times 4.2 + 2\times 170\times(85-45.16)^2\times 4.2 + 300\times 4.2\times 45.16^2$$
$$= 8275334.413 \text{ mm}^4$$
$$= 827.53 \times 10^4 \text{ mm}^4$$

$\therefore$ $$I_p = I_{xx} + I_{yy} = (4158 + 827.53)\times 10^4 = 4985.53\times 10^4 \text{ mm}^4$$

Torsional shear stress on the weld $f_{s1} = \dfrac{292.92P\times r_i}{4985.53\times 10^4}\times 10^3$



Now, $$r_i = \sqrt{(170-45.16)^2+150^2} = 195.15 \text{ mm}$$

$$\frac{T}{I_p}\times r_i = \frac{292.92P\times 195.15}{4985.53\times 10^4} = 1.15\times 10^{-3}P \text{ N/mm}^2$$

$$\theta = \tan^{-1}\frac{150}{(170-45.16)} = 50.23^\circ$$

$$f_{s1} = 1.15\times 10^{-3}P \text{ N/mm}^2$$

Direct shear stress on the weld

$$f_{sx} = \frac{P\times\sin 30^\circ}{(2\times 170+300)4.2} = \left(1.86\times 10^{-4}\right)P \text{ N/mm}^2$$

$$f_{sy} = \frac{P\times\cos 30^\circ}{(2\times 170+300)4.2} = \left(3.22\times 10^{-4}\right)P \text{ N/mm}^2$$

Net stress in x direction $= f_{sx} + f_{s1}\sin\theta = (1.86\times 10^{-4})P + (1.15\times 10^{-3}\times\sin 50.23^\circ)P$

$= 1.07\times 10^{-3}$ P

Net stress in y direction $= f_{sy} + f_{s1}\cos\theta$

$= (3.22\times 10^{-4})P + (1.15\times 10^{-3})P\times\cos 50.23^\circ$

$= 1.06\times 10^{-3}P$ N/mm²

Net stress at critical section $= P\left(\sqrt{(1.07)^2+(1.06)^2}\right)\times 10^{-3}$

$= 1.57\times 10^{-3}P$ N/mm²

For safety, $(1.51 \times 10^{-3})P \leq \dfrac{410}{\sqrt{3} \times 1.25}$

$\Rightarrow \quad P \leq 125.411 \times 10^3$ N

$\Rightarrow \quad \leq 125$ kN

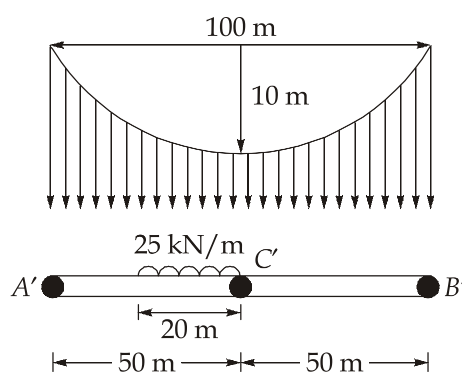
Q.7 (c) Solution:

(i)

Analysis for live load:

Live load on girder will be transferred to cables in the form of uniformly distributed load. Let the intensity of uniformly distributed load acting upwards on girder be w_l kN/m.

Let, the reactions developed at A' and B' due to w_l and live load be V'_A and V'_B respectively



Now, $\Sigma M'_B = 0$

$\Rightarrow \quad V'_A \times 100 + w_l \times 100 \times 50 - 25 \times 20 \times 60 = 0$

$\Rightarrow \quad 100V'_A + 5000w_l - 30000 = 0$ ...(i)

Also, $\Sigma M'_C = 0$

$\Rightarrow \quad V'_A \times 50 + w_l \times 50 \times 25 - 25 \times 20 \times 10 = 0$

$\Rightarrow \quad 50V'_A + 1250w_l - 5000 = 0$ ...(ii)

Solving (i) and (ii), we get

$V'_A = -100$ kN

$w_l = 8$ kN/m

Also, $V'_A + V'_B + w_l \times 100 = 25 \times 20$

$\Rightarrow \quad -100 + V'_B + 8 \times 100 = 500$

$\Rightarrow \quad V'_B = -200$ kN

This uniformly distributed load 'w_l' is acting downwards on cable throughout its length.

So, vertical reaction at A, $$V_{A1} = \frac{w_l l}{2} = \frac{8 \times 100}{2} = 400 \text{ kN}$$

Horizontal reaction at A, $$H_{A1} = \frac{w_l l^2}{8h} = \frac{8 \times 100^2}{8 \times 10} = 1000 \text{ kN}$$

Analysis for dead load:

As the dead load of 15 kN/m is acting on whole span of girder, it will be transferred as uniformly distributed load of 15 kN/m to cable,

So, vertical reaction at A, $$V_{A2} = \frac{w_d l}{2} = \frac{15 \times 100}{2} = 750 \text{ kN}$$

(where w_d is uniformly distributed load dead transferred to cable due to dead load on girder)

Horizontal reaction at A, $$H_{A2} = \frac{w_d l^2}{8h} = \frac{15 \times 100^2}{8 \times 10} = 1875 \text{ kN}$$

Therefore, total vertical reaction at A, $V_A = V_{A1} + V_{A2}$
$$= 400 + 750 = 1150 \text{ kN}$$

Total horizontal reaction at A, $H_A = H_{A1} + H_{A2}$
$$= 1000 + 1875 = 2875 \text{ kN}$$

So, maximum tension in cable $= \sqrt{H_A^2 + V_A^2} = \sqrt{1150^2 + 2875^2} = 3096.47 \text{ kN}$

(ii)

Length of wire, $L = l + \frac{8}{3}\frac{h^2}{l}$ (where h is central dip)

$$= 30 + \frac{8}{3} \times \frac{(0.5)^2}{30} = 30.022 \text{ m}$$

So, Weight of cable, $W = 30.022 \times \frac{\pi}{4} \times (0.02)^2 \times 78.5 = 0.74 \text{ kN}$

Now, Horizontal thrust, $H = \frac{Wl}{8h} = \frac{0.74 \times 30}{8 \times 0.5} = 5.55 \text{ kN}$

$$\text{Vertical reaction, } V = \frac{W}{2} = \frac{0.74}{2} = 0.37 \text{ kN}$$

So,
$$T_{max} = \sqrt{H^2 + V^2} = \sqrt{5.55^2 + 0.37^2} = 5.56 \text{ kN}$$

Hence,
$$\text{maximum stress in wire} = \frac{5.56}{\frac{\pi}{4} \times (0.02)^2} = 17698.03 \text{ kN/m}^2 \simeq 17.7 \text{ N/mm}^2$$

Now,
$$\delta h = \frac{3}{16}\frac{l}{y_c}\delta L = \frac{3}{16}\frac{l^2 \alpha \Delta t}{h} \qquad [\because \delta l \approx l\alpha\Delta T]$$

$$= \frac{3}{16} \times \frac{30^2}{0.5} \times 1.2 \times 10^{-5} \times \Delta t = 4.05 \times 10^{-3}\, \Delta t$$

Also,
$$\frac{\delta f}{f} = \frac{-\delta h}{h}$$

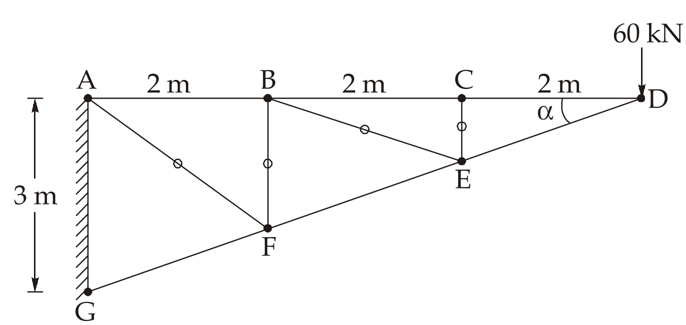
$$\Rightarrow \quad \left(\frac{20 - 17.7}{17.7}\right) = \frac{4.05 \times 10^{-3} \times \Delta t}{0.5}$$

$$\Rightarrow \quad \Delta t = 16.04° \text{ C}$$

Hence, the fall in temperature required to raise the stress in wire to 20 N/mm^2 is 16.04° C.

Q.8 (a) Solution:

(i) Vertical deflection of *D*



- ***P* system of forces**

$$\tan \alpha = \frac{3}{2+2+2} = 0.5$$

$$\therefore \qquad \sin\alpha = \frac{1}{\sqrt{5}} \text{ and } \cos\alpha = \frac{2}{\sqrt{5}}$$

Joint D:

Resolving vertically

$$P_{DE}\sin\alpha = 60$$

$$\Rightarrow \qquad P_{DE} = 60\sqrt{5}\text{ kN (Compressive)}$$

Resolving forces horizontally

$$P_{DE}\cos\alpha = P_{DC}$$

$$\Rightarrow \qquad P_{DC} = 60\sqrt{5}\times\frac{2}{\sqrt{5}} = 120\text{ kN (Tensile)}$$

Joint C:

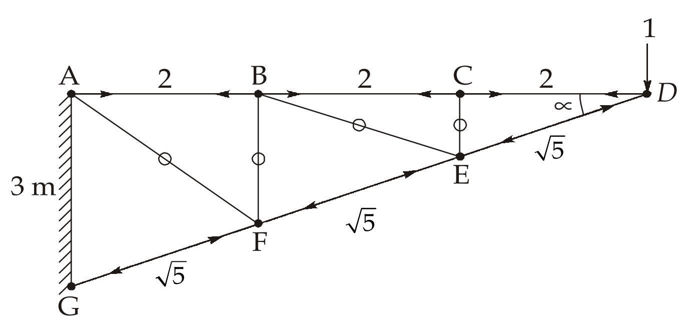
$P_{CE} = 0,$ $\qquad P_{CB} = P_{DC} = 120$ kN (Tensile)

$P_{BF} = 0,$ $\qquad P_{BA} = P_{CB} = 120$ kN (Tensile)

$P_{BE} = 0,$ $\qquad P_{EF} = 60\sqrt{5}$ kN (Compressive)

$P_{FA} = 0,$ $\qquad P_{GF} = 60\sqrt{5}$ kN (Compressive)

- **K system of forces:**



Now, the given load is removed and a unit load is applied at *D* in vertical direction at *D*.

The forces in members due to unit load will be $\frac{1}{60}$ of forces produced due to 60 kN at *D* as shown in figure.

Vertical deflection at *D*,

$$\Delta_{VD} = \frac{\sum PKL}{AE}$$

Calculations are tabulated below:

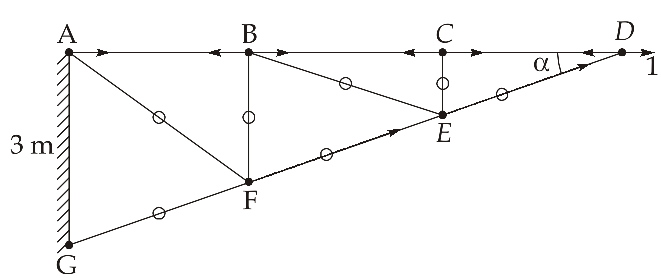
Member	P (kN)	K	L (mm)	$A\left(mm^2\right)$	$\frac{PkL}{A}$ (kN/mm)
AB	120	2	2000	2000	240
BC	120	2	2000	2000	240
CD	120	2	2000	2000	240
DE	$-60\sqrt{5}$	$-\sqrt{5}$	$\sqrt{5}\times 1000$	3000	223.6
EF	$-60\sqrt{5}$	$-\sqrt{5}$	$\sqrt{5}\times 1000$	3000	223.6
FG	$-60\sqrt{5}$	$-\sqrt{5}$	$\sqrt{5}\times 1000$	3000	223.6
AF	0	0	$2\sqrt{2}\times 1000$	1000	0
BF	0	0	2000	1000	0
BE	0	0	$\sqrt{5}\times 1000$	1000	0
CE	0	0	1000	1000	0

$$\sum = 1390.8$$

Now, $E = 200 \text{ kN/mm}^2$

So, $\Delta_{VD} = \frac{1390.8}{200} = 6.954 \text{ mm}$

(ii) Horizontal deflection of *D*



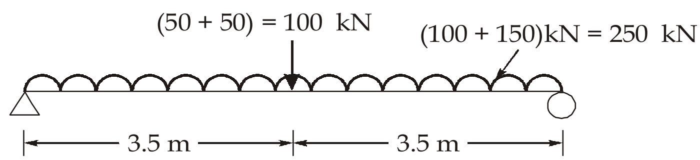
Apply a unit load in horizontal direction as shown in figure. Now, forces will be zero in all members except CD, BC and BA

$$K_{BA} = K_{BC} = K_{CD} = 1 \text{ (Tensile)}$$

So, horizontal deflection at D,

$$\Delta_{HD} = \frac{\sum PKL}{AE} = \frac{1}{200}\times 3\times\left[\frac{120\times 1\times 2000}{2000}\right] = 1.8 \text{ mm}$$

Q.8 (b) Solution:



From figure: $wL = 250$ kN

$\Rightarrow$ $$w = \frac{250}{7} \text{ kN/m}$$

$$\text{Factored UDL, } w_u = \frac{250}{7} \times 1.5 = 53.571 \text{ kN/m}$$

$$\text{Factored point load , } P_u = 100 \times 1.5 = 150 \text{ kN}$$

$$\text{Maximum shear force} = \left(75 + \frac{53.571 \times 7}{2}\right) = 262.5 \text{ kN}$$

$$\text{Maximum bending moment, } M_u = \frac{w_u l^2}{8} + \frac{P_u l}{4} = \frac{53.571 \times 7^2}{8} + \frac{150 \times 7}{4} = 590.6 \text{ kN/m}$$

$$Z_{P,\text{ required}} = \frac{M_u}{f_y / \gamma_{mo}} = \frac{590.6 \times 10^6}{\left(\frac{250}{1.1}\right)} = 2598640 \text{ mm}^3$$

For ISMB 500

$$Z_P = \text{Shape factor} \times Z_{ez}$$
$$= 1.12 \times 2091.6 \times 10^3 = 2342.592 \times 10^3 < Z_{P,\text{ required}}$$

Hence not safe.

For ISMB 550

$$Z_{PZ} = 1.149 \times 2359.8 \times 10^3 = 2711.41 \times 10^3 \text{ mm}^3 > Z_{PZ,\text{ required}}$$

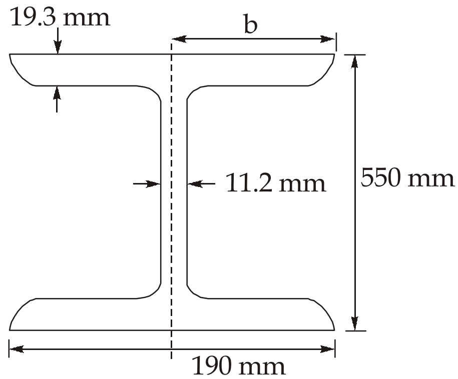
Let us adopt ISMB 550

Let's classify the section

$$\text{Outstand of the flange, } = \frac{b}{t_f} = \frac{190/2}{19.3} = 4.92 < 9.4\varepsilon \qquad \left[\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1\right]$$

$\therefore$ Flange is plastic

Check for web



Radius at root, R_1 = 18 mm

$$d = h - 2(R_1 + t_t) = 550 - 2\,(18 + 19.3) = 475.4 \text{ mm}$$

$$\frac{d}{t_w} = \frac{475.4 \text{ mm}}{11.2 \text{ mn}} = 42.45 < 84\varepsilon$$

∴ Web is also plastic.

Also, $\frac{d}{t_w} < 67\varepsilon$

∴ Shear buckling check of web is not required.

Check for shear

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} \times h t_w = \frac{250}{\sqrt{3} \times 1.1} \times 550 \times 11.2 \times 10^{-3} \text{ kN}$$

$$= 808.29 \text{ kN}$$

∴ $V = 262.5 \text{ kN} < V_d$ (safe in shear) (OK)

$$0.6\, V_d = 0.6 \times 808.29 = 484.97 \text{ kN}$$

∴ $V < 0.6\, V_d$

Hence, the section belongs to low shear case.

Check for bending

$$M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_{mo}} \le 1.2 Z_{ez} \frac{f_y}{\gamma_{mo}}$$

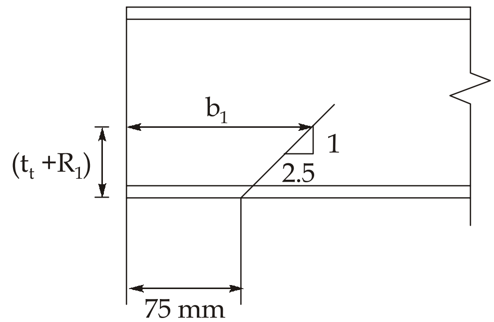
$$= 1 \times 2711.41 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} \le 1.2 \times 2359.8 \times 10^3 \times \frac{250}{1.1} \times 10^{-6}$$

= 616.23 kN-m < 643.58 kN (OK)

$$M_d = 616.23 \text{ kN}$$

$$\text{B.M}_{max} = 590.62 \text{ kN-m} < M_d$$

Check for web crippling



$$\text{Bearing length, } b_1 = 75 \text{ mm} + 2.5 \times (t_f + R_1) \text{ mm}$$
$$= 75 \text{ mm} + 2.5 \times (19.3 + 18) \text{ mm} = 168.25 \text{ mm}$$
$$R = 262.5 \text{ kN}$$
$$F_w = \frac{f_y}{\gamma_{m0}} \times b_1 t_w = \frac{250}{1.1} \times 168.25 \times 11.2 \times 10^{-3}$$
$$= 428.27 \text{ kN} > 262.5 \text{ kN} \quad \text{(ok)}$$

∴ Safe in crippling

Check for deflection

$$\delta_{max} = \frac{5}{384} \times \frac{wL^4}{EI} + \frac{PL^3}{48EI} = \frac{5}{384} \times \frac{35.714 \times 7^4 \times 10^{12}}{2 \times 10^5 \times \left(2359.8 \times 10^3 \times \frac{550}{2}\right)} + \frac{100 \times 10^3 \times 7^3 \times (10^3)^3}{48 \times 2 \times 10^5 \times \left(2359.8 \times 10^3 \times \frac{550}{2}\right)}$$
$$= 8.6 \text{ mm} + 5.51 \text{ mm} = 14.1 \text{ mm}$$
$$\frac{\text{Span}}{300} = \frac{7000}{300} = 23.33 \text{ mm}$$
$$\therefore \quad \delta_{max} < \frac{\text{Span}}{300}$$

Hence beam is safe in deflection.

Q.8 (c) Solution:

(i) Safety measures for scaffolding, ladders framework and other equipments:

1. All scaffolds and working platforms should be securely fastened to the building or structure. If independent of a building, they should be braced or guyed properly.
2. All wooden ladders or bamboo ladders must be strong enough to carry the anticipated load.
3. Ladders in heavy duty work should not exceed 6 m in length, for light work it should not exceed 8 m in length.
4. Dismantling of scaffold should be in a proper sequence.

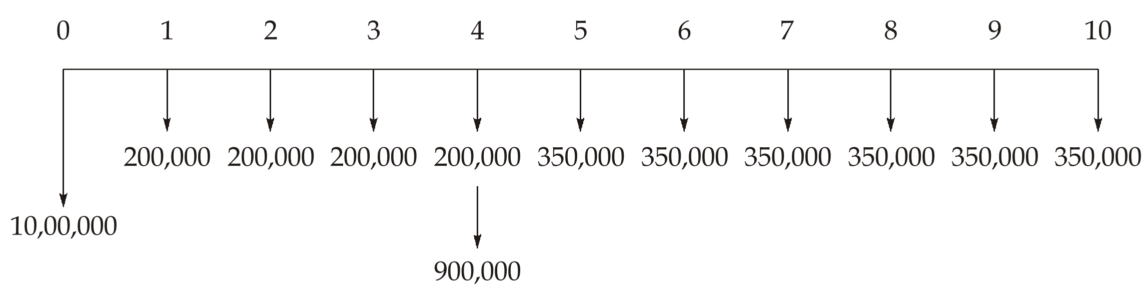
5. No un-insulated electric wires should exist within 3 m of working platform.
6. In case, scaffolds are to be kept stairway, wide enough to allow two persons to pass, should be erected with handrails on both sides.

Fire safety in building:

1. The early warning system for fire alarm must be provided in the buildings wihich may be :
   (a) Smoke detector for air-conditioned areas.
   (b) Heat sensitive detector for non-air conditioned areas.
2. The building must have a separate staircase at every 30 m distance in addition to lifts and emergency fire escapes.
3. The design of different structural elements should take into account fire resistance ratings.
4. Fire safety of electrical installations should be provided as per IS 1664-1982.
5. For non-electrical installations, references may be taken from IS 1647-1960.
6. Chimney, flues and smoke pipes should be provided as per IS 1645-1960.

(ii)

The cash flow diagram of P plan (A) is shown below and its present worth P_A on the basis of 10 years of life is obtained.



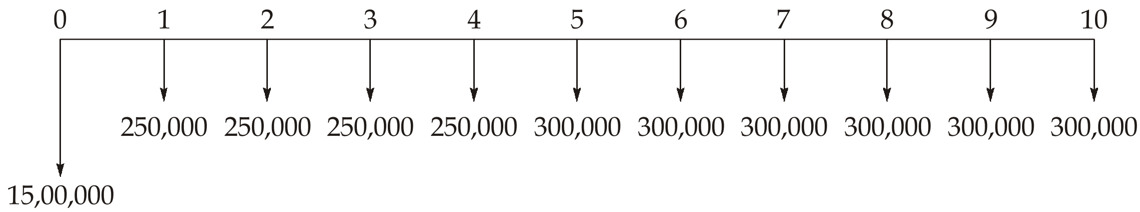
$$P_A = 1000000 + 200,000\left[\frac{P}{A}, 12\%, 4\right] + 900,000\left[\frac{P}{F}, 12\%, 4\right] + 350,000\left[\frac{P}{A}, 12\%, 6\right]\left[\frac{P}{F}, 12\%, 4\right]$$

$$= 1000,000 + 200,000\left[\frac{(1+0.12)^4 - 1}{(0.12)(1+0.12)^4}\right] + 900,000\left[\frac{1}{(1+0.12)^4}\right] + 350,000\left[\frac{(1+0.12)^6 - 1}{(0.12)(1+0.12)^6}\right]\left[\frac{1}{(1+0.12)^4}\right]$$

= 1000,000 + 607469.87 + 571966.2706 + 914505.786

$\therefore \quad P_A = 3093941.929 \simeq 30, 93, 42$

The cash flow diagram of Plan (B) is shown in figure below and its present worth P_B is obtained.



$$P_B = 1500000 + 250,000\left[\frac{P}{A}, 12\%, 4\right] + 300,000\left[\frac{P}{A}, 12\%, 6\right]\left[\frac{P}{F}, 12\%, 4\right]$$

$$= 1500,000 + 250,000\left[\frac{(1+0.12)^4 - 1}{(0.12)(1+0.12)^4}\right] + 300,000\left[\frac{(1+0.12)^6 - 1}{(0.12)(1+0.12)^6}\right]\left[\frac{1}{(1+0.12)^4}\right]$$

$$= 1500,000 + 759337.3367 + 783862.1045$$

$\therefore$ $P_B = 3043199.441 \simeq 30, 43, 200$

Since, $P_B(10) < P_A(10)$

So, Plan (B) is economical.

■■■■