ACE Engineering Academy
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ESE – 2019 MAINS OFFLINE TEST SERIES

CIVIL ENGINEERING

TEST – 13 SOLUTIONS

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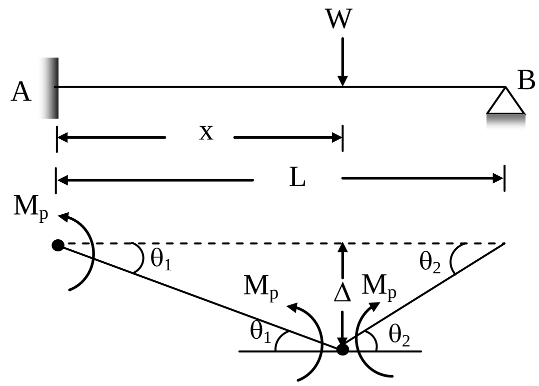
01(a).

Sol: Since end A of the beam is fixed, a plastic hinge will develop at A. A plastic hinge will also be formed below the concentrated load and a mechanism is formed as shown in figure.

$\Delta = x\,\theta_1 = (L - x)\,\theta_2$



or $\theta_1 = \dfrac{L-x}{x}\theta_2$

External work done = load × deflection

$= W_u \times x \times \theta_1$

` Internal work done = moment × rotation

$= M_p\,\theta_1 + M_p\,(\theta_1 + \theta_2)$

$= 2M_p \times \left(\dfrac{L-x}{x}\right)\theta_2 + M_p\theta_2$

$= M_p\theta_2\left(2\left(\dfrac{L-x}{x}\right)+1\right)$

$= M_p\left(\dfrac{2L-2x+x}{x}\right) = M_p\left(\dfrac{2L-x}{x}\right)$

By the principle of virtual work,

External work done = Internal work done

$$W_u \times x \times \left(\dfrac{L-x}{x}\right)\theta_2 = M_p\left(\dfrac{2L-x}{x}\right)$$

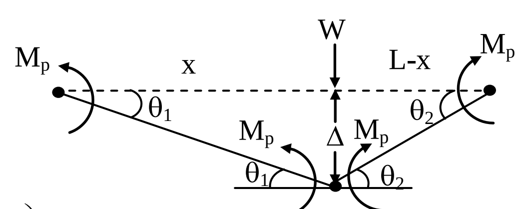
$$W_u = \dfrac{2L-x}{x(L-x)}M_p$$

If both the ends are fixed

Than one more plastic hinge will develop at B

External work done = W_u x θ_1

Internal work done = $2M_p\,\theta_1 + 2M_p\,\theta_2$



$= 2M_p\left(\theta_1 + \dfrac{x\theta_1}{(L-x)}\right)$

$= 2M_p\theta_1\left(1 + \dfrac{x}{L-x}\right) = \dfrac{2M_p}{x}\left(\dfrac{L}{L-x}\right)$

By the principle of virtual work,

External work done = Internal work done

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

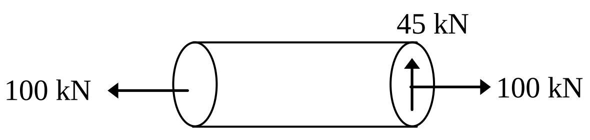
$$W_u = \frac{2L}{x(L-x)} M_p$$

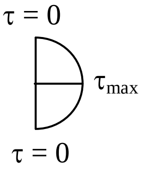
Increase in the ratio of collapse load = $\dfrac{2L}{x(L-x)} \times \dfrac{x(L-x)}{2L-x}$

$$= \frac{2L}{2L-x}$$

01(b).

Sol:





$P = 100 \times 10^3$ N $S = 45 \times 10^3$ N

$S_{yt} = 300$ MPa   FOS = 3

$\therefore$ Normal stress = $\sigma_x = \dfrac{P}{A} = \dfrac{100 \times 10^3}{\frac{\pi}{4} d^2} = \dfrac{127.32 \times 10^3}{d^2}$

$$\tau_{max} = \frac{4}{3}\tau_{avg} = \frac{4}{3} \times \frac{S}{A}$$

$$= \frac{4}{3} \times \frac{45 \times 10^3}{\frac{\pi}{4} \times d^2} = \frac{76.39 \times 10^3}{d^2}$$

$\therefore$ Major principal stress = $\sigma_1 = \dfrac{\sigma_x}{2} + \sqrt{\left(\dfrac{\sigma_x}{2}\right)^2 + \tau_{xy}^2}$

$$= \frac{127.32 \times 10^3}{2d^2} + \sqrt{\left(\frac{127.32 \times 10^3}{2d^2}\right) + \left(\frac{76.39 \times 10^3}{d^2}\right)^2}$$

$$= \frac{163.09 \times 10^3}{d^2}$$

According to maximum principal stress theory,

$$\sigma_1 \leq \frac{S_{yt}}{FOS}$$

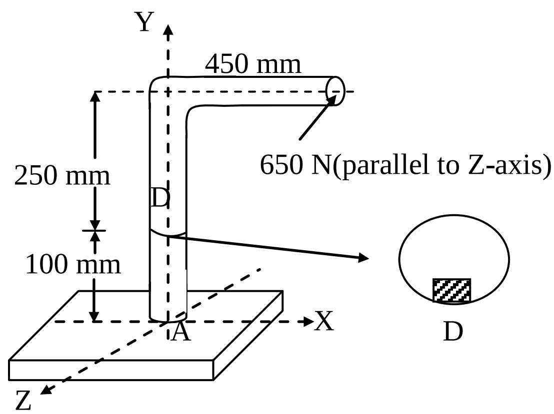
$$\frac{163.09 \times 10^3}{d^2} \leq \frac{300}{3}$$

$$d^2 \geq 1630.90$$

$$d = 40.38 \text{ m}$$

01(c).

Sol:



As point 'D' is on the extreme fiber of the cross section

∴ Shear stress is zero at point D due to applied point load

Torque = 650 × 0.45 = 292.5 Nm

Bending moment = 650 × 0.25 = 162.5 Nm

$$\tau = \frac{TR}{J} = \frac{16T}{\pi d^3}$$

$$= \frac{16 \times 292.5 \times 10^3}{\pi (30)^3}$$

$$\tau = 55.2 \text{ N/mm}^2$$

$$\sigma = \frac{32M}{\pi d^3}$$

$$= \frac{32 \times 162.5 \times 10^3}{\pi \times 30^3}$$

$$\sigma = 61.33 \text{ N/mm}^2$$

Principal stresses

$$\sigma_{p_1}/\sigma_{p_2} = \frac{\sigma}{2} \pm \frac{1}{2}\sqrt{\sigma^2 + 4\tau^2}$$

$$= \frac{61.33}{2} \pm \frac{1}{2}\sqrt{61.33^2 + 4\times 55.2^2}$$

$$= 30.665 \pm 63.14$$

$$\sigma_{p_1}/\sigma_{p_2} = (93.80, -32.48)\ \text{N/mm}^2$$

Location:

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

$$= \frac{2\times 55.2}{61.33}$$

$\text{Tan } 2\theta_p = 1.8$

$$\theta_{p_{1/2}} = 30.47^\circ, 120.47^\circ$$

01(d).

Sol: The various methods of artificial seasoning are as follows:

(1) Boiling (2) Chemical seasoning

(3) Electrical seasoning (4) Kiln seasoning

(5) Water seasoning

(1) **Boiling:** The timber is immersed this is a very quick method. The timber is thus boiled with water for about three to four hours. It is then dried very slowly under a shed. The periods of seasoning and shrinkage are reduced by this method, but it affects the elasticity and strength of wood. In place of boiling water, the timber may be exposed to the action of hot steam. This method of seasoning proves to be costly.

(2) **Chemical seasoning:** This is also known as the salt seasoning. In this method, the timber is immersed in a solution of suitable salt. It is then taken out and seasoned in the ordinary way. The interior surface of timber dries in advance of exterior one and chances of formation of external cracks are reduced.

(3) **Electrical seasoning:** In this method, the use is made of high frequency alternating currents. The timber, when it is green, offers less resistance to the flow of electric current. The resistance increases as the wood dries internally which also results in the production of heat. This is the most rapid method of seasoning. But the initial and maintenance costs are so high that it becomes uneconomical to season timber on commercial base by this method.

(4) **Kiln seasoning:** In this method, the drying of timber is carried out inside an airtight chamber or oven. The process of seasoning is as follows:

(i) The timber is arranged inside the chamber such that spaces are left for free circulation of air.

(ii) The air which is fully saturated with moisture and which is heated to a temperature of about 35°C to 38°C is then forced inside the chamber by suitable arrangement.

(iii) This forced air is allowed to circulate round the timber pieces. As air is fully saturated with moisture, the evaporation from the surfaces of timber pieces is prevented. The heat gradually reaches inside the timber pieces.

(iv) The relative humidity is now gradually reduced.

(5) **Water seasoning:** In this method, the following procedure is adopted:

(i) The timber is cut into pieces of suitable sizes.

(ii) These pieces are immersed wholly in water, preferably in running water of a stream. The care should be taken to see that the timber is not partly immersed.

(iii) The thicker or larger end of timber is kept pointing on the upstream side.

(iv) The timber is taken out after a period of about 2 to 4 weeks. During this period, the sap contained in timber is washed away by water.

(v) The timber is then taken out of water and allowed to dry under a shed having free circulation of air. The water that has replaced sap from the timber dries out and the timber is seasoned.

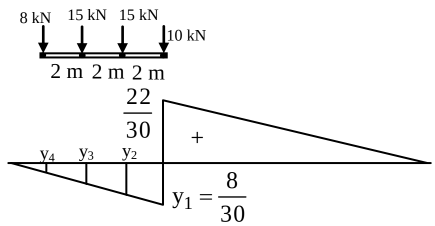
01(e).

Sol: The beam is shown in fig. ILD for shear force at 8 m form left support is shown in fig, along with possible load position for maximum negative shear force. Maximum negative SF at C.

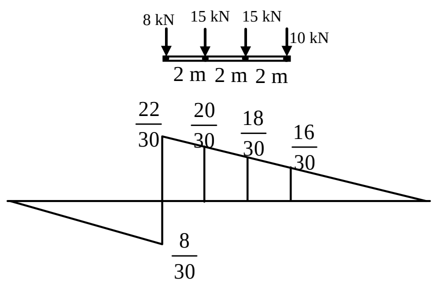
$$= 10\, y_1 + 15\, y_2 + 15\, y_3 + 8\, y_4$$

$$= 10\times\frac{8}{30} + 15\times\frac{6}{30} + 15\times\frac{4}{30} + 8\times\frac{2}{30} = 8.2 \text{ kN}$$

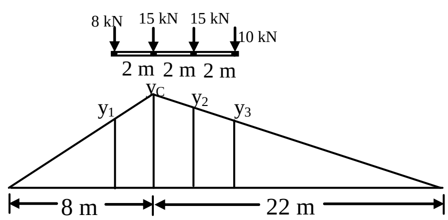
Load position for maximum –ve F_C



Load position for maximum +ve F_C



ILD for M_C



For maximum positive SF at C, load position is as shown in figure. S.F. at C

$$=10\times\frac{16}{30}+15\times\frac{18}{30}+15\times\frac{20}{30}+8\times\frac{22}{30}$$

= 30.2 kN

Check for another position, i.e., when W_3 = 15 kN load is on the section

$$\text{S.F at C} = 10\times\frac{18}{30}+15\times\frac{20}{30}+15\times\frac{22}{30}-8\times\frac{6}{30}$$

= 25.4 kN

$\therefore$ Maximum positive shear force is = 30.2 kN

ILD for bending moment at C is as shown in figure. The maximum ordinate.

$$y_c=\frac{z(L-z)}{L}=\frac{8(30-8)}{30}$$

To find the load position for maximum moment, average load on portion AC and CB are to be found as loads crosses section C on after another

Calculations to find load position for maximum M_C

Load crossing	Average load		Remarks
	AC W_{1av}	BC W_{2av}	
10 kN	$\frac{38}{8}$	$\frac{10}{22}$	$W_{1av} > W_{2av}$
15 kN	$\frac{23}{8}$	$\frac{25}{22}$	$W_{1av} > W_{2av}$
15 kN	$\frac{8}{8}$	$\frac{40}{22}$	$W_{1av} < W_{2av}$

Hence, load position for maximum moment at C is when second 15 kN load is on C. Referring for Figure.

Maximum $M_c = 8\ y_1 + 15\ y_c + 15\ y_2 + 10\ y_3$

$$= 8\left(\frac{6}{8}\right)y_c + 15y_c + 15\left(\frac{20}{22}\right)y_c + 10\left(\frac{18}{22}\right)y_c$$

$= 251.21$ kNm, since $y_c = 5.867$

02(a)(i).

Sol:

1. Pig iron:

Composition: Pig iron contains 3-4% carbon, 0.5-3.5% silicon, 0.5-2% manganese, 0.02-0.1% sulphur and 0.03-1% phosphorus.

Uses: Pig iron is most suitable for making columns, base plates, door brackets, etc.

2. Cast iron:

Composition: Pig iron is remelted with limestone (flux) and coke and refined in Cupola furnace. It is then poured into moulds of desired size and shape. The product is known as cast iron containing about 2-4% of carbon in two forms, i.e., as the compound cementite-in a state of chemical combination; and as free carbon-in a state of mechanical mixture. Carbon in the first form is called combined carbon, and graphite in the latter form. The quality of cast iron thus depends upon the state in which carbon exists in it. The word cast iron is a misnomer as steel with carbon content less than 2% can also be cast. The striking difference between steel and cast iron is that the former is plastic and forgeable while the latter is not. However, some of the modern cast iron develop a fair degree or plasticity and toughness.

Uses: On account of cheapness, strength, ease with which it may be melted and cast into more or less intricate shapes, ease of machining, high damping capacity, and ease with which its hardness may be varied, cast iron is the most used of the cast metals employed in engineering constructions and machines. Some of the more common uses of cast iron are making ornamental castings such as wall brackets, lamp posts; bathroom fittings such as cisterns, water pipes, sewers manhole covers, sanitary fittings and; rail chairs, carriage wheels and machine parts subjected to shocks. It is used as basic material for manufacturing wrought iron and mild steel.

02(a)(ii).

Sol: Admixtures may be classified as accelerators, retarders, water proofers, workability agents surface active agents, pozzolanas etc.

Accelerators: Normally reduce the setting time, accelerate the rate of hydration of cement and consequently the rate of gain of strength. The examples of accelerators are sulphates with an exception of calcium sulphate, alkali carbonates aluminates and silicates, aluminium chloride, calcium chloride, sodium chloride, sodium and potassium hydroxides, calcium formate, formaldehyde, para formaldehyde etc.

Retarders: Normally increase the setting time and thus delay the setting of cement. Since these reduce the rate of hydration, more water is available and better is the workability. Retarders increase the compressive strength under freezing and thawing. Calcium sulphate, sugar, starch, cellulose, ammonium, ferrous and ferric chlorides, sodium hexametaphosphate, lignosulphonic acid and their salts, carbohydrates, hydrocarboxylic acids and their salts are few examples of retarders.

Water Proofers: Cement mortar or concrete should be impervious to water under pressure and also should have sufficient resistance to absorption of water. The examples of water repellent materials such as soda and potash soaps are chemically active, whereas calcium soaps, resin, vegetable oil, fats, waxes and coal tar residue are examples of chemically inactive materials.

Air Entraining Agents: The air intentionally introduced in the cement during its manufacture or during making concrete is known as entrained air. It is different from entrapped air where the continuous channels are formed, thus increasing the permeability. In the case of entrained air, the voids formed are discontinuous and are less than 0.05 mm in diameter. Air entrainment increases workability, and resistance of concrete to weathering. The possibility of bleeding segregation and laitance is also reduced.

Pozzolanas: These are siliceous materials which are themselves inactive but react, in the presence of water, with lime to form compounds having cementitious properties. The examples of pozzolana are lime, fly ash, burnt clay and blast furnace slag. Pozzolanas react with free lime in cement and improve the durability of concrete, and reduce the rate of hardening of concrete, which is the principal objection to its use.

02(a)(iii).

Sol: Qualities of Good Bricks:

The good bricks which are to be used for the construction of important structures should possess the following qualities.

(i) The bricks should be table-moulded, well-burnt in kiln, copper-coloured, free from cracks and with sharp and square edges. The colour should be uniform and bright.

(ii) The bricks should be uniform in shape and should be of standard size.

(iii) The bricks should give a clear metallic ringing sound when struck with each other.

(iv) The bricks when broken or fractured should show a bright homogenous and uniform compact structure free from voids.

(v) The brick should not absorb water more than 20 percent by weight for first class bricks and 22 percent by weight for second class bricks, when soaked in cold water for a period of 24 hours.

(vi) The bricks should be sufficiently hard. No impression should be left on bricks surface, when it is scratched with finger nail.

(vii) The bricks should not break into pieces when dropped flat on hard ground from a height of about one meter.

(viii) The bricks should have low thermal conductivity and they should be sound-proof.

(ix) The bricks, when soaked in water for 24 hours, should not show deposits of white salts when allowed to dry in shade.

(x) No brick should have the crushing strength below 5.50 N/mm^2.

Factors effecting thickness of brick wall:

The thickness specified for a wall is determined by such factors as damp proofing considerations, whether or not the wall has a cavity, load-bearing requirements, and expense. Wall thickness specification has proven considerably various, and while some non-load-bearing brick walls may be as little as half a brick thick, others brick walls will be much thicker.

Test conducted on bricks

1. Absorption
2. Crushing strength
3. Hardness
4. Presence of soluble salts
5. Shape and size
6. Soundness
7. Structure

02(b).

Sol: Given:

Span = l = 4 m

Load = W = 50×10^3 N

h = 20 mm

E = 2×10^5 N/mm^2

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

$$= \frac{(200)(300)^3}{12} = 45 \times 10^7 \text{ mm}^4$$

Equivalent static load = W_e

$$\therefore W(h+\delta) = \frac{1}{2}W_e\delta$$

$$\delta = \frac{W_e\ell^3}{48EI} = \frac{W_e(4\times10^3)^3}{48\times2\times10^5\times45\times10^7} = 1.48\times10^{-5}\,W_e \text{ mm}$$

$$\therefore 50\times10^3(20+1.48\times10^{-5}W_e) = \frac{1}{2}\times W_e\times1.48\times10^{-5}W_e$$

$$\therefore W_e^2 = 13.5\times10^{10} + 100000\,W_e$$

$$\therefore W_e = 4.208\times10^5 \text{ N} = 420.8 \text{ kN}$$

$$\therefore \delta = 1.48\times10^{-5}\times(4.208\times10^5) = 6.231 \text{ mm}$$

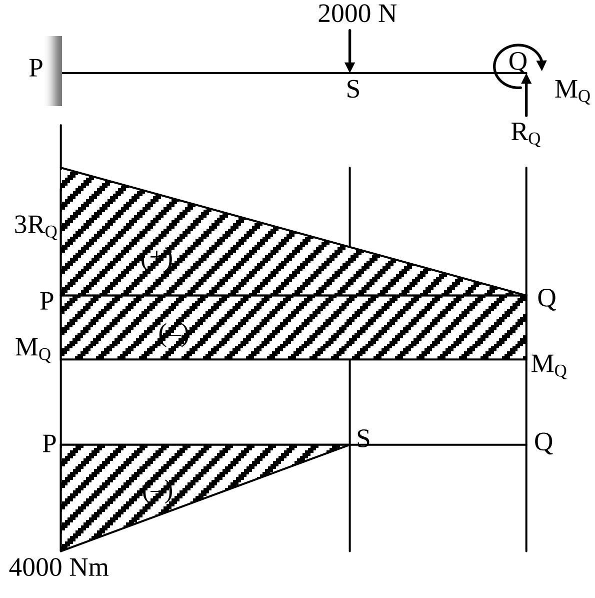
$$\therefore M_{max} = \frac{W_e\ell}{4} = \frac{4.208\times10^5\times4000}{4} = 1.052\times10^8 \text{ N-mm}$$

$$f_{max} = \frac{M}{I}.y$$

$$= \frac{1.052\times10^8}{45\times10^7}\times150 = 35.07 \text{ N/mm}^2$$

02(c).

Sol: Bending moment diagram has been drawn by parts as shown in figure.



For fixed beam, we know that,

$\Sigma A_{BMD} = 0$

$$\frac{1}{2}\times 3\times 3R_Q - 3\times M_Q - \frac{1}{2}\times 2\times 4000 = 0$$

$9R_Q - 6\,M_Q = 8000$ ……………..(i)

Also, $\Sigma A_i\, x_i = 0$

$$\left(\frac{1}{2}\times 3\times 3R_Q\right)\times\frac{2}{3}\times 3 - (3\times M_Q)\times\frac{3}{2} - \left(\frac{1}{2}\times 2\times 4000\right)\times\left(1+\frac{2}{3}\times 2\right) = 0$$

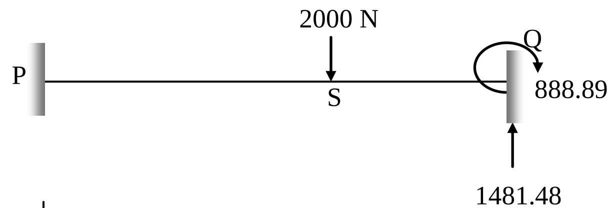
$$9R_Q - \frac{9M_Q}{2} = \frac{7\times 4000}{3}$$

$54R_Q - 27\,M_Q = 56000$ ……………….(ii)

Solving equations (i) and (ii), we obtain,

$R_Q = 1481.48$ N

$M_Q = 888.89$ Nm



Shear Force Calculations:

$F_Q = 0$

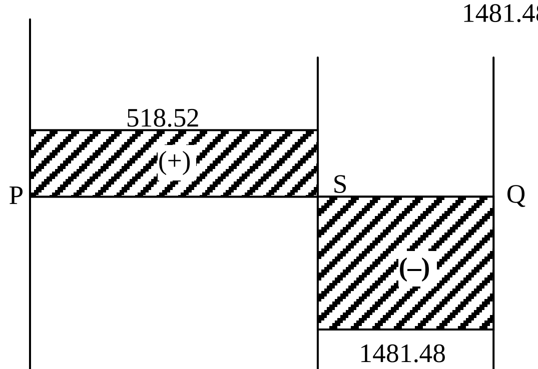
$F_{Q+} = -1481.48$

$F_S^- = -1481.48$ N

$F_{S+} = -1481.48 + 2000$

$\Rightarrow F_{S+} = 518.52$ N

$F_P^- = 518.52$ N



Bending Moment Calculations:

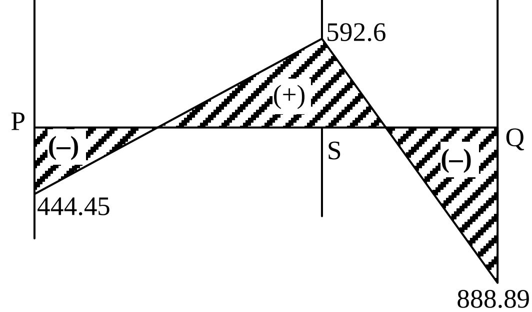
$M_{Q-} = 0$, $M_{Q+} = -888.89$

$M_S = 1481.48 \times 1 - 888.89$

$= +592.6$ Nm

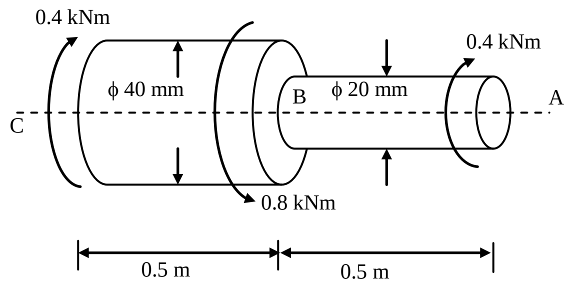
$M_P = 1481.48 \times 3 - 888.89 - 2000 \times 2$

$\Rightarrow M_P = -444.45$



03(a)(i).

Sol: Stepped shafts made of steel



Length (BC) = 0.5 m

Length (AB) = 0.5 m

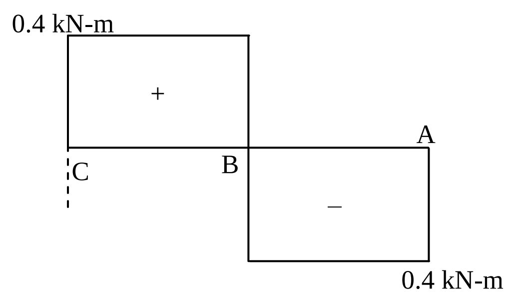
Diameter (BC) = 40 mm

Diameter (AB) = 20 mm

Angle of twist of one end of the shaft with respect to other is obtained by summation using the general formula.

$$\theta = \sum_{i=1}^{n} \frac{T_i L_i}{G_i I_{p_i}}$$

Twisting moment diagram



$$\theta = \theta_{BC} + \theta_{AB} = \frac{T_{BC} L_{BC}}{GJ_{BC}} + \frac{T_{AB} L_{AB}}{GJ_{AB}}$$

$$\Rightarrow L_{BC} = L_{AB} = L$$

$$\Rightarrow \frac{L}{G}\left[\frac{T_{BC}}{J_{BC}} + \frac{T_{AB}}{J_{AB}}\right]$$

$T_{BC} = +\, 0.4$ kN-m

$T_{AB} = -0.4$ kN-m

$$J_{BC} = \frac{\pi}{32} D_{BC}^4 = \frac{\pi}{32} \times (40 \times 10^{-3})^4$$

$J_{BC} = 2.513 \times 10^{-7}$ m^4

$$J_{AB} = \frac{\pi}{32} \times (20 \times 10^{-3})^4$$

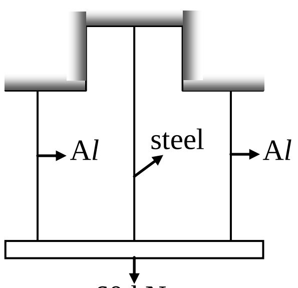
$J_{AB} = 1.57 \times 10^{-8}$ m^4

$$\theta = \frac{0.5}{84 \times 10^9}\left[\frac{0.4}{2.513 \times 10^{-7}} + \frac{(-0.4)}{1.57 \times 10^{-8}}\right]$$

$\theta = 0.142 \times 10^{-3}$ (Counter clockwise)

03(a)(ii).

Sol:



As the Bar is rigid, elongation in each wire will be same. Let load P is carried by aluminimum wire.

$\delta_{st} = \delta_{al}$

$$\frac{(60 - 2P) \times 8}{200 \times 200} = \frac{P \times 4}{300 \times 66.7}$$

$$\frac{60 - 2P}{P} = 1$$

P = 20 kN

Load in steel wire = 60 – 2P = 20 kN

∴ Load in all 3 wires is 20 kN

03(b).

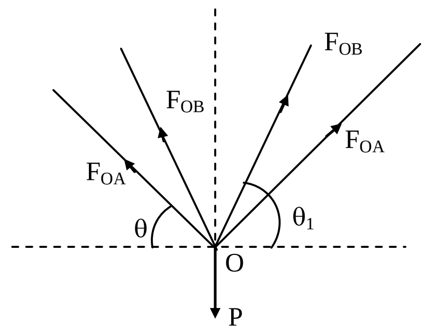
Sol: Due to symmetry $F_{OA} = F_{OD}$ and $F_{OB} = F_{OC}$

$D_S = 1$

Let redundant force in members OA and OD are F_{OA}

Consider vertical equilibrium of joint 'O'

$\Rightarrow 2F_{OA} \sin\theta + 2F_{OB} \sin\theta_1 = P$



$$\Rightarrow 2\left[F_{OA} \times \frac{3}{5} + F_{OB} \times \frac{1}{\sqrt{2}}\right] = P$$

$$F_{OB} = \left[\frac{P}{2} - F_{OA} \times \frac{3}{5}\right]\sqrt{2} \quad \_\_\_\_\_\_ (1)$$

From principle of least work

$$\Rightarrow \frac{\partial U}{\partial F_{OA}} = 0$$

$$\Rightarrow U = \frac{\sum P^2 L}{2AE}$$

$$\frac{\partial U}{\partial F_{OA}} = \sum_{n=1}^{4} \frac{P\left(\frac{\partial P}{\partial f_{OA}}\right)L}{AE}$$

$$= \sum_{n=1}^{4} \frac{L}{AE} P\left(\frac{\partial P}{\partial f_{OA}}\right)$$

$$= 2\left[\frac{L}{AE} P \frac{\partial P}{\partial F_{OA}}\right]_{OA} + 2\left[\frac{L}{AE} P \frac{\partial P}{\partial F_{OA}}\right]_{OB}$$

$$= 2\left[\frac{5}{AE} F_{OA}.1\right] + 2\left[\frac{3\sqrt{2}}{AE} F_{OB}\left(\frac{-3\sqrt{2}}{5}\right)\right]$$

$$\Rightarrow \frac{5F_{OA}}{AE} + \frac{\left[\frac{3\sqrt{2}}{AE}\left(\frac{1}{2} - F_{OA} \times \frac{3}{5}\right)\sqrt{2}\right]}{\frac{-3\sqrt{2}}{5}} = 0$$

$$5F_{OA} - \frac{18\sqrt{2}}{5}\left[\frac{P}{2} - F_{OA} \times \frac{3}{5}\right] = 0$$

$$F_{OA}\left(5+\frac{54\sqrt{2}}{25}\right)=\frac{9\sqrt{2}P}{5}$$

$F_{OA} = 0.316$ P

Substitute F_{OA} in equation (1)

$$F_{OB}=\left[\frac{P}{2}-0.316P\times\frac{3}{5}\right]\sqrt{2}$$

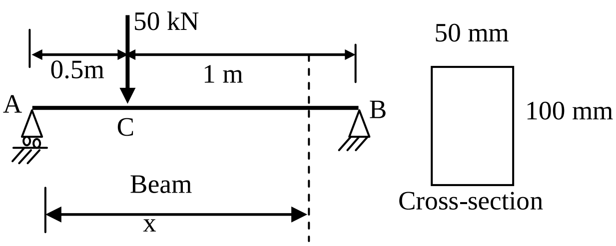
$$F_{OB}=0.4389P$$

$F_{OA} = F_{OD} = 0.316P$

$F_{OB} = F_{OC} = 0.4389P$

03(c).

Sol:



Maximum bending moment occurs under load.

$$M_C=\frac{50\times0.5\times1}{1.5}=16.67\,kN-m$$

Maximum bending stress

$$f=\frac{M}{Z}=\frac{16.67\times10^6}{\left(\frac{50\times100^2}{6}\right)}=200\,MPa$$

(Note: Tensile stress will develop on bottom face and compressive stress on top face).

$M_x = R_A(x) - 50(x - 0.5)$

Here, $R_A = \frac{50\times2}{3}$

$$EI\left(\frac{d^2y}{dx^2}\right) = M_x = R_A(x) - 50(x - 0.5)$$

$$= \frac{50\times 2}{3}(x) - 50(x - 0.5)$$

$$EI\left(\frac{d^2y}{dx^2}\right) = 25 - \frac{50}{3}(x)$$

Integrating

$$EI\left(\frac{dy}{dx}\right) = 25x - \frac{50}{3}(\frac{x^2}{2}) + c_1$$

Again integrating

$$EI(y) = 25\frac{x^2}{2} - \frac{50x^3}{3\times 3\times 2} + c_1x + c_2$$

Boundary Conditions :

At A, x = 0, y = 0, $\therefore c_2 = 0$

At B, x = 1.5 m, y = 0

$$0 = 12.5(1.5)^2 - \frac{50}{18}(1.5)^3 + c_1(1.5) + c_2$$

$\Rightarrow c_2 = -12.5$

At x = 1 m from A (or 0.5 m from B)

$$EI(y) = 25\frac{(1)^2}{2} - \frac{50(1)^3}{18} - 12.5(1) + 0$$

$$y = \frac{-2.78}{EI}$$

$$EI = \left(2\times 10^5\right)\left(\frac{50\times 100^3}{12}\right) = 8.33 \times 10^{11} \text{ N-mm}^2$$

$EI = 833.33 \text{ kN-m}^2$

$$\therefore y = \frac{-2.78}{833.33} = 0.00334\text{ m} = 3.34\text{ mm}$$

04(a)(i).

Sol: The following are the most commonly used stone quarrying methods.

1. Excavation
2. Wedging
3. Heating
4. Blasting.

04(a)(ii).

Sol: Requirements of a Good Aggregate:

Following are the desirable properties or requirements of a good aggregate.

(1) Adhesion (2) Cementation
(3) Durability (4) Hardness
(5) Shape (6) Strength
(7) Toughness

(1) **Adhesion:** The aggregates which are to be used for the construction should have less affinity with water as compared with the binding material. If this quality is absent in the aggregate, it will lead to the separation of bituminous or cement coating in the presence of water.

(2) **Cementation:** The binding quality of the aggregate depends on its ability to form its own binding material under different loading so as to make the rough broken stone pieces grip together to resist displacement.

(3) **Durability:** The durability of an aggregate indicates its resistance to the action of weather and is largely dependent upon its petrological composition. The metal is subjected to the oxidizing influence of air and rain water. It is therefore desirable that the aggregate should possess sufficient soundness to resist the action of weather and age so that the life of the structure made with it may be prolonged.

(4) **Hardness:** The aggregates should be reasonable hard to offer resistance to the actions of abrasion and attrition. The aggregates are always subjected to the constant rubbing action. It is known as abrasion and it will be increased due to the presence of abrasive material like sand between the exposed top surface and the tyres of moving vehicles. The abrasive action is very severe for roads which are used by the steel tyred vehicles. The mutual rubbing of stones is known as attrition and it may also cause a little wear in the aggregates.

(5) Shape: The shape of aggregates may be rounded, cubical, angular, flaky or elongated. The flaky and elongated particles possess less strength and durability and their use in the construction should be avoided as for as possible. The rounded particles are preferred in cement concrete construction. But they are unsuitable in W.B.M. construction, bituminous construction and in granular base course because their stability due to interlocking in less. The angular particles are preferred in such types of construction.

(6) Strength: The aggregates should be sufficiently strong to withstand the stresses developed due to the wheel loads of the traffic. This property is especially desirable for the road aggregates which are to be used in top layers of the pavement. Thus, the wearing course of road should be composed of aggregate which possess enough strength in addition to enough resistance to crushing.

(7) Toughness: The toughness of an aggregate is that property which enables the aggregate to resist fracture when struck with a hammer and it is necessary in a metal to withstand the impact blows caused by traffic. The magnitude of impact is governed by the roughness of surface, speed of the vehicle and other vehicular characteristics. It is desirable that the aggregate is reasonably tough.

Fineness modulus: Fineness Modulus is a ready index of coarseness or fineness of the material. Fineness modulus is an empirical factor obtained by adding the cumulative percentages of aggregate retained on each of the standard sieves ranging from 80 mm to 150 micron and dividing this sum by an arbitrary number 100. The larger the figure, the coarser is the material.

Many a time, fine aggregates are designated as coarse sand, medium sand and fine sand may be really medium or even coarse sand. To avoid this ambiguity fineness modulus could be used as a yard stick to indicate the fineness of sand.

The following limits may be taken as guidance:

Fine sand: Fineness Modulus: 2.2 – 2.6

Medium sand: F. M. : 2.6 – 2.9

Coarse sand : F. M. : 2.9 -3.2

A sand having a fineness modulus more than 3.2 will be unsuitable for making satisfactory concrete.

04(a)(iii).

Sol: A wood panel glued under pressure from an odd number (usually 3 to 13) of layers of veneers is known as plywood. This results in improved dimensional stability, stiffness and strength. The outer most vaneer sheets in a plywood panel are called/faces. The interior plies which have their grain directions parallel to that of the faces are termed as core. Other piles which have grain directions perpendicular to that in the face are termed as cross bands.

Plywood may be classified upon direction of grains in the plies and on the type of adhesive used. Normally the alternate plies are oriented at 30° or 60° in star plywood. The faces are arranged with the grain at 45° to that of the centres in diagonal plywood. When the plies are bounded together with water soluble glues such as casein glue, interior grade plywood is obtained and when bonded with phenol formaldehyde adhesive it is identified as exterior grade plywood which is completely water proof.

Advantages:

1. It has good strength both along as well as across the grains.
2. The wood shrinks or swells more across the grains. Since plywood has cross-grained construction, the tendency to shrink or swell is reduced.
3. It has better splitting resistance due to the grains in adjacent veneers in cross direction as such nailing can be done very safely even near the edges.
4. Plywood can be curved into desired shapes.
5. High-grade plywood is superior to most metals in strength-to-weight ratio.

Uses: These are extensively used for partitions, ceilings, doors, concrete form work, plywood boards, lamin boards (built-up boards with core strips up to 7 mm in thickness and block boards (built-up boards with core strips upto 25 mm in width) etc.

04(b).

Sol: $\theta_A = 0$ & θ_B & $\theta_C \neq 0$ $\Delta = 15$ mm

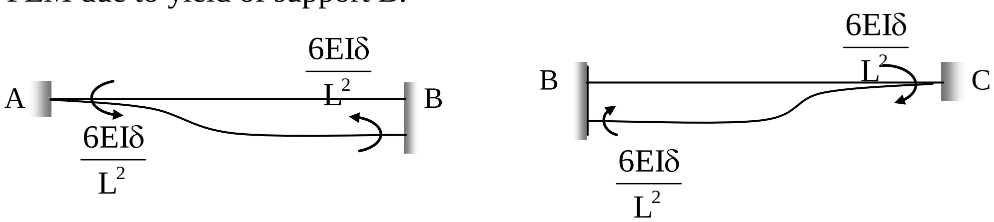
$$M_{AB} = -\frac{Wab^2}{L^2} = -\frac{100 \times 4 \times 2^2}{6^2} = -44.44 \text{ kN-m}$$

$$M_{BA} = \frac{Wa^2b}{L^2} = \frac{100 \times 4^2 \times 2}{6^2} = 88.89 \text{ kNm}$$

$$M_{CB} = -\frac{wL^2}{12} = -\frac{20\times 5^2}{12} = -41.67 \text{ kNm}$$

M_{CD} = – 20 × 1.5 = – 30 kNm

FEM due to yield of support B.



For span AB:

$$M_{AB} = M_{BA} = -\frac{6EI}{L^2}\Delta = \frac{-6\times 200\times 10^5\times 120\times 10^{-6}}{6^2}\frac{15}{1000} = -6\text{kNm}$$

For span BC:

$$M_{BC} = M_{CB} = \frac{6EI}{L^2}\Delta = \frac{6\times 200\times 10^5\times 120\times 10^{-6}}{5^2}\frac{15}{1000} = 8.64 \text{ kNm}$$

Slope deflection equations are

$$M_{AB} = M_{FAB} + \frac{2EI}{6}\left[2\theta_A + \theta_B - 3\frac{\Delta}{6}\right] = -44.44 + \frac{EI}{3}\theta_B - 6 = -50.44 + \frac{EI}{3}\theta_B \rightarrow (1)$$

$$M_{BA} = M_{FBA} + \frac{2EI}{6}\left[2\theta_B + \theta_A - 3\frac{\Delta}{6}\right] = 88.89 + \frac{2EI}{3}\theta_B - 6 = 82.89 + \frac{2EI}{3}\theta_B \rightarrow (2)$$

$$M_{BC} = M_{FBC} + \frac{2EI}{5}\left[2\theta_B + \theta_C + 3\frac{\Delta}{5}\right] = -41.67 + \frac{4EI}{5}\theta_B + \frac{2EI}{5}\theta_C + 8.64$$

$$= -33.03 + \frac{4EI}{5}\theta_B + \frac{2EI}{5}\theta_C$$

$$M_{CB} = M_{FCB} + \frac{2EI}{5}\left[2\theta_C + \theta_B + 3\frac{\Delta}{5}\right] = 41.67 + \frac{4EI}{5}\theta_C + \frac{2EI}{5}\theta_B + 8.64$$

$$= 50.31 + \frac{4EI}{5}\theta_C + \frac{2EI}{5}\theta_B$$

M_{CD} = – 20 × 1.5 = – 30 kNm

In all the above equations there are only 2 unknowns and accordingly the boundary conditions are

$M_{BA} + M_{CB} = 0$

$M_{CB} + M_{CD} = 0$

$$M_{BA} + M_{BC} = 82.89 + \frac{2EI}{3}\theta_B - 33.03 + \frac{4EI}{5}\theta_B + \frac{2EI}{5}\theta_C = 49.86 + \frac{22}{15}EI\theta_B + \frac{2}{5}EI\theta_C = 0 \rightarrow (5)$$

$$M_{CB} + M_{CD} = 50.31 + \frac{4EI}{5}\theta_C + \frac{2EI}{5}\theta_B - 30 = 20.31 + \frac{2EI}{5}\theta_B + \frac{4EI}{5}\theta_C = 0 \quad \rightarrow (6)$$

Solving equations (5) & (6)

$$\theta_B = -\frac{31.35}{EI}$$

$$\theta_C = \frac{-9.71}{EI}$$

Substituting the values in the slope deflections we have.

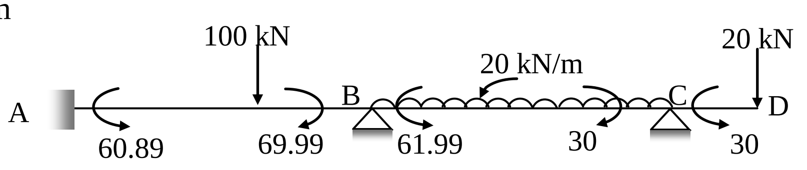
$$M_{AB} = -50.44 + \frac{EI}{3} \times \left(-\frac{31.35}{EI}\right) = -60.89 \text{ kNm}$$

$$M_{BA} = 82.89 + \frac{2EI}{3} \times \left(\frac{-31.35}{EI}\right) = 61.99 \text{ kNm}$$

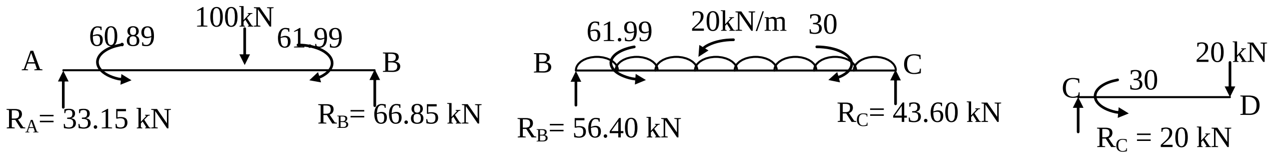
$$M_{BC} = -33.03 + \frac{4EI}{5}\left(\frac{-31.35}{EI}\right) + \frac{2EI}{5}\left(\frac{-9.71}{EI}\right) = -61.99 \text{ kNm}$$

$$M_{CB} = 50.31 + \frac{4EI}{5}\left(\frac{-9.71}{EI}\right) + \frac{2EI}{5}\left(-\frac{31.35}{EI}\right) = 30 \text{ kNm}$$

M_{CD} = – 30 kNm



Consider the free body diagram of continuous beam for finding reaction



Reactions

Span AB:

$R_B \times 6 = 100 \times 4 + 61.99 - 60.89$

$R_B = 66.85$ kN

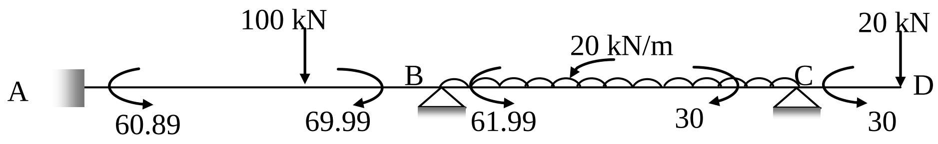
$R_A = 100 - R_B = 33.15$ kN

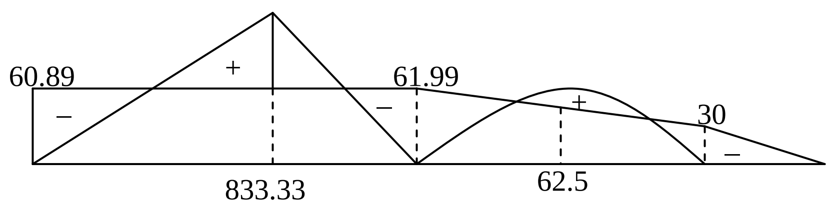
Span BC:

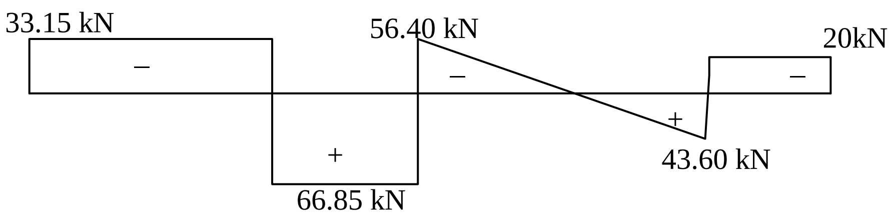
$$R_B \times 5 = 20 \times \frac{5}{2} \times 5 + 61.99 - 30$$

$$R_B = 56.40 \text{ kN}$$

$R_C = 20 \times 5 - R_B = 43.60$ kN







S.F.D

04(c)(i).

Sol: Flexural stiffness for a cantilever beam, $k = \frac{3EI}{L^3}, k = \frac{3 \times 2.1 \times 10^6 \times 1300}{(300)^3}$

$= 303$ kg/cm $= 303 \times 9.81$ N/cm

$= 303 \times 9.81 \times (1/100)$ N/cm

$= 2.97 \times 10^5$ N/cm

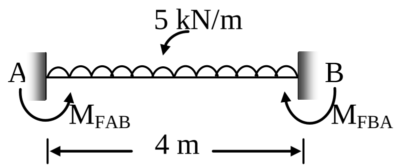
Natural frequency, $\omega_n = \sqrt{\frac{k}{m}} = \sqrt{\frac{2.97 \times 10^5}{500}} = 24.37$ rad/s

or $f = \frac{\omega_n}{2\pi} = 3.88$ cps

Natural period, $T = \frac{1}{f} = \frac{2\pi}{\omega_n} = 0.26$ s

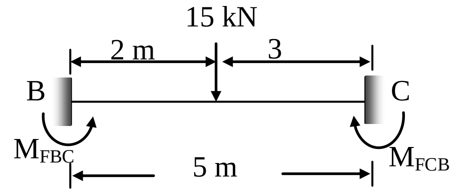
04(c)(ii).

Sol:



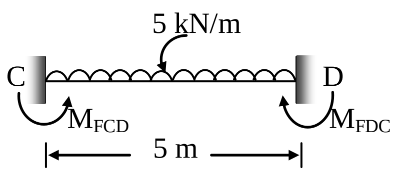
$$M_{FAB} = \frac{-w\ell^2}{12} = \frac{-5\times 4^2}{12} = -6.67 \text{ kN-m}$$

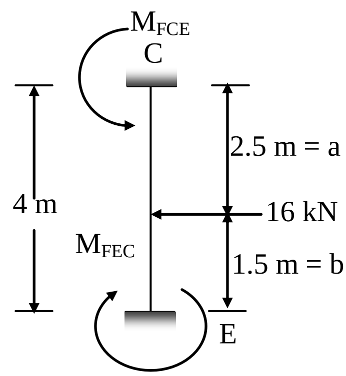
$$M_{FBA} = \frac{w\ell^2}{12} = \frac{5\times 4^2}{12} = 6.67 \text{ kN-m}$$



$$M_{FBC} = \frac{-wab^2}{\ell^2} = \frac{-15\times 2\times 3^2}{5^2} = -10.8 \text{ kN-m}$$

$$M_{FCB} = \frac{wa^2b}{\ell^2} = \frac{5\times 2^2\times 3}{5^2} = 7.2 \text{ kN} - \text{m}$$





$$M_{FCD} = \frac{-w\ell^2}{12} = \frac{-5\times 5^2}{12}$$

$$= -10.42 \text{ kN-m}$$

$$M_{FDC} = \frac{w\ell^2}{12} = 10.42 \text{ kN-m}$$

$$M_{FCE} = \frac{-wab^2}{\ell^2} = -\frac{16\times 2.5\times 1.5^2}{4^2} = -5.625 \text{ kN-m}$$

$$M_{FEC} = \frac{wa^2b}{\ell^2} = -\frac{16\times 2.5^2\times 1.5}{4^2} = 9.375 \text{ kN-m}$$

The fixed end moments (FEMS) have been worked out on the approximation that the 4 'beams' in the frame act independently. Ends BA, BC, CB, CD, DC, & CE are not really fixed ends. They are assumed to be, as a first approximation in the moment distribution process.

05(a).

Sol: Unit weight of concrete = 24 kN/m^3

A = (1.2 × 1.8) – (0.8 × 1.4) = 1.04 m^2

g = (1.04 × 24) = 25 kN/m

P = 7000 kN

e = 800 mm and L = 40 m

$$I = \frac{1}{12}[(1200 \times 1800^3) - (800 \times 1400^3)]$$

$$= 40 \times 10^{10}\ \text{mm}^4$$

$$Z_b = Z_t = Z = \frac{(40 \times 10^{10})}{900} = 444 \times 10^6\ \text{mm}^3$$

$$M_g = (0.125 \times 25 \times 40^2) = 5000\ \text{kN-m}$$

M_q = 2000 kN-m

M = (M_g + M_q) = 5000 + 2000 = 7000 kN-m

$$\text{Lever arm, } a = \left(\frac{M}{P}\right) = \frac{7000 \times 10^3}{7000}$$

$$= 1000\ \text{mm}$$

Shift of pressure line, e′ = (a – e)

= (1000 – 800)

= 200 mm

The resultant stresses are obtained as

$$F_{sup} = \left[\frac{P}{A} + \frac{Pe'}{Z_t}\right]$$

$$= \left(\frac{7000 \times 10^3}{1.04 \times 10^6}\right) + \left(\frac{7000 \times 10^3 \times 200}{444 \times 10^6}\right)$$

= 6.73 + 3.153 = 9.88 N/mm^2

$$F_{inf} = \left[\frac{P}{A} - \frac{Pe'}{Z_b}\right]$$

$$= \left(\frac{7000 \times 10^3}{1.04 \times 10^6}\right) - \left(\frac{7000 \times 10^3 \times 200}{444 \times 10^6}\right) = 6.73 - 3.153 = 3.58\ \text{N/mm}^2$$

05(b).

Sol: Yield strength of steel $f_y = 250$ Mpa

Ultimate tensile stress of steel $f_u = 410$ Mpa

Gross sectional area of smaller plate

$A_g = 250 \times 10 = 2500 mm^2$

Design tensile strength of the plate based on gross section yielding

$$P = T_{dg} = A_g \frac{f_y}{\gamma_{m0}} = 2500 \times \frac{250}{1.1} = 568.18 \times 10^3 \text{ N}$$

Let S and L_w be the size and effective length of fillet weld respectively.

Size of fillet weld S = 5 mm

Effective throat thickness $t_t = K \times S = 0.7 \times 5$

$= 3.5$ mm

Equating the factored tensile load P= Design shear strength of fillet weld P_{dw}

$$568.18 \times 10^3 = L_w . t_t \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

$$568.18 \times 10^3 = L_w \times 3.5 \times \left(\frac{410}{\sqrt{3} \times 1.25}\right)$$

$L_w = 857.21$ mm

Assuming above effective weld length to be arranged top and bottom and right vertical edge of the plate as shown above figure

Equating $L_w = 2 \times \text{overlap} + 250$

$$\text{Overlap} = \frac{857.21 - 250}{2} = 303.61 \text{ mm}$$

$\simeq$ 305 mm

Adopt 860 mm effective length of fillet weld with an overlap 305 mm as shown in figure

05(c).

Sol: Modular ratio, $m = \frac{E_s}{E_c} = \frac{2 \times 10^5}{3 \times 10^4} = 6.67$

Initial stress in tendon, $\sigma_o = \frac{P}{A_s}$

$$= \frac{560 \times 10^3}{60 \times \frac{\pi}{4} \times 3^2}$$

$= 1320.4\ \text{N/mm}^2$

$$f_c = \frac{P}{A} + \frac{Pe}{I}.e = \frac{560 \times 10^3}{325 \times 325} = 5.3\ \text{N/mm}^2$$

1. Loss of stress due to elastic deformation

$\Delta\sigma = mf_c = 6.67 \times 5.3$

$= 35.35\ \text{N/mm}^2$

2. Loss of stress due to relaxation of steel

$$\Delta\sigma = \frac{5}{100} \times 1320.4$$

$= 66.02\ \text{N/mm}^2$

3. Loss of stress due to shrinkage of concrete

$\Delta\sigma = \varepsilon_{sh} \times E_s$

$= 1.9 \times 10^{-4} \times 2 \times 10^5$

$= 38\ \text{N/mm}^2$

4. Loss of stress due to creep of concrete

$\Delta\sigma = \varepsilon_c\, f_c\, E_s$

$= 25 \times 10^{-6} \times 5.3 \times 2 \times 10^5$

$= 26.5\ \text{N/mm}^2$

Total loss of stress

$= 35.35 + 66.02 + 38 + 26.5$

$= 165.87\ \text{N/mm}^2$

After all losses stress in wires

$= 1320.4 – 165.87$

$= 1154.53\ \text{N/mm}^2$

Prestressing force, $P = 1154.53 \times 60 \times \frac{\pi}{4} \times 3^2$

Stress in concrete

$$= \frac{P}{A} = \frac{1154.53 \times 60 \times \frac{\pi}{4} \times 3^2}{325 \times 325} = 4.64 \text{ N/mm}^2$$

05(d).

Sol: Reorder point (ROP) It is the level of inventory to be maintained before releasing an order in case of uniform demand and constant lead time.

Reorder point = Demand or usage per period × Lead time

Uncertainty in Demand on Reorder Point: Generally, demand is never uniform all throughout the year. In case, the demand has a mean D_m and standard deviation σ_d, the reorder point is expressed as given below:

$$\text{Reorder point} = D_m \times \text{Lead Time} + Z \times \sigma_d \times \sqrt{\text{Lead time}}$$

Where, Z is standard normal variate for a given service level. In order to find the value of Z, one can use normal distribution table. Values of Z for some commonly used service levels are given in the Table. The service level is the probability of having material in stock when demand of this material occurs in a construction project.

Service Level	90%	92%	94%	95%	96%	98%	99%
Z	1.29	1.41	1.56	1.65	1.75	2.05	2.33

Given, Unit item cost (C) = Rs.200

Ordering cost (A) = 100 per order

Annual usage (D) = 10,000 units

Carrying rate (I) = 25%

Economic order quantity: EOQ = $\sqrt{\dfrac{2\times A\times D}{I\times C}}$

$$= \sqrt{\frac{2\times 100\times 10000}{0.25\times 200}}$$

$$= \sqrt{(40,000)}$$

= 200 units

Given, lead time = 2 weeks

Average inventory level= $\dfrac{\text{EOQ}}{\text{Lead time}}$ =200/2 = 100 units

Weekly usage = 10,000 units/52 weeks = 192 units

Reorder inventory level = Weekly usage × Lead time

= 192 units × 2 weeks = 384 units

05(e).

Sol: $\therefore$ Load on staircase = 17.4 kW/m^2

Load on landing:

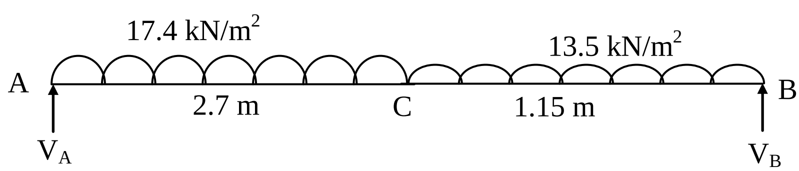
Self weight of slab = 0.2 × 25 = 5 kN/m^2

Live load = 3 kN/m^2

Finisher = 1 kN/m^2

Total load = 9 kN/m^2

Factored load = 9 × 1.5 = 13.5 kN/m^2



$\Sigma M_A = 0$

$\Rightarrow V_B(3.85) - 17.4 \times \frac{2.7^2}{2} - 13.5 \times 1.15(2.7 + \frac{1.15}{2})$

$\Rightarrow V_B = 29.68$ kN

$\therefore V_A = 17.4\,(2.7) + 13.5\,(1.15) - 29.68$

$= 32.825$ kNm

BM in AC: $32.825\,x - \frac{17.4x^2}{2}$ ('x' from A)

For maximum BM: $\frac{dM}{dx} = 0 \Rightarrow x = 1.886$ m

$\therefore$ Maximum BM = 30.96 kNm

In CB:- BM = $29.68\,x - \frac{13.5x^2}{2}$ ('x' from B)

For maximum BM: $\frac{dM}{dx} = 0 \Rightarrow x = 2.19$ m (not possible in BC)

$\therefore$ Maximum BM = 30.96 kNm

Effective depth required $= \sqrt{\frac{M_u}{Q_b}} = \sqrt{\frac{30.96 \times 10^6}{0.135 \times 20 \times 1000}} = 105.9$ mm

Provided d = 180 mm

$\therefore$ Hence safe

Area of steel required:-

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

$$= 0.5 \times \frac{20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 30.96 \times 10^6}{20 \times 1000 \times 180^2}}\right] \times 1000 \times 180 = 506.16 \text{ mm}^2$$

Number of 10 mm bars required $= \frac{506.16}{\frac{\pi}{4} \times 10^2} = 6.44 \approx 7$ No's

06(a).

Sol: Project management process : Projects are composed of processes. A process is "a series of actions bringing about a result". Project processes are performed by people and generally fall into one of two major categories.

1. Project management processes
2. Product oriented processes

Project management processes: These can be organized into five group's of one or more processes each.

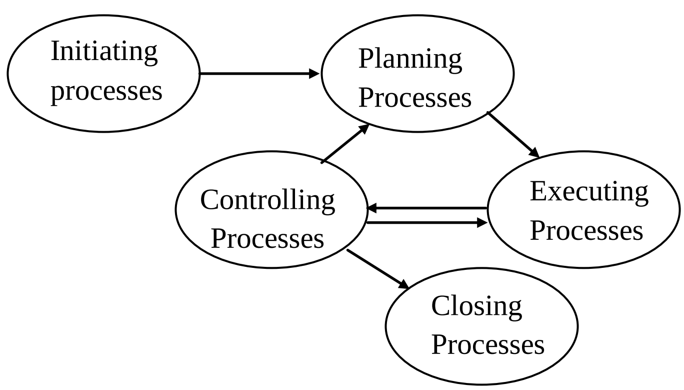
i. Initiating processes-authorizing the project or phase.

ii. Planning processes – defining and refining objectives and selecting the best of the alternatives.

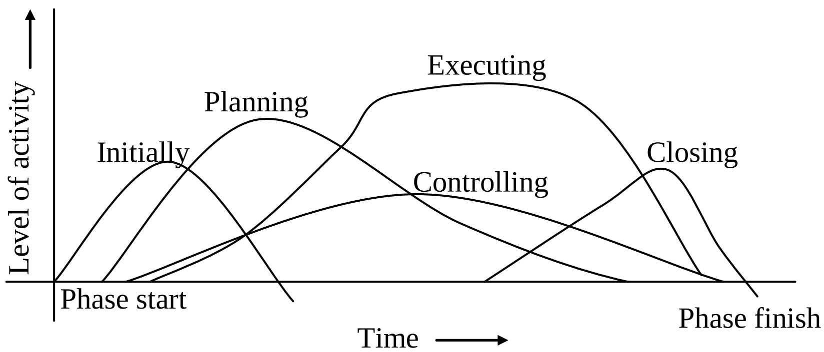
iii. Executing processes-co-ordinating people and other resources to carryout the plan.

iv. Controlling processes – ensuring the project objectives are met by monitoring and measuring progress required to identify variances from plan. So that corrective action can be taken when necessary

v. Closing processes – Formalizing acceptance of the project (or) phase and bringing into an orderly end.



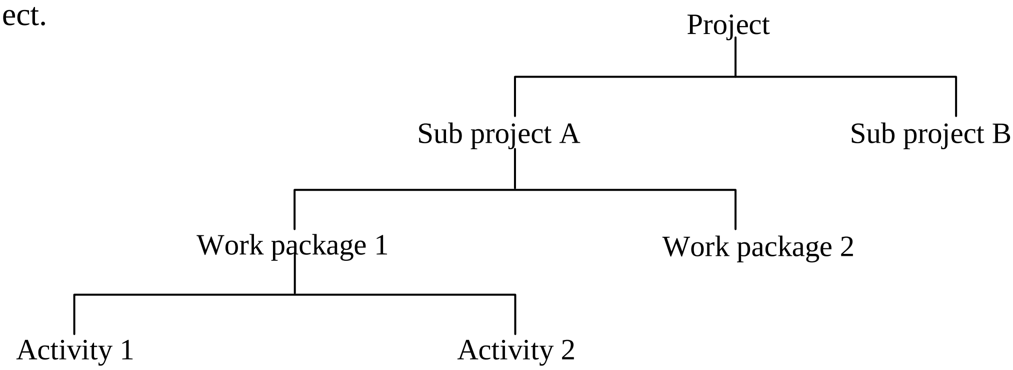
Product oriented process: These are typically defined by the project life cycle and vary by application area. Project management processes and product oriented process overlap and interact through out the project.



Work breakdown structure (WBS): It is a deliverable-oriented grouping of project components that organizes and defines the total scope of the project; work not in the WBS is out side the scope of the project.

As with the scope statement, the WBS is often used to develop or confirm a common understanding of project slope. Each descending level represents an increasingly detailed description of the project deliverables.

A WBS is normally presented in chart form and each item in the WBS is generally assigned a unique identifier. The items at the lowest level of the WBS may be referred to as work package project.



Breakeven Analysis: Breakeven Analysis is another way of performing sensitivity analysis wherein we are more concerned about finding the value (called the breakeven point) at which the reversal of decision takes place. It is in contrast to sensitivity analysis, where not much emphasis is placed on finding this breakeven value. In sensitivity analysis, we ask what will happen to the project if the invoice or billing declines, if the costs increase, or if something else happens.

However, we may also be interested in knowing how much should be produced and sold at a minimum to ensure that the project does not ‘lose money’. Such an exercise is called ‘breakeven analysis’ and the minimum quantity at which loss is avoided in called the breakeven point. The breakeven analysis is also referred to as cost – volume profit analysis.

In order to illustrate the concept of break – even analysis, we take a simple example wherein a ready – mix concrete (RMC) manufacturer wants to find out the minimum production of concrete which will just be enough to recover its total cost incurred in a particular month. The total cost (TC) incurred in a month is the sum total of its indirect cost (IC) and direct cost (DC). The indirect costs in this example are those costs that are incurred irrespective of concrete production taking place or not. However, the direct costs are proportional to the volume or quantity of production. A detailed discussion on direct and indirect costs is provided elsewhere in this text.

The total cost $TC = IC + DC = IC + n \times UDC$

The relation between DC and UDC is given by the following expression.

$$DC = n \times UDC$$

From the above expression, it is clear that UDC is the direct cost for one unit of production.

Let the sales price fixed by the RMC supplier be P per unit of concrete sold. If the quantity of concrete sold is n units, the revenue R would be computed by the expression:

$$R = n \times P$$

Gross profit Z for the period would be defined as:

$$Z = R - TC = n \times P - IC - n \times UDC = n \times (P - UDC) - IC$$

The net profit Z_1 after taking taxes into account is given by:

$$Z_1 = Z \times (1 - t)$$

Where t is the tax rate.

Breakeven point defined as one where profit equals zero. In order to determine the concrete quantity n at which the RMC seller just recovers its total cost, we equate total cost to revenue.

Thus at breakeven point, Total cost TC = Revenue R. (Note that at this point, profit Z = 0)

At breakeven, we have:

$$TC = R$$

$\Rightarrow IC + n \times UDC = n \times p$

The quantity produced at breakeven point is denoted with B

Thus,

$IC + B \times UDC = B \times P$

$$\Rightarrow B = \frac{IC}{(P - UDC)}$$

The denominator in the above expression (P – UDC) is also known as 'contribution'. For n less then B, the RMC seller is making losses, while for n greater than B, the RMC seller is making profits.

The result of breakeven analysis is shown on the breakeven chart. The chart contains direct cost line and indirect cost line besides the total cost line. It also has the revenue line. The point of intersection of the total cost and revenue line is known as breakeven point. The ordinate corresponding to this intersection point gives the breakeven quantity of concrete to be produced in order to just recover the total cost incurred by the RMC seller.

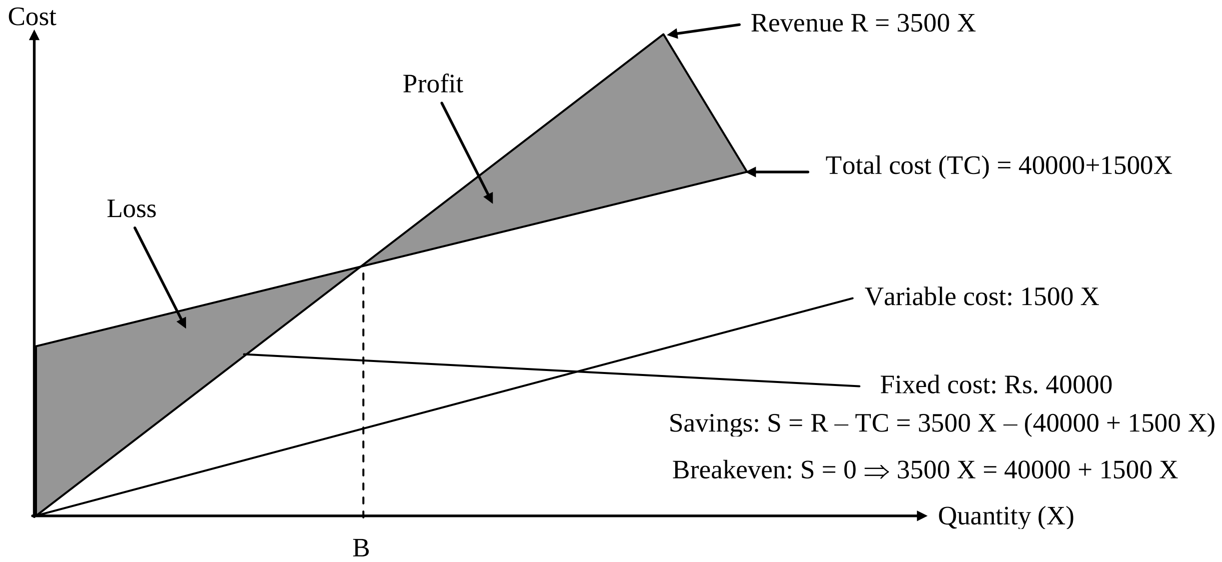
An illustration of breakeven chart is given for an example problem.

Let us assume that a ready-mix concrete company sells RMC for a price of Rs.3,500/m^3. The fixed cost of the company for the production is Rs. 40,000/month and the variable cost associated with per-m^3 production of RMC is Rs.1500.

It is desired to determine the breakeven quantity.

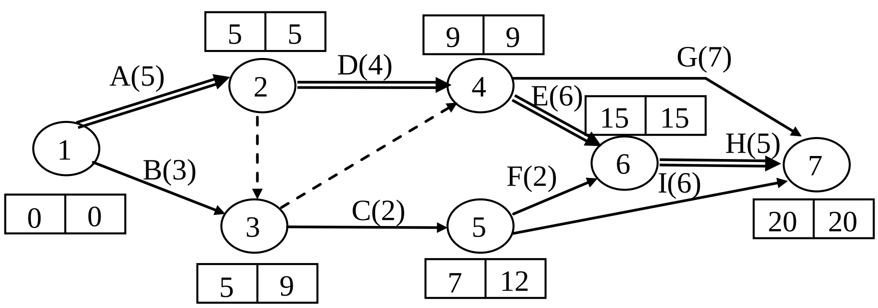
The breakeven point can be found out from the breakeven chart drawn for above example. The chart is self-explanatory.

Companies try to lower the breakeven point by resorting to different means such as (1) increasing the sales price, (2) reducing the total cost of production, or (3) increasing the quantity of production. The measures are adopted either in isolation or in combination.



06(b)(i).

Sol:



Critical path: A-D-E-H

Project completion time 20 days

Slacks:

Slack @ 1, 2, 4, 6 & 7 = 0 (Critical events)

Slack @ 3 = 9 – 5

= 4

Slack @ 5 = 12 – 7

= 5

Floats:

Float for A, D, E, & H = 0 (Critical activities)

Activity B: TF = 9 – 0 – 3 = 6

Activity C: TF = 12 – 5 – 2 = 1

Activity F : TF = 15 – 7 – 3 = 5

Activity G : TF = 20 – 9 -7 = 4

Activity I : TF = 20 – 7 – 6 = 7

06(b)(ii).

Sol: Dozing time = $\dfrac{61\,(\text{m})}{4\times\left(\dfrac{1000}{60}\right)\dfrac{\text{m}}{\text{min}}}$ = 0.915 minutes

Return time $= \dfrac{61}{8\times\dfrac{1000}{60}} = \dfrac{61}{8\times 16.7} = 0.456$ minutes

Fixed time = 0.05 minutes

Cycle time = Fixed time + Variable time

= (0.05) + (0.915 + 0.456)

C.T = 1.421 minutes

Output production of Dozer = $\left[\dfrac{\text{Blade capacity}(\text{m}^3)}{\text{cycle time}(\text{min})}\times 60\right]\dfrac{\text{m}^3}{\text{hr}}\times \text{Job efficiency}$

$= \left[\dfrac{7.65\ \text{LCM}}{1.421\ \text{min}}\times 60\right]\dfrac{\text{m}^3}{\text{hr}}\times\left(\dfrac{50}{60}\right)$ = 269.18 LCM per hour

06(b)(iii).

Sol:

i) Speed of the compactor, V = 2000 m/hr

ii) Drum width (D) = 2m

iii) Soil layer thickness compaction (t) = 0.25 m

iv) No. of passes = 6

v) Efficiency = 0.8

Output capacity of the compactor $= \left[\dfrac{\text{Speed} \times \text{Drum width} \times \text{Layer thickness}}{\text{No. of passes}}\right] \times \text{Efficiency}$

$$= \frac{(2000 \text{ meter/hour})(2 \text{ meter})(0.25 \text{ mete})}{6} \times 0.8$$

$= 133.33 \text{ m}^3/\text{hr}$

06(c).

Sol: Given:

Size = 300 mm × 600 mm (b × d)

A_{st} = 3-20mmϕ

l_e = 5 m

M20, Fe-415

Safe load = ?

1. Max. depth of N.A, $x_{u\,max} = 0.48\ d$

$= 0.48 \times 600$

$= 288$ mm

2. **Actual depth of N.A:**

C = T

$0.36\ f_{ck}\ x_u = 0.87\ f_y\ A_{st}$

$$0.36 \times 20 \times 300 x_u = 0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 20^2$$

$x_u = 157.54$ mm

$x_u < x_{u\,max}$ $\therefore$ Under reinforced Section

3. **Moment of Resistance:**

$MR = T \times Z = 0.87\ f_y\ A_{st}\ (d - 0.42 x_u)$

$$0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 20^2 (600 - 0.42 \times 157.54)$$

$= 181.65 \times 10^6$ N-mm

4. Maximum Bending Moment:

$$M_u = \frac{w_u L_e^2}{8}$$

$$= \frac{w_u \times 5^2}{8}$$

5. Safe load the beam can carry

Equate maximum Bending Moment to M.R

$$\frac{w_u \times 5^2}{8} = 181.65$$

$$w_u = \frac{181.65 \times 8}{25} = 58.128 \text{ kN/m}$$

$$w = \frac{58.128}{1.5} = 38.752 \text{ kN/m}$$

Dead load, $w_D = \gamma BD$ (assuming effective cover = 50 mm, D = 650 mm)

$$= 25 \times 0.3 \times 0.65 = 4.875 \text{ kN/m}$$

$w = w_D + w_L = 38.752$ kN/m

$w_L = 37.752 - 4.875 = 33.877$ kN/m

07(a).

Sol: Assume width of main & cover plate = B

Thickness of main plate t = 10mm

Thickness of each cover plate t_{cp}=6 mm

Direct axial pull P = 375 kN

Assume nominal diameter of rivet

$$\phi = 6.04\sqrt{t} = 6.04\sqrt{10} = 19.1 \text{ mm}$$

$$\simeq 20\text{mm}$$

Gross diameter of rivet, d = 21.5 mm

As the joint is double cover butt joint, The rivets will be in double shear strength

Shear strength of one rivet in double shear

$P_s = 2 \times \frac{\pi}{4} d^2 \times \tau_{vf} = 2 \times \frac{\pi}{4} \times (21.5)^2 \times 100$

$= 72.61 \times 10^3$ N = 72.61 kN

Strength of one rivet in bearing

$P_b = d \times t \times \sigma_{pf} = 21.5 \times 10 \times 300$

$= 64.5 \times 10^3$ N = 64.5 kN

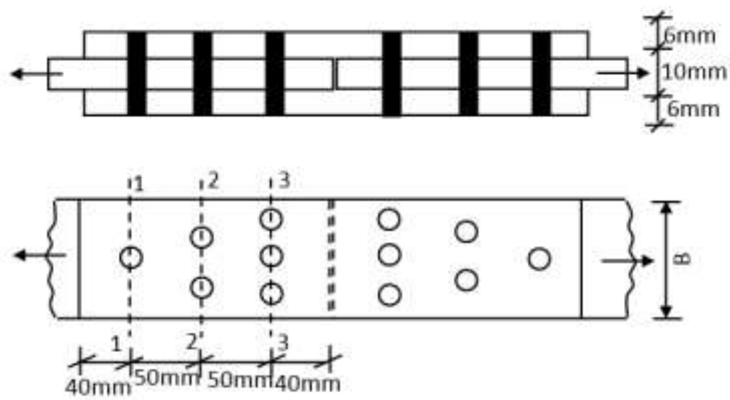
Strength of one rivet or Rivet value

R_v = Smaller of P_s and P_b = 64.5 kN

Number of rivets (n)

$$n = \frac{\text{Direct axial pull}}{\text{Rivet Value}} = \frac{P}{R_v} = \frac{375}{64.5} = 5.81 \approx 6 \text{No's}$$

Provide 6 no's of rivets to each side of the joint and arrange them using diamond pattern as shown in figure



The width of main plate and cover plate can be computed based on tensile strength of main plate and cover plate at section 1-1 and section 3-3 respectively

Tensile strength of main plate at section 1-1

$P \leq P_{t\,1\text{-}1} = A_{net} \times \sigma_{at} = (B - n \times d)\, t \times \sigma_{at}$

$P = (B - 1 \times 21.5) \times 10 \times 150$

$375 \times 10^3 = (B - 21.5) \times 1500$

Width of main plate B=271.5mm ≈ 275mm

Tensile strength of cover plate at section 3-3

$P \leq P_{ct\,3\text{-}3} = A_{net} \times \sigma_{at} = (B - nd) \times t_{cp} \times \sigma_{at}$

$= (B - 3 \times 21.5) \times (2 \times 6) \times 150$

$375\times10^3 = (B - 3 \times 21.5) \times 1800$

Width of cover plate B=272.83mm≈ 275mm

Width of main and cover plate B = 275 mm

Tensile strength of main plate at critical section 1-1

$P_{t\,1\text{-}1} = A_{net} \times \sigma_{at} = (B - n\times d)\, t \times \sigma_{at}$

$= (275 - 1 \times 21.5) \times 10 \times 150$

$= 380.25\times10^3$ N = 380.25 kN ≥ P = 375 kN

Tensile strength of cover plate at critical section 3-3

$P_{ct\,3\text{-}3} = A_{net} \times \sigma_{at} = (B - n\times d)\, t \times \sigma_{at}$

$= (275 - 3\times 21.5) \times 12 \times 150$

$= 378.9\times10^3$ N = 378.9 kN ≥ P=375 kN

Strength of rivets $P_r = n \times R_v = 6 \times 64.5 = 387$ kN≥ P=375 kN

Hence riveted connection is safe

Strength of riveted joint P_c = 378.9 kN

Strength of solid plate P_{sp}

$P_{sp} = A_g \times \sigma_{at} = (275 \times 10) \times 150$

$= 412.5 \times 10^3$ N = 412.5 kN

Efficiency of the Riveted joint $\eta = \frac{P_c}{P_{sp}} \times 100$

$$\eta = \frac{378.9}{412.5} \times 100 = 91.85\%$$

Efficiency of the Riveted joint η = 91.85%

Minimum pitch of the bolt p = 2.5 ϕ

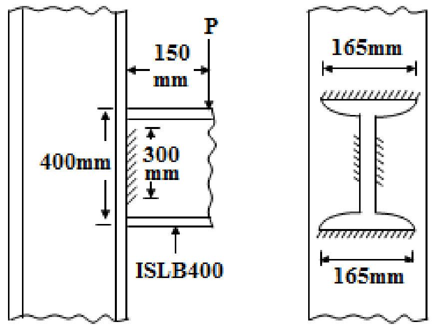
= 2.5 ×20 = 50mm ≈ 60mm

Minimum end distance e = 1.5 d

= 1.5 ×21.5 = 32.25 mm ≈ 40 mm

07(b).

Sol:



For grade Fe410 steel

$f_u = 410$ MPa & $f_y = 250$ N/mm^2

For workshop welding $\gamma_{mw} = 1.25$

Size of fillet weld S = 6 mm

Throat thickness of flange weld,

$t_t = K.S = 0.7 \times 6 = 4.2$ mm

Let 'P' the design or factored eccentric load in kN.

Eccentricity of load e =150mm

Design bending moment $M = P \times e = 150$ P N-mm

M = Pe = 200 P N-mm

Vertical shear stress in fillet weld due to P is q_1

$$q_1 = \frac{P}{\text{effective sectional area of fillet weld}}$$

$$= \frac{P}{2\times 165\times 4.2 + 2\times 300\times 4.2} = 0.000256 \text{ P N/mm}^2$$

Stress due to bending moment $f = \frac{M}{I} y_{max}$

$$y_{max} = \frac{400}{2} = 200 \text{ mm}$$

I = Moment of inertia of fillet weld about bending axis.

$$= 2\times\left[\frac{165\times 4.2^3}{12} + 165\times 4.2\times\left(\frac{400}{2}\right)^2\right] + \left[\frac{300^3\times 4.2}{12}\right]\times 2 = 74.34\times 10^6 \text{ mm}^4$$

$$f = \frac{M}{I}\cdot y_{max} = \frac{150P\times 200}{74.34\times 10^{6}} = 0.000403\,P\,N/mm^{2}$$

Equivalent stress $(f_e) = \sqrt{3q^2 + f^2}$

$$= \sqrt{(3\times 0.000256P)^2 + (0.000403P)^2}$$

$= 0.000867P$

For safety of weld $f_e \le \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}\times 1.25}$

$$\Rightarrow 0.000867\,P \le \frac{410}{\sqrt{3}\times 1.25}$$

$\Rightarrow P \le 218.42$ kN

Maximum load P = 218.42 kN

07(c).

Sol: Permissible tensile stress σ_{at} = 150 N/mm^2

Permissible shear stress in weld τ_{vf} = 108 MPa

Assume the angles are tacking welded

The maximum tensile capacity of the angles $P_t = A_{net}\times\sigma_{at}$

Net effective sectional area of double angles

A_{net} = Gross sectional area of angles = A_g-Area of rivet holes

= 2× [(125 +75 - 10) × 10] -0 =3800 mm^2

The tensile capacity of angle $P_t = A_{net}\times\sigma_{at}$

= 3800× 150 = 570× 10^3 N

= 570 kN

Load on each angle P = 570/2 = 285 kN

Design of fillet weld

Let S and L_w be the size and effective length of fillet weld respectively.

Minimum size of fillet weld S_{min} = 5 mm

Maximum size of fillet weld S_{max}= $\frac{3}{4}\times 10$ = 7.5 mm

Adopt minimum size of fillet weld S= 8 mm

Effective throat thickness t_t = K.S = 0.70×8

= 5.6 mm

Tensile load on each angle $P_t = \frac{570}{2}$

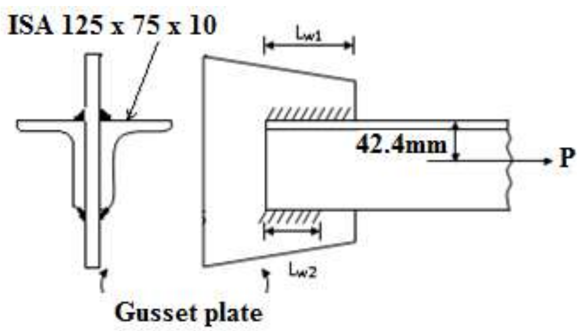
= 285 kN

To calculate effective length of fillet weld (L_w) by equating tensile load on each angle to the strength of fillet weld (P_s)

$P_t = P_s = L_w \times t_t \times \tau_{vf}$

$285\times10^3 = L_w\times5.6\times108$

L_w = 471.23 mm



The C.G of an angle located at a distance 42.4 mm from edge of an angle

Arranging required weld length on top edge and bottom edges only.

Let L_{w1} and L_{w2} are length of top and bottom edges welds only.

$L_{w1} + L_{w2} = L_w$ = 471.23 ------ (1)

Taking moments of load and weld strengths on top edge of an angle

$L_{w1} \times t_t \times\tau_{vf}\times 0 + L_{w2} \times t_t \times\tau_{vf}\times(125 - 42.4)$

$= 111.6\times10^3\times42.1$

$0 + L_{w2}\times5.6\times108\times125 = 285\times10^3\times42.1$

L_{w2} = 158.71 mm

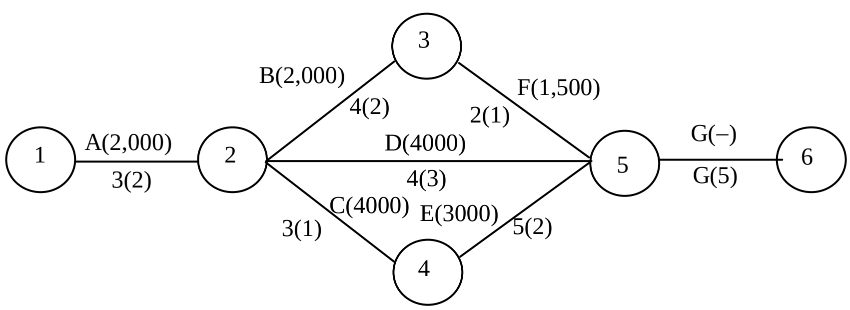
L_{w1} = 471.23–158.71

= 312.52

08(a).

Sol:

Activity	Normal		Crash		Cost slope
	Duration	Cost (Rs)	Duration	Cost (Rs)	Rs/day
(1, 2)	3	5000	2	7000	2000
(2, 3)	4	6000	2	10000	2000
(2, 4)	3	9000	1	17000	4000
(2, 5)	4	5000	3	9000	4000
(4, 5)	5	7000	2	16000	3000
(3, 5)	2	8000	1	9500	1500
(5, 6)	5	20000	5	20000	



1-2-4-5-6 is the critical path & 16 days is project duration

Total project cost = [Normal cost of activities (1, 2), (2, 3), (2, 4), (2, 5), (4, 5) and (3, 5)] + (Indirect cost per day) × Duration of the project]

= (5000 + 6000 + 9000 + 5000 + 7000 + 8000 + 20000) + (6000 × 16)

= (60000) + (96000) = 156000.

The activity (1, 2) is on the critical path and has the least slope (Rs. 2000/day), and hence, can be crashed first. This activity can be crashed by one day; thus, the project duration reduces by a day. Project duration has become 15 days now.

The project cost to complete in 15 days

= (cost to compete the project in 16 days) + (cost of crashing by a day) – (saving in indirect cost)

= 156000 + 2000 – 6000 = 152000

Therefore obtained a reduction of Rs. 4000/- and a reduction in duration by 1 day.

The next higher cost slope (Rs. 3000 per day) on the critical path is for activity (4, 5) and crash this activity to a maximum of 3 days (from a normal duration of 5 days to a crash duration of 2 days).

Crash it in two steps of one day each.

With one day of crashing, the project duration will become 14 days now.

The project cost to complete in 14 days

= the project cost to complete in 15 days + cost of crashing by a day – saving in indirect cost

= 152000 + 3000 – 6000

= 149000

Options available for further crashing

Option	**Cost (Rs/day)**
C and B	4000 + 200 = 6000
C and F	4000 + 3000 = 7000
E and B	3000 + 2000 = 5000
E and F	3000 + 1500 = 4500

The project duration, thus, reduces by another day and the cost has also decreased by Rs. 3000 over the previous crash cost.

The activity (4, 5) is crashed again by a day. The project duration becomes 13 days now.

The project cost to complete in 13 days.

= The project cost to complete in 14 days + cost of crashing by a day – saving in indirect cost

= 149000 + 3000 – 6000

= 146000

The project duration, thus reduces by another day and the cost has also decreased by Rs. 3000 over the previous crash cost.

There is still scope of crashing activity (4, 5). However, note that there are two critical paths 1-2-4-5-6 and 1-2-3-5-6, both of 13 days duration. The available options for crashing are given in Table. It may further be noted that the cost of crashing one day is the summation of individual cost slopes.

The lowest-cost slope option is given by activities E and F. The crash costs of these two activities combined together is Rs.4500. Thus, we crash activities E and F by 1 day, which makes the project duration equal to 12 days.

The project cost to complete in 12 days

= The project cost to complete in 13 days + cost of crashing by a day – saving in indirect cost

= 146000 + 4500 – 6000

= 144500

The project duration, thus, reduces by another day and the cost has also decreased by Rs.1500 over the previous crash cost.

Again, we have two critical paths 1-2-4-5-6 and 1-2-3-5-6, both with project duration of 12 days. The available option for crashing is activities C and B together. The crash cost would be equal to 4000 + 2000 = 6000. The project duration reduces by another day and becomes equal to 11 days.

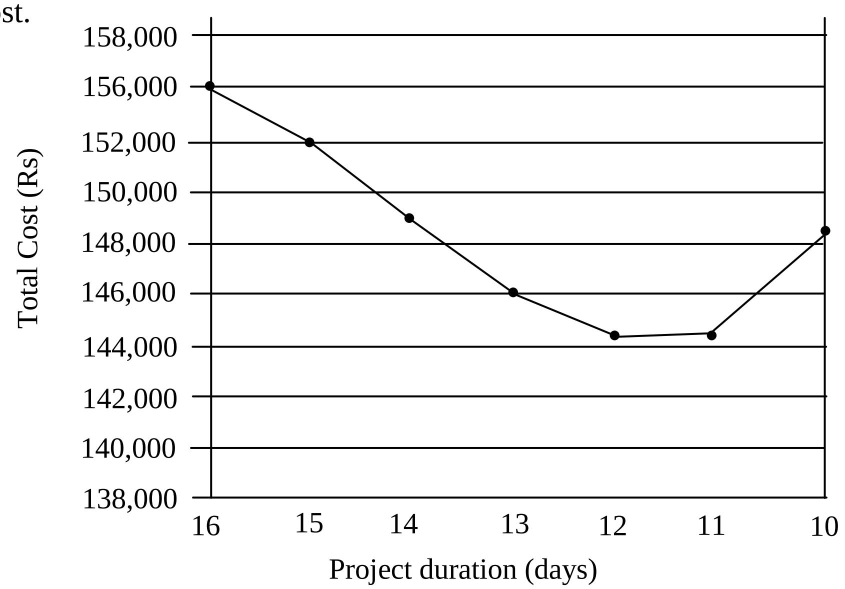
The project cost to complete in 11 days

= The project cost to complete in 12 days + cost of crashing by a day – saving in indirect cost

= 144500 + 6000 – 6000

= 144500

The project duration reduces by another day but there is no increase or decrease in the cost over the previous crash cost.



Time Vs Cost

Now, we observe the we have 3 critical paths, 1-2-4-5-6, 1-2-3-5-6 and 1-2-5-6, each of 11 days duration. Out only available option for crashing is to crash activities B, C and D together by one day, and the cost of crashing the three activities together is equal to the sum of cost slopes of activities (2, 3), (2, 4) and (2, 5) – that is, Rs. 10000. Thus, project duration becomes 10 days.

The project cost to complete in 10 days

= The project cost to complete in 11 days + cost of crashing by a day – saving in indirect cost

= 144500 + 10000 – 6000

= 148500

The project duration reduces by another day but the cost has increased by Rs. 4000 over the previous crash cost.

Thus, we have reached a stage where the decrease in duration is accompanied by a significant increase in the direct cost, forcing us to stop further crashing. If we combine all our results in a graph showing how project length affects the schedule costs, we obtain the curve as in Figure, which shows the minimum project cost corresponding to project durations of 11 days and 12 days.

08(b).

Sol:

1. Design Constants:

For M25 concrete and Fe-415 steel

σ_{cbc} = 8.5 N/mm^2

σ_{st} = 230N/mm^2 $\rightarrow$ in Bending tension

σ_{st} = 150 N/mm^2 $\rightarrow$ in Direct tension

$$\text{Modular ratio, } m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times 8.5}$$

$$= 10.98 \simeq 11$$

$$K = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{11\times 8.5}{11\times 8.5 + 230} = 0.289$$

$$J = 1 - \frac{K}{3} = 1 - \frac{0.289}{3} = 0.904$$

$$Q = \frac{1}{2} \times \sigma_{cbc} JK = \frac{1}{2} \times 8.5 \times 0.904 \times 0.289$$

$$= 1.11$$

$$\frac{L}{B} = \frac{6}{4} = 1.5 < 2$$

Hence both long and short wall resist the load by cantilever action for height h = 1 m and by horizontal action resist the load in the top H – h. In such water tanks, moment due to horizontal action is considerable and it governs the selection of thickness of walls.

2. Horizontal Frame Action:

$P_h = \gamma(H - h) = 9.8\ (3 - 1) = 19.6\ \text{kN/m}^2$

Fixed end moments:

$$\frac{P_h L^2}{12} = \frac{19.6 \times 6^2}{12} = 58.8\ \text{kN-m} \rightarrow \text{In long wall}$$

$$\frac{P_h B^2}{12} = \frac{19.6 \times 4^2}{12} = 26.13\ \text{kN-m} \rightarrow \text{In short wall}$$

S.No	Member	Stiffness	Total stiffness	D.F
1.	Short wall	$\frac{4EI}{4} = EI$	$\frac{5}{3}EI$	$\frac{3}{5}$
2.	Long wall	$\frac{4EI}{6} = \frac{2}{3}EI$		$\frac{2}{5}$

Shortwall	Longwall
3/5	2/5
+26.13	–58.8
+19.6	+13.06
+45.73	–45.74

Corner moment = M_c = 45.73 kN-m

Thickness of wall required, $d = \sqrt{\dfrac{M}{Qb}}$

$$= \sqrt{\frac{45.73\times10^6}{1.11\times1000}} = 202.97 \text{ mm}$$

Provide D = 250 mm

d = 250 – 30 = 220 mm

Eccentricity, $e = \dfrac{250}{2} - 30 = 95 \text{ mm}$

Direct pull on Longwall and Shortwall

$$T_L = P_h\frac{B}{2} = 19.6\times\frac{4}{2} = 39.2 \text{ kN}$$

$$T_B = P_h\frac{L}{2} = 19.6\times\frac{6}{2} = 58.8 \text{ kN}$$

3. Design of Longwall:

At Corner

Design moment, M = $M_c - T_L.x$

= 45.73 – 39.2 × 0.095

= 42 kN-m

$$A_{st_1} = \frac{M}{\sigma_{st}Jd} = \frac{42\times10^6}{230\times0.904\times220}$$

= 918.185 mm^2

$$A_{st_2} = \frac{T_L}{\sigma_{st}} = \frac{39.2\times10^3}{150} = 261.33 \text{ mm}^2$$

$$A_{st} = A_{st_1} + A_{st_2} = 1179.52 \text{ mm}^2$$

Spacing, Use 16ϕ

$$\frac{1000\times\frac{\pi}{4}\times16^2}{1179.52} = 170.4 \text{ mm}$$

Provide 16 mmϕ @ 170 mm c/c

At Middle:

$$B.M = \frac{P_h L^2}{8} - M_c$$

$$= \frac{19.6 \times 6^2}{8} - 45.73 = 42.47 \text{ kN-m}$$

Design moment = $M - T_L.x$

$= 42.47 - 39.2 \times 0.095$

$= 38.746$ kN-m

$$A_{st_1} = \frac{\text{Design moment}}{\sigma_{st} Jd} = \frac{38.746 \times 10^6}{230 \times 0.904 \times 195}$$

$= 955.64 \text{ mm}^2$

$$A_{st_2} = \frac{T_L}{\sigma_{st}} = 261.33 \text{ mm}^2$$

$$A_{st} = A_{st_1} + A_{st_2} = 1216.97 \text{ mm}^2$$

Spacing: use 16 ϕ

$$S = \frac{1000 \times \frac{\pi}{4} \times 16^2}{1216.97} = 165.16 \text{ mm}$$

Provide 16mm @ 160 mm c/c

Reinforcement for Shortwall:

At Corner:

$m = M_c - T_B.x = 45.73 - 58.8 \times 0.095$

$= 40.14$ kN-m

$$A_{st_1} = \frac{40.14 \times 10^6}{230 \times 0.904 \times 195}$$

$= 990.03 \text{ mm}^2$

$$A_{st_2} = \frac{58.8 \times 10^3}{150} = 392 \text{ mm}^2$$

$$A_{st} = A_{st_1} + A_{st_2} = 1382.03 \text{ mm}^2$$

$$S = \frac{1000 \times \frac{\pi}{4} \times 16^2}{1382.03} = 145.4 \text{ mm}$$

Provide 16 mmϕ @ 140 mm c/c

At Middle:

$$m = \frac{\gamma(H-h)B^2}{8} - \text{moments at ends}$$

$$= \frac{9.8(3-1)\times 4^2}{8} - 45.73 = -6.52 \text{ kN-m}$$

It is very small moment. Hence provide minimum steel.

Reinforcement in Vertical Direction:

Cantilever Moment $= \gamma H \frac{h^2}{6} = 9.8 \times 3 \times \frac{1^2}{6} = 4.9$ kN-m

$$A_{st} = \frac{M}{\sigma_{st} Jd} = \frac{4.9 \times 10^6}{230 \times 0.904 \times 195} = 120.85 \text{ mm}^2$$

Provide minimum steel.

08(c).

Sol: Given:

Size of room = 4.0 m × 9.5 m

Wall thickness ‘b’ = 230 mm

Live load, w_L = 4 kN/m^2

Floor Finish = 1 kN/m^2

M20, Fe-415

Design the slab

1. Assume the effective depth from serviceability criteria

$$d = \frac{\ell}{20} = \frac{4000}{20} = 200 \text{ mm}$$

$$D = 200 + 20 + \frac{10}{2} = 225 \text{ mm}$$

2. Effective Span (L_e) :

L_{ex}

$$\left.\begin{array}{l} \text{i) } \ell_x + b_s = 4 + 0.23 = 4.23 \\ \text{ii) } \ell_x + d = 4 + 0.2 = 4.2 \end{array}\right\} \text{Smaller}$$

L_{ey}

$$\left.\begin{array}{l} \text{i) } \ell_y + b_s = 9.5 + 0.23 = 9.73 \\ \text{ii) } \ell_y + d = 9.5 + 0.2 = 9.7 \end{array}\right\} \text{Smaller}$$

Provided l_{ex} = 4.2 m and l_{ey} = 9.7 m

$$\frac{\ell_{ey}}{\ell_{ex}} = \frac{9.7}{4.2} = 2.3 > 2$$

$\therefore$ Designed as one way slab

3. Loads:

Self weight of slab = 0.225 × 25

= 5.625 kN/m

Imposed load, w_L = 4 kN/m^2

Floor finish = 1 kN/m^2

Total load, w = 10.625 kN/m^2

Factored load, w_u = 1.5 × 10.625 = 15.9375 kN/m^2

Factored Bending moment (per m run)

$$M_u = \frac{w_u . L_{ex}{}^2}{8} = \frac{15.9375 \times 4.2^2}{8}$$

= 35.14 kN-m

4. Checking the Effective Depth:

$$d = \sqrt{\frac{M_u}{R_u . b}} = \sqrt{\frac{35.14 \times 10^6}{0.138 \times 20 \times 1000}}$$

= 112.84 mm < 200

$\therefore$ O.K

However provided

$D = 125 + 20 + \frac{10}{2} = 150$ mm

d = 125 mm

5. Area of tension steel in shorter span (Main steel):

$$M_u = 0.87 f_y A_{st}\left[d - \frac{0.42\times 0.87 f_y A_{st}}{0.36 f_{ck} b}\right]$$

$$35.14\times 10^6 = 0.87\times 415\times A_{st}\times 125\left[1 - \frac{A_{st}}{1000\times 125}\times\frac{415}{20}\right]$$

$35.14\times 10^6 = 45.13\times 10^3 A_{st} - 7.49 A_{st}^2$

$A_{st} = 918.7\ mm^2$

6. Minimum Steel:

0.12% of bD

$$= \frac{0.12}{100}\times 1000\times 150 = 180\ mm^2 < A_{st}$$

$\therefore$ O.K

7. Maximum Steel:

$\ngtr$ 4% of bD

$$= \frac{4}{100}\times 1000\times 150 = 6000\ mm^2 > A_{st}$$

$\therefore$ O.K

The required steel is in between minimum and maximum steel hence safe.

Spacing, use 10 mm ϕ bars

$$S = \frac{1000 a_{st}}{A_{st}} = \frac{1000\times\frac{\pi}{4}\times 10^2}{918.7}$$

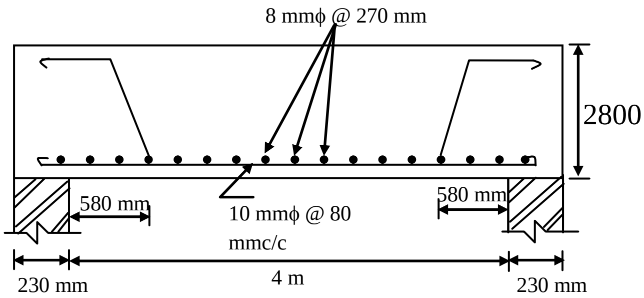
$= 85.49$ mm

Provided 10 mm ϕ @ 80 mm c/c

8. Distribution Steel:

0.12% of bD = 180 mm^2

$$S = \frac{1000 \times \frac{\pi}{4} \times 8^2}{180} = 279.25 \text{ mm}$$



Provide 8 mm bars @ 270 mm c/c