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ESE – 2019 MAINS
OFFLINE TEST SERIES

CIVIL ENGINEERING

TEST – 9
SOLUTIONS

All Queries related to **ESE – 2019 MAINS Test Series** Solutions are to be sent to the following email address
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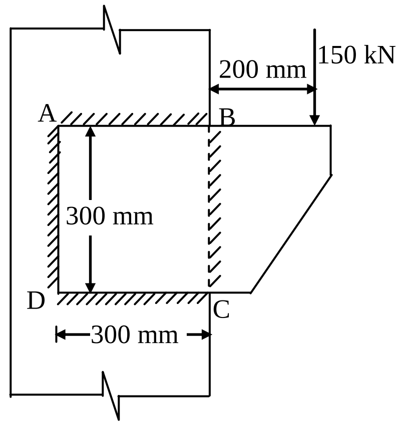
01(a).

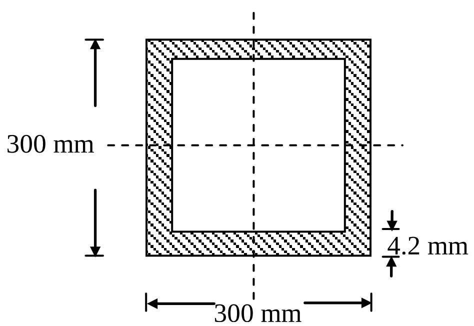
Sol:



Given that size of weld S = 6 mm

Throat thickness, $t_t = 0.7\ S = 0.7 \times 6 = 4.2$ mm

Given connection load is lying in the plane of fillet weld group.



Since the welded area is symmetric about both axes, C.G lies at center.

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

Moment of inertia about x-x and y-y axes.

$$I_{xx} = I_{yy} = 2 \times \frac{t \times 300^3}{12} + \left[\frac{300 \times t^3}{12} + 300 \times t \times 150^2\right] \times 2$$

$$= \frac{2 \times 4.2 \times 300^3}{12} + \left[\frac{300 \times 4.2^3}{12} + 300 \times 4.2 \times 150^2\right] \times 2$$

$= 75.6 \times 10^6\ mm^4$

$\therefore$ Polar moment of inertia of the weld

$I_P = I_{xx} - I_{yy}$

$I_p = 151.2 \times 10^6 \text{ mm}^4$

Twisting moment

$$M = P.e = 150 \times \left(200 + \frac{300}{2}\right) = 52500 \text{ kN} - \text{mm}$$

Vertical shear stress in weld due to P

$$q_1 = \frac{P}{\text{Effective sectional area of the weld}}$$

$$q_1 = \frac{150 \times 10^3}{4 \times 300 \times 4.2} = 29.76 \text{ N/mm}^2$$

Shear stress in weld due to twisting moment at heavily stressed point B due to M

$$q_2 = \frac{M}{I_p} \times r_{max} = \frac{52.5 \times 10^6 \times \sqrt{150^2 + 150^2}}{151.2 \times 10^6} = 73.65 \text{ N/mm}^2$$

$$\tan\theta = \frac{150}{150} = 1 \Rightarrow \theta = 45^o$$

Maximum resultant shear stress $(q_x)_{max}$

$$(q_R)_{max} = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$$

$$= \sqrt{29.76^2 + 73.65^2 + 2 \times 29.76 \times 73.65 \times \cos 45}$$

$(q_R)_{max} = 97 \text{ N/mm}^2$

The maximum stress occurs at B and its value is $(q_R)_{max} = 97 \text{ N/mm}^2$.

01(b).

Sol:

The five common defects that occur in timber are as follows:

(I) Defects due to conversion.

(II) Defects due to fungi.

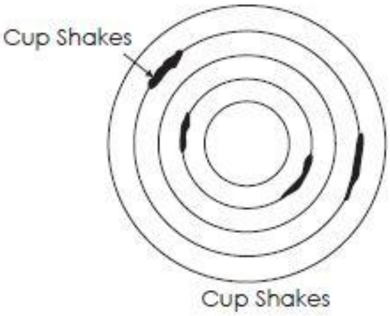
(III) Defects due to insects.

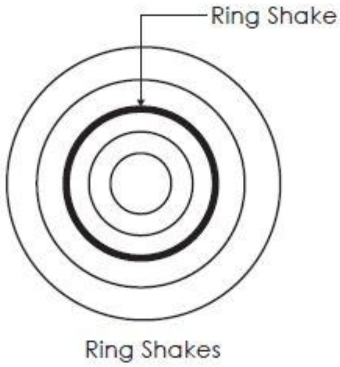
(IV) Defects due to natural forces.

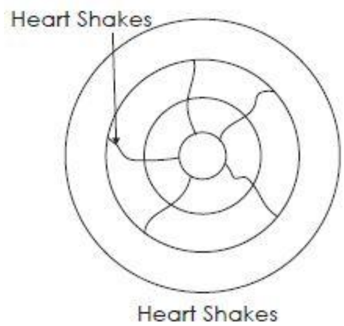
(V) Defects due to seasoning.

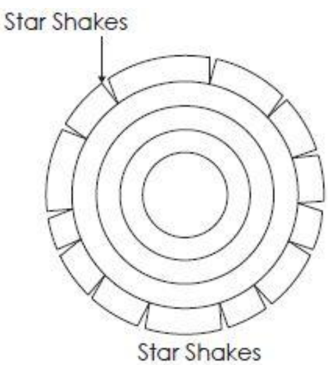
The different types of defects in timber due to natural forces are as follows:

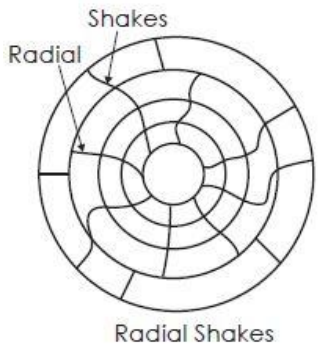
Shakes:



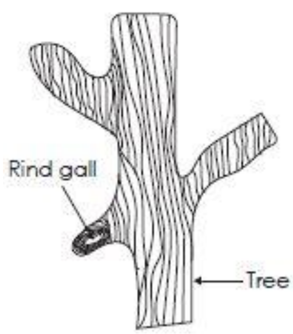




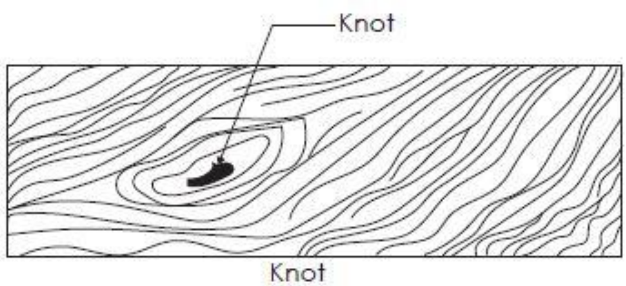




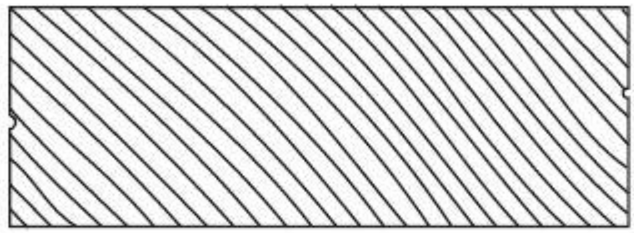
Ring galls:



Knots:

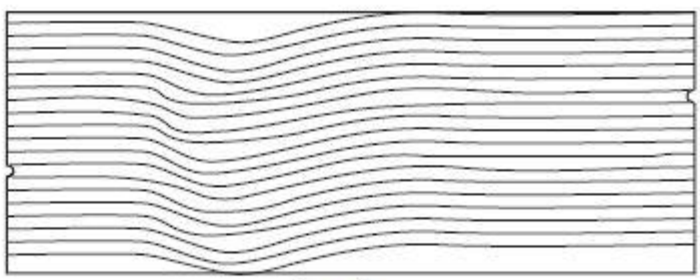


Twisted fibres:



Twisted fibres

Upsets:



Upset

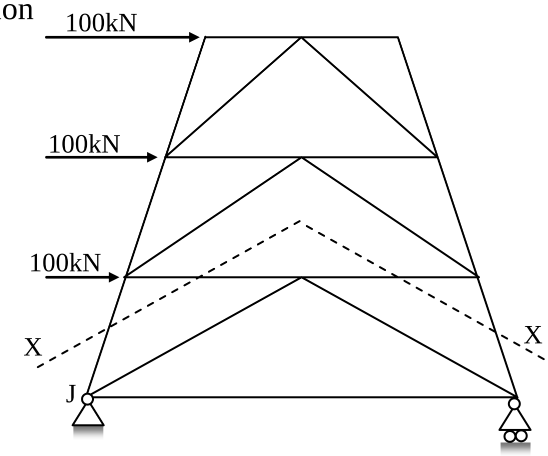
01(c). (i)

Sol: Using method by section

Taking section x – x as shown

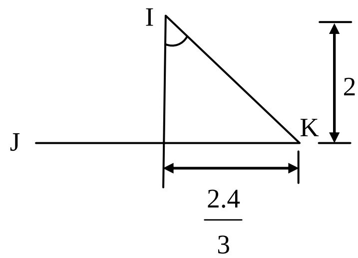
Consider top side of the section

Apply $\Sigma M_G = 0$



The vertical component of force IK will cause moment about G and horizontal component of force IK will pass through G and not cause any moment.

At joint I



$$\tan\theta = \frac{2.4/3}{2.7}$$

$$\theta = 16.5^o$$

Assume force in member IK as compressive $\Sigma M_G = 0$

$$F_{IK}\cos 16.5\left(7.5 - \frac{2.4}{3} - \frac{2.4}{3}\right) = 100 \times 2.7 + 100 \times 5.4$$

$$F_{IK} = \frac{810}{5.657}$$

$$= 143.19 \text{ kN (compressive)}$$

01(c). (ii)

Sol: Deflection at centre is given by

$$\delta = \frac{WL^3}{192EI}$$

$$\text{Stiffness K } = \frac{192EI}{L^3}$$

$$W = \sqrt{\frac{kg}{W}}$$

$$W = \frac{192EI}{L^3 \times W} g$$

01(d).

Sol: Apply unit horizontal load at B and calculate all member force (K).

Joint B :

$\Sigma H = 0$

$F_{DB} \cos \theta_2 + F_{CB} \cos\theta_1 = 1$

$\Sigma V = 0$

$F_{DB} \sin\theta_2 = F_{BC} \sin \theta_1$

$$\cos\theta_1 = \frac{6}{6\sqrt{2}}$$

$$\sin \theta_1 = \frac{6}{6\sqrt{2}}$$

$$\cos\theta_2 = \frac{6}{10}$$

$$\sin \theta_2 = \frac{8}{10}$$

$$F_{DB} \times \frac{8}{10} = F_{BC} \times \frac{6}{6\sqrt{2}}$$

$$F_{BC} = 0.8\sqrt{2}F_{DB}$$

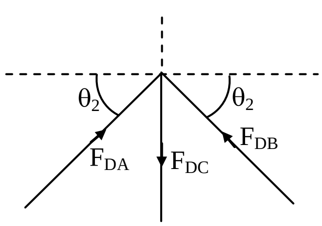
$$-\ F_{DB} \times \frac{6}{10} + 0.8\sqrt{2}F_{DB} \times \frac{6}{6\sqrt{2}} = 1$$

$-\ 0.6\ F_{DB} + 0.8\ F_{DB} = 1$

$F_{DB} = 5$(compressive)

$F_{BC} = 4\sqrt{2}$(Tension)

At Joint D :



$\Sigma H = 0$

$F_{DA} = F_{DB} = 5$(compressive)

$\Sigma V = 0$

$2F_{DA} \sin\theta_2 = F_{DC}$

$F_{DC} = 8$(Tension)

At Joint C :

$\Sigma H = 0$

$F_{AC} \cos\theta_1 = F_{BC} \cos\theta_1$

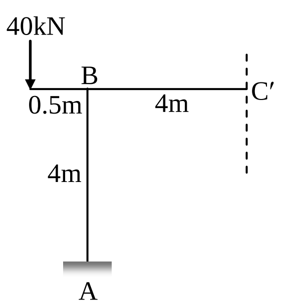
$F_{AC} = F_{BC} = 4\sqrt{2}$(Tension)

Member	K	$\delta = l\,\alpha\, t$(mm)	kδ
AD	– 5	1.1	–5.5
AC	$4\sqrt{2}$	0.93	5.26
DC	8	0.22	1.76
BD	–5	1.1	–5.5
BC	$4\sqrt{2}$	0.93	5.26
			1.28

Horizontal movement at roller B = 1.28 mm

01(e).

Sol: Take advantage of symmetry in analysis.



Step 1 : Fixed end moments

$\overline{M}_{AB} = \overline{M}_{BA} = \overline{M}_{BC} = \overline{M}_{CB} = 0$

Moment joint B = 40 × 0.5 = 20 kNm

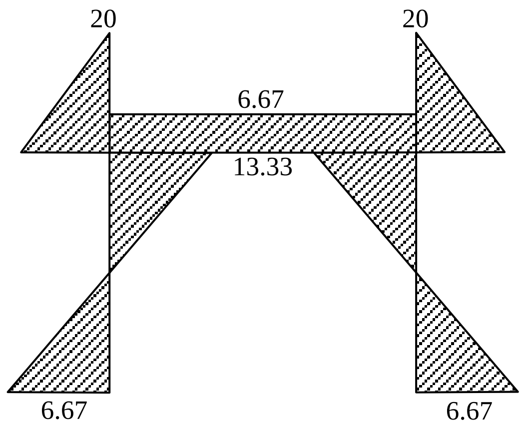
Step 2: Distribution factors

Joint	Member	Relative stiffness	Total Stiffness	Distribution factors
B	BA	$\frac{4EI}{4}$	$\frac{3EI}{2}$	$\frac{2}{3}$
	BC′	$\frac{1}{2}\left(\frac{4EI}{4}\right)$		$\frac{1}{3}$

Step 3: Non Sway Analysis

	A	B	B	
		$\frac{2}{3}$	$\frac{1}{3}$	
FEM	0	0	0	+ 20
Balancing COM		−13.33	− 6.67	
	− 6.67			
Non sway moments (kNm)	−6.67	−13.33	−6.67	20

Bending Moment Diagram (kNm)



02(a).

Sol:

Step-1: Fixed end moments

$$\overline{M}_{AB} = \frac{-36\times 4\times 2^2}{6^2} = -16\,\text{kNm}$$

$$\overline{M}_{BA} = 32\,\text{kNm}$$

$$\overline{M}_{BC} = \overline{M}_{CB} = 0$$

Step-2: Slope deflection equations

$$M_{AB} = -16 + \frac{2EI}{6}(\theta_B)$$

$$M_{BA} = 32 + \frac{2EI}{6}(2\theta_B)$$

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

$$M_{CB} = \frac{2EI}{3}\left(2\theta_C + \theta_B - \frac{3\delta_C}{3}\right)$$

Step-3: Joint Equilibrium condition at joint B

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

$$32 + \frac{2EI}{6}(2\theta_B) + \frac{2EI}{3}\left[2\theta_B + \theta_C - \frac{3\delta_C}{3}\right] = 0$$

$$3\theta_B + \theta_C - \delta_C = \frac{-48}{EI}$$

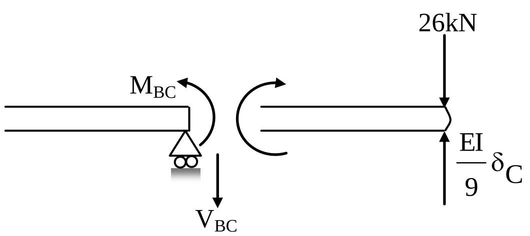
At joint C :

$$M_{CB} = 0$$

$$\frac{2EI}{3}\left(2\theta_C + \theta_B - \frac{3\delta_C}{3}\right) = 0$$

$$\delta_C = \theta_B + 2\theta_C$$

Swear conduction:



$\Sigma M_B = 0$

$$\frac{EI\delta_C}{9} \times 3 - 26 \times 3 - M_{BC} = 0$$

Substitute M_{BC}

$$\frac{EI\delta_C}{9} \times 3 - 26 \times 3 - \frac{2EI}{3}\left(2\theta_B + \theta_C - \frac{3\delta_C}{3}\right) = 0$$

$$4\theta_B + 2\theta_C - 3\delta_C = \frac{-234}{EI}$$

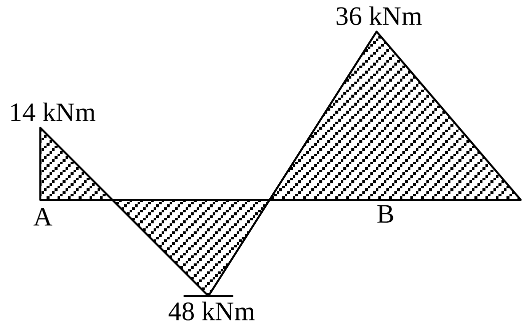
Solving the above equation we get

$$\theta_B = \frac{6}{EI},\ \theta_C = \frac{60}{EI},\quad \delta_C = \frac{126}{EI}$$

Step-4: Bending moment calculations

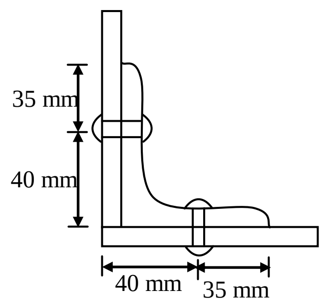
$M_{AB} = -14$, $M_{BA} = 36$, $M_{BC} = -36$, $M_{CB} = 0$

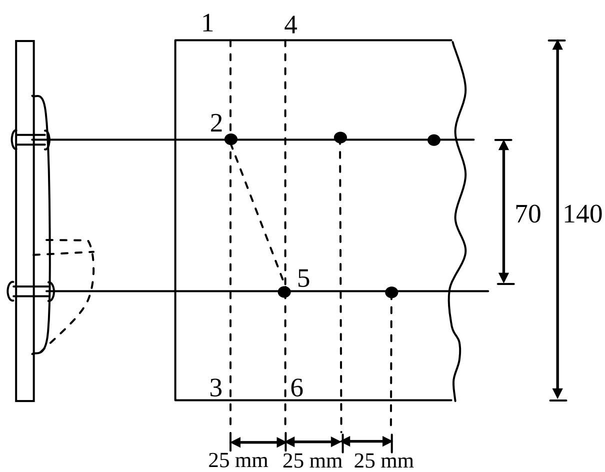
Step: BMD



02(b).

Sol: An angle section with gusset plate may be shown in the sketch below.





Nominal diameter of rivet $\phi = 16$ mm

Gross diameter of rivet $d = 16 + 1.5 = 17.5$ mm

Net sectional area of an angle along section A_{net} for section 1 – 2 – 3 (chain riveting)

$$= (140 - 17.5) \times 10 = 1225 \text{ mm}^2$$

A_{net} along section 1 – 2 – 5 – 6 (staggered riveting)

$$A_{net} = (B - nd) \times t + \frac{P^2 t}{4g}$$

$$= (140 - 2 \times 17.5) \times 10 + \frac{25^2 \times 10}{4 \times 70}$$

$$= 1072.32 \text{ mm}^2$$

Hence critical net sectional area

$A_{net} = 1072.32 \text{ mm}^2$

(ii) Maximum tensile force taken by tension member $P_t = A_{net} \times \sigma_{st}$

$$= 1072.32 \times 150 = 160.848 \times 10^3 \text{ N}$$

$$P_t = 160.85 \text{ kN}$$

02(c). (i)

Sol:

Manufacturing process of glass essentially consists of following five steps.

1. Collection of raw material
2. Batch Preparation.
3. Melting
4. Fabrication
5. Annealing

The different types of Glass used in civil engineering works:

Bullet proof glass: It is made up of several alternate layers of plate glass and vinyl resin plastic. The thickness of outer layers is small as compared to inner layers.

Fiber glass: Fiber glass consists of minute glass rods (fibers) made up of the parent material itself. It soft and flexible.

Float glass: It is extensively used in residential and commercial buildings. It is made by floating the molten glass out of furnace over molten tin. It is superior to ordinary glass in terms of energy consumption, cost effectiveness, appearance and strength.

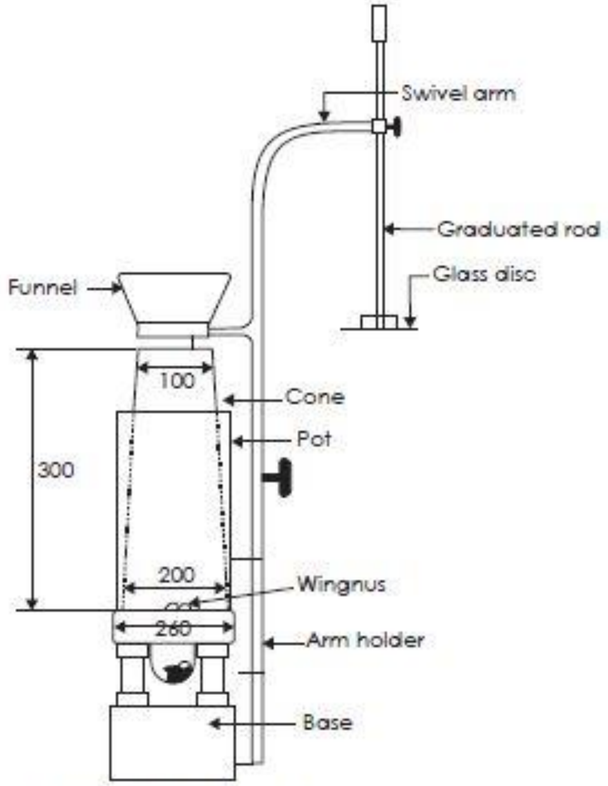
Safety glass: It is also called shatter proof glass. It is extensively used in automobile glass. The glass does not actually breaks but cracks therefore preventing damage from flying splinters, pieces of glass. It is formed by placing a celluloid between two sheets of plate glass and gluing together to make a unit.
Ultra violet ray glass: This glass effectively transmits almost all of the ultra-violet rays incident on the glass irrespective of the angle of incidence.

02(c). (ii)
Sol:

As per IS: 6461 (Part VII) – 1973, workability is defined as that property of a freshly mixed concrete which determines the ease and homogeneity with which it can be mixed, placed, compacted and finished. In other words, workability is the amount of work needed to produce 100% compacted concrete.
To determine the workability of concrete which has very less workability is Vee-Bee Consistometer test

Vee-Bee Consistometer Test:
In this method the workability of concrete is assessed based on the time required for transforming a concrete specimen in the shape of a conical frustum into a cylinder.
The apparatus consists of a vibrating table, a metal pot, Slump cone and standard tamping rod.
The concrete specimen is placed in the slump cone in a manner similar to that in the slump cone test.
The slump cone is placed inside the cylinder of the vibrating table.
The cone is then lifted up and the electric vibrator is switched on and the concrete is allowed to spread into the cylinder.
The time taken for the concrete specimen to take the shape of the cylinder is noted down.

The consistency of the concrete is expressed in VB degree which is equal to the recorded time in seconds.



Vee-Bee Consistometer

03(a). (i)

Sol:

The rebound hammer test is one of the commonly adopted test for measuring the surface hardness of concrete. This rebound hammer has a weight of around 2 kgs and has an impact energy of 2.2 J. When the plunger of the rebound hammer is pushed against the concrete surface, the plunger retracts and the spring controlled hammer mass slides within the tubular casing. The hammer is released when the spring is fully tensed, causing impact against the concrete through the plunger. When the spring controlled mass rebounds, it takes with it a rider which slides along a graduated scale and is visible through a small window in the side of the casing. The equipment can be operated horizontally or vertically. The measurement taken on the scale is an arbitrary one which is referred to as rebound number. A calibration curve relating the compressive strength of the concrete with the rebound number is plotted. The test is suitable for concrete having strength in the range of 20 to 60 MPa. The concrete surface must be smooth, clean and dry. Loose material should be removed from the surface. This test gives a measure of relative hardness of the test zone. It provides useful information about the surface layer of concrete upto 30mm deep.

03(a). (ii)

Sol:

Vegetation and Organic matter: Organic matter if present in brick earth will produce porous bricks. This is due to the evolution of gas during the burning of the organic matter, resulting in the formation of small potes.

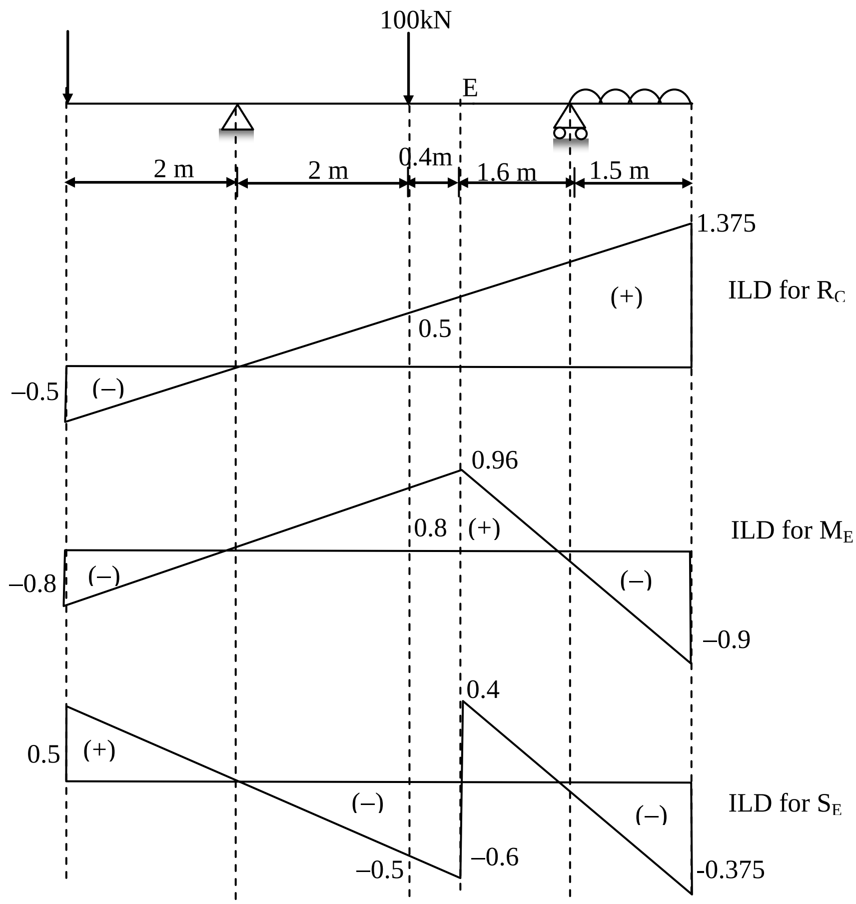
Pebbles of stone and gravel: These don't allow the clay to be mixed uniformly and thoroughly and result in weak and porous brick. Bricks containing grits are likely to crack and cannot be readily cut or worked.

Alkaline salts: Alkaline salts, if present, act as hydroscopic substances. They absorb moisture from the atmosphere in due course of time and create damp conditions. The moisture on drying leaves behind a greyish white deposit known as efflorescene.

Limestone and Kankar: Presence of large quantities of lime and limestone in limps is detrimental to brick earth, as lumps of limestone if burnt along with the brick, slake afterwards and split the brick.

03(b)(i).

Sol:



03(b)(ii).

Sol: Size of matrix = 3 × 3

1st column of stiffness matrix

K_{11} = Force generated in direction (1) due to unit loud at (1)

$$= \frac{12EI}{\ell^3} + \frac{12EI}{\ell^3} = \frac{24EI}{\ell^3}$$

K_{21} = Moment generated in direction (2) due to unit load at (1)

$$= \frac{-6EI}{\ell^2}$$

K_{31} = Moment generated in direction (3) due to unit load at (1)

$$= \frac{-6EI}{\ell^2}$$

Similarly 2nd Column of stiffness matrix

$$K_{12} = \frac{-6EI}{\ell^2}$$

$$K_{22} = \frac{4EI}{\ell} + \frac{4EI}{\ell} = \frac{8EI}{\ell}$$

$$K_{32} = \frac{2EI}{\ell}$$

3rd Column of stiffness matrix:

$$K_{13} = \frac{-6EI}{\ell^2}$$

$$K_{23} = \frac{2EI}{\ell}$$

$$K_{33} = \frac{4EI}{\ell} + \frac{4EI}{\ell} = \frac{8EI}{\ell}$$

Stiffness matrix:

$$[K] = \begin{bmatrix} \frac{24EI}{\ell^3} & \frac{-6EI}{\ell^2} & \frac{-6EI}{\ell^2} \\ \frac{-6EI}{\ell^2} & \frac{8EI}{\ell} & \frac{2EI}{\ell} \\ \frac{-6EI}{\ell^2} & \frac{2EI}{\ell} & \frac{8EI}{\ell} \end{bmatrix}$$

03(c).

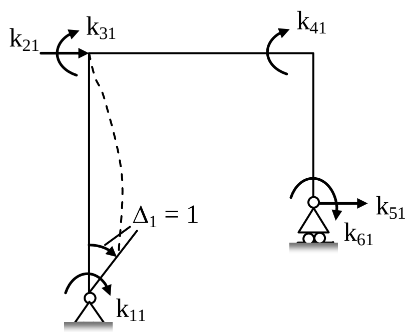
Sol: The stiffness matrix can be developed by giving a unit displacement successively at coordinates 1 to 6 and determining the forces required at all the coordinates. To generate the first column of the stiffness matrix, give a unit displacement 1 without any displacement at other coordinates as shown in figure. Compute the forces at all the coordinates using equation

$$k_{11} = \frac{4E(4I)}{10} = 1.600EI$$

$$k_{21} = -\frac{6E(4I)}{10^2} = -0.240EI$$

$$k_{31} = \frac{2E(4I)}{10} = 0.800EI$$

$$k_{41} = k_{51} = k_{61} = 0$$



To generate the second column of the stiffness matrix, give a unit displacement at coordinate 2 without any displacement at other coordinates as shown in figure

$$k_{12} = -\frac{6E(4I)}{10^2} = -0.240EI$$

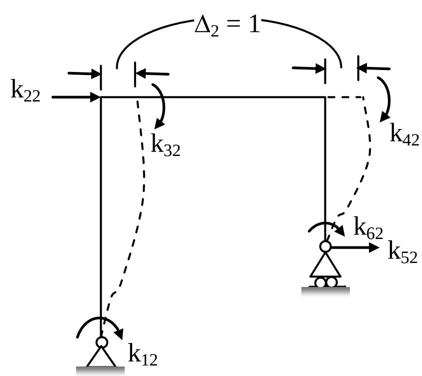
$$k_{22} = \frac{12E(4I)}{10^3} + \frac{12EI}{5^3} = 0.144EI$$

$$k_{32} = -\frac{6E(4I)}{10^2} = -0.240EI$$

$$k_{42} = -\frac{6EI}{5^2} = -0.240EI$$

$$k_{52} = -\frac{12EI}{5^3} = -0.096EI$$

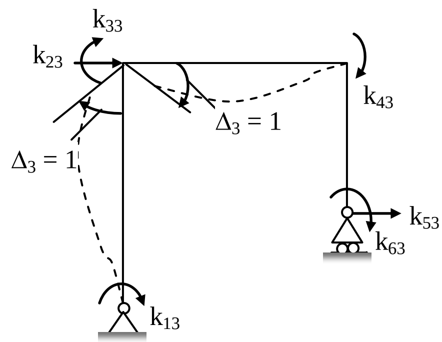
$$k_{62} = -\frac{6EI}{5^2} = 0.240EI$$



To generate the third column of the stiffness matrix, give a unit displacement at coordinate 3 without any displacement at other coordinates as shown in figure.

Compute the forces at all the coordinates using equation

$$k_{13} = \frac{2E(4I)}{10} = 0.800EI$$



$$k_{23} = -\frac{6E(4I)}{10^2} = -0.240EI$$

$$k_{33} = \frac{4E(4I)}{10} + \frac{4E(4I)}{10} = 3.200EI$$

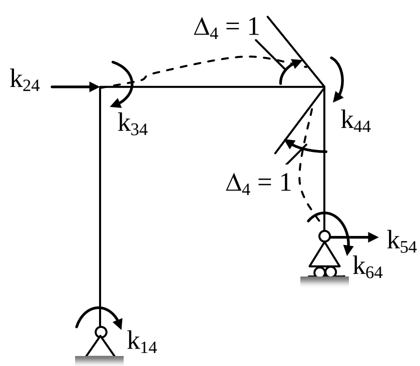
$$k_{43} = \frac{2E(4I)}{10} = 0.800EI$$

$k_{53} = k_{63} = 0$

To generate the fourth column of stiffness matrix, give a unit displacement at coordinate 4 without any displacement at other coordinates as shown in figure. Compute the forces at all the coordinates using equation

$k_{14} = 0$

$$k_{24} = -\frac{6EI}{5^2} = -0.240EI$$



$$k_{34} = \frac{2E(4I)}{10} = 0.800EI$$

$$k_{44} = \frac{4E(4I)}{10} + \frac{4EI}{5} = 2.400EI$$

$$k_{54} = \frac{6EI}{5^2} = 0.240EI$$

$$k_{64} = \frac{2EI}{5} = 0.400EI$$

To generate the fifth column of stiffness matrix, give a unit displacement at coordinate 5 without any displacement at other coordinates as shown in figure. Compute the forces at all the coordinates using equation

$k_{15} = 0$

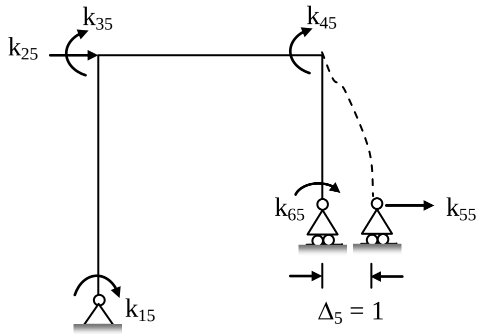
$$k_{25} = -\frac{12EI}{5^3} = -0.096EI$$

$k_{35} = 0$

$$k_{45} = \frac{6EI}{5^2} = 0.240EI$$

$$k_{55} = \frac{12EI}{5^3} = 0.096EI$$

$$k_{65} = \frac{6EI}{5^2} = 0.240EI$$



To generate the sixth column of stiffness matrix, give a unit displacement at coordinate 6 without any displacement at other coordinates as shown in figure. Compute the forces at all the coordinates using equation

$k_{16} = 0$

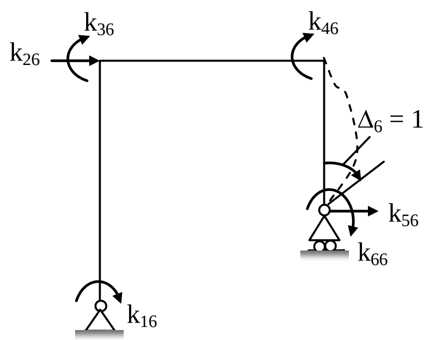
$$k_{26} = -\frac{6EI}{5^2} = -0.240EI$$

$k_{36} = 0$

$$k_{46} = \frac{2EI}{5} = 0.400EI$$

$$k_{56} = \frac{6EI}{5^2} = 0.240EI$$

$$k_{66} = \frac{4EI}{5} = 0.800EI$$



Thus all the elements of the stiffness matrix have been determined. it may be noted that the resulting stiffness matrix is symmetrical.

04(a).

Sol: Let the C.G be at a distance $\bar{y}$ from bottom

then

$$\bar{y} = \frac{(400\times 50)\times 25 + (50\times 200)\times(50+100) + (250\times 50)(50+200+25)}{(400\times 50)+(50\times 200)+(250\times 50)}$$

$$= \frac{5\times 10^5 + 15\times 10^5 + 34.375\times 10^5}{42500}$$

= 127.94 mm from bottom

$$\therefore I = \left\{\frac{400\times50^3}{12} + 400\times50\times\left(127.94 - \frac{50}{2}\right)^2\right\} + \left\{\frac{50\times200^3}{12} + (50\times200)[127.94-(50+100)]^2\right\}$$

$$+\left\{\frac{250\times50^3}{12} + (250\times50)\left[127.94 - \left(250+\frac{50}{2}\right)\right]^2\right\}$$

$$= \{41.67\times10^5 + 2119.33\times10^5\} + \{333.33\times10^5 + 48.66\times10^5\} + \{26.04\times10^5 + 2703.33\times10^5\}$$

$$= 5.272 \times 10^8 \text{ mm}^4$$

$y_{max} = 300 - 127.94 = 172.06$ mm

$$\therefore \text{ Section modulus } z = \frac{I}{y_{max}} = \frac{5.272\times10^8}{172.06} = 3064047 \text{ mm}^3$$

To find position of equal area axis

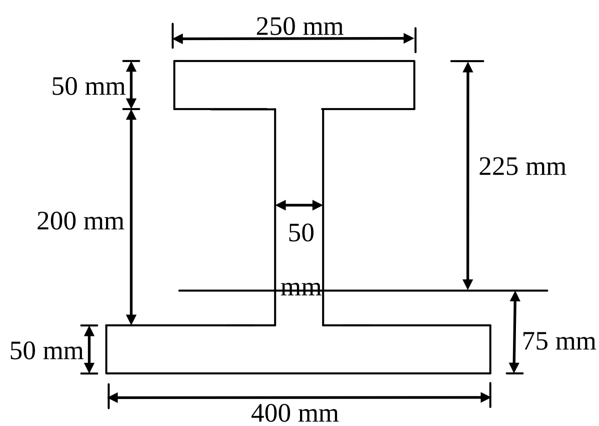
Assuming that the plastic N.A lies at y_p from bottom flange

$$(400\times50) + 50(y_p - 50) = \frac{1}{2}(\text{Total area})$$

$$= \frac{1}{2}\times\{400\times50 + 50\times200 + 250\times50\}$$

$$\Rightarrow y_p - 50 = \frac{\frac{1}{2}(42500) - 400\times50}{50} = \frac{1250}{50} = 25 \text{ mm}$$

$= 25 + 50 = 75$ mm from bottom



$z_p = A_1y_1 + A_2y_2 + A_3\, y_3 + A_4y_4$

$$= (250\times 50)\times\left(225-\frac{50}{2}\right)+(225-50)\times 50\times 50\times\left(\frac{225-50}{2}\right)+(75-50)\times 50\times\left(\frac{75-50}{2}\right)$$

$$+(400\times 50)\times\left(75-\frac{50}{2}\right)$$

$= 2.5 \times 10^6 + 0.7656 \times 10^6 + 0.0156 \times 10^6 + 1 \times 10^6$

$= 4.28125 \times 10^6\ \text{mm}^3$

$\therefore$ Shape factor = $\dfrac{z_p}{z} = \dfrac{4.28125\times 10^6}{3064047} = 1.397$

$\therefore$ Plastic moment = $M_p = f_y \times z_p$

$= 250 \times 4.28125 \times 10^6$

$= 1.0703 \times 10^9$ N-mm

04(b).

Sol:

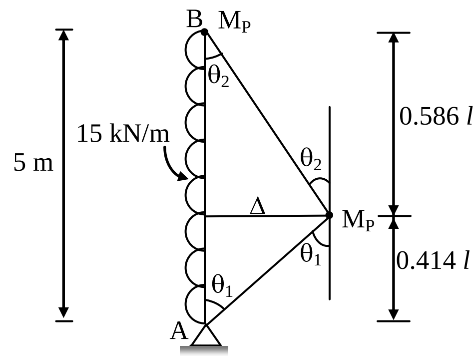
Fiber reinforced concrete can be defined as a composite material consisting of mixtures of cement, fine aggregate, coarse aggregate, water and discontinuous, discrete, uniformly dispersed fibers.

Plain cement concrete has certain undesired properties such as low tensile strength and low ductility. This is due to the formation of internal microcracks during the hydration of cement, making the concrete brittle and weak in tension. It has been observed that the admission of small, closely spaced and uniformly distributed fibers into concrete would act as crack arresters and would subsequently improve the static and dynamic properties of the concrete. This type of concrete is known as fiber reinforced concrete. Continuous meshes, woven fabrics and long wires or rods are not considered to be discrete fibers.

The fibers can be imagined as an aggregate with an extreme deviation in shape from the rounded smooth aggregate. The fibers interlock and entangle around aggregate participles and considerably reduce the workability, while the mix becomes more cohesive and less prone to segregation. The fibers suitable for reinforcing the concrete are generally produced from steel, glass and organic polymers. Fiber reinforced concrete has special applications in the construction of hydraulic structures, highway pavements, bridges and tunnel linings.

04(c).

Sol: (i) Beam Mechanism in Beam AB:



Equating the internal work done to external work done, we get

$M_P (\theta_1 + \theta_2) + M_P \theta_2 = \{15(5)\}$ (Average virtual displacement)

$$= 75\left(\frac{0+\Delta}{2}\right)$$

where $\Delta = 0.414\, l\theta_1 = 0.586\, l\theta_2$

$$\Rightarrow \theta_1 = \frac{0.586\ell\theta_2}{0.414\ell} = 1.415\theta_2$$

$l = 5$ m

$$\therefore M_P(1.415\theta_2 + \theta_2) + M_P\theta_2 = 75 \times \frac{(0.586 \times 5\theta_2)}{2}$$

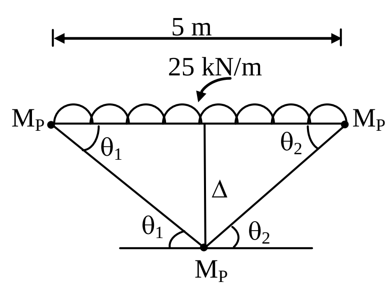
$\Rightarrow 3.415\ M_P\theta_2 = 109.875\ \theta_2$

$$\Rightarrow M_P = \frac{109.875}{3.415}$$

$= 32.17$ kN-m

$M_P = 32.17$ kN-m

(ii) Beam Mechanism in Beam BC:



Equating the internal work done to external work done, we get

$M_P\,\theta_1 + M_P\,(\theta_1 + \theta_2) + M_P\,\theta_2 = 25(5)$ (Average virtual displacement)

$$= 125\left(\frac{\Delta}{2}\right)$$

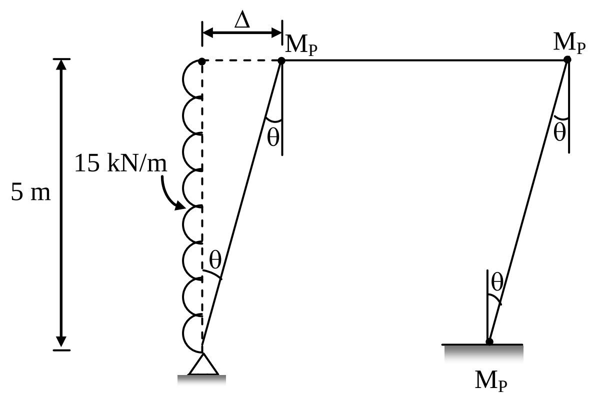
where $\Delta = 2.5\theta_1 = 2.5\theta_2 \Rightarrow \theta_1 = \theta_2$

$$\therefore M_P\theta_1 + M_P(\theta_1 + \theta_1) + M_P\theta_1 = \frac{125}{2}(2.5\theta_1)$$

$\Rightarrow 4M_P\theta_1 = 156.25\,\theta_1$

$\Rightarrow M_P = 39.06$ kN-m

(iii) Sway Mechanism:



Equating the internal work done to external work done, we get

$M_P\,\theta + M_P\,\theta + M_P\,\theta = (15 \times 5)$ (Average virtual displacement)

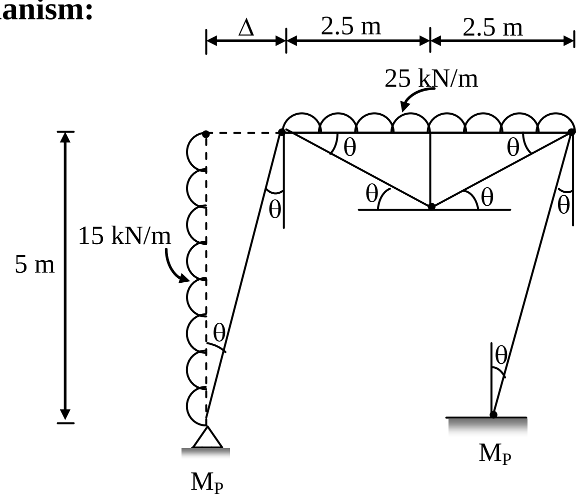
$$= 75\left(\frac{\Delta}{2}\right)$$

where $\Delta = 5$

$$\therefore 3M_P\theta = \frac{75}{2}(50)$$

$\Rightarrow M_P = 62.5$ kN-m

(iv) Combined Mechanism:



Equating the internal work done to external work done, we get

$$M_P(\theta+\theta)+M_P(\theta+\theta)\,M_P\theta=(15\times5)\left(\frac{\Delta_1}{2}\right)+(25\times5)\left(\frac{\Delta_2}{2}\right)$$

where $\Delta_1 = 5\theta$ and $\Delta_2 = 2.5\theta$

$$\therefore 5M_P\theta=\frac{75}{2}(5\theta)+\frac{125}{2}(2.5\theta)$$

$$\Rightarrow M_P=\frac{(187.5+156.25)\theta}{5\theta}=68.75\text{ kNm}$$

$\Rightarrow M_P = 68.75$ kN-m

$\therefore$ From (i), (ii), (iii) and (iv), we conclude that combined mechanism is the real mechanism with maximum value of $M_P = 68.75$ kN-m

05(a).

Sol: The mix proportion for M25 grade of concrete based on standard mix is

1 : 1 : 2 (weight batching)

$\Rightarrow w_{FA} = w_C$

$W_{CA} = 2w_C$

Given w/c = 0.4

$\Rightarrow w_W = 0.4\ w_C$

To prepare $1m^3$ of this concrete

$$1m^3 = \frac{w_w}{1\times 9.81} + \frac{w_C}{3.15\times 9.81} + \frac{WFA}{2.6\times 9.81} + \frac{W_{CA}}{2.5\times 9.81}$$

$$1 = \frac{0.4W_C}{9.81} + \frac{W_C}{3.15\times 9.81} + \frac{W_C}{2.6\times 9.81} + \frac{2W_C}{2.5\times 9.81}$$

$\Rightarrow \quad W_C = 5.158$ kN

$= 526$ kg

$W_W = 0.4\ W_C$

$= 0.4 \times 526$

$= 210$ kg

$W_{FA} = W_C$

$= 526$ kg

$W_{CA} = 2W_C$

$= 2 \times 526$

$= 1052$ kg

05(b). (i)

Sol:

Cement being the binding agent in concrete, the strength of concrete mainly depends on the strength of the cement paste. The strength of the cement paste in turn depends on the percentage of cement present in it. Thus, the strength of concrete decreases with increase in water cement ratio, as the percentage of cement present in the cement paste decreases with increase in water cement ratio. According to Abram's law on water-cement ratio, the strength of concrete is only dependent on the water-cement ratio provided the mix is workable. It has to be noted here that with a decrease in water-cement ratio the strength of the concrete increases, but it will become more and more difficult to achieve complete compaction of concrete. Further, when the water-cement ratio is below a practical limit, the strength of the concrete falls rapidly as some cement will be left unreacted and air voids are introduced.

05(b). (ii)

Sol:

Segregation of Concrete:

The separation of coarse aggregate from fine aggregate, cement paste from coarse aggregate or water from the matrix of a freshly prepared concrete is called segregation. It can occur due to improper placing, dropping from heights, improper proportioning, prolonged transportation, over vibration of concrete. It can be reduced by increasing air entrainment, using dispersing agents and pozzolana.

Bleeding of Concrete:

The emergence and flow of water from freshly placed concrete is called as bleeding. Bleeding is a special case of segregation which is mainly caused due to excessive vibration of concrete. It can be reduced by the use of uniformly graded aggregates, pozzolana, air entraining agents.

05(c).

Sol: Fixed End Moments:

$$\text{FEM at A} = \Sigma\frac{Wab^2}{\ell^2} = \frac{120\times3\times6^2}{9^2} + \frac{180\times6\times3^2}{9^2}$$

$$= 280 \text{ kN-m}$$

$$\text{FEM at B} = \Sigma\frac{Wab^2}{\ell^2} = \frac{180\times3\times6^2}{9^2} + \frac{120\times6\times3^2}{9^2}$$

$$= 320 \text{ kN-m}$$

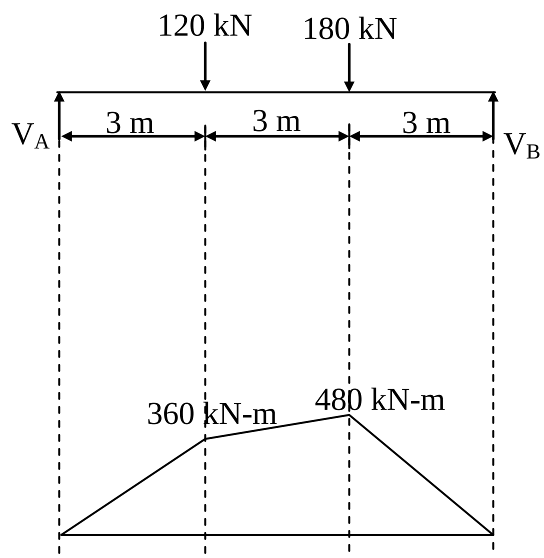
Free B.M.D: Consider the span as simply supported beam calculate reactions at A & B

$\Sigma M_A = 0$

$v_B \times 9 = 180 \times 6 + 120 \times 3$

$v_B = 160$ kN

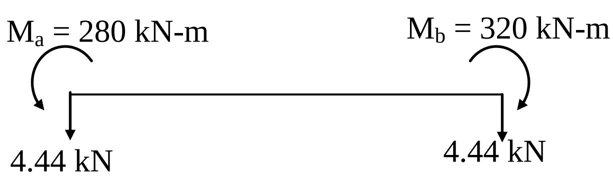
$v_A = 140$ kN



Reaction V at each support due to end moments alone

$$V = \frac{280 - 320}{9} = -4.44 \text{ kN}$$

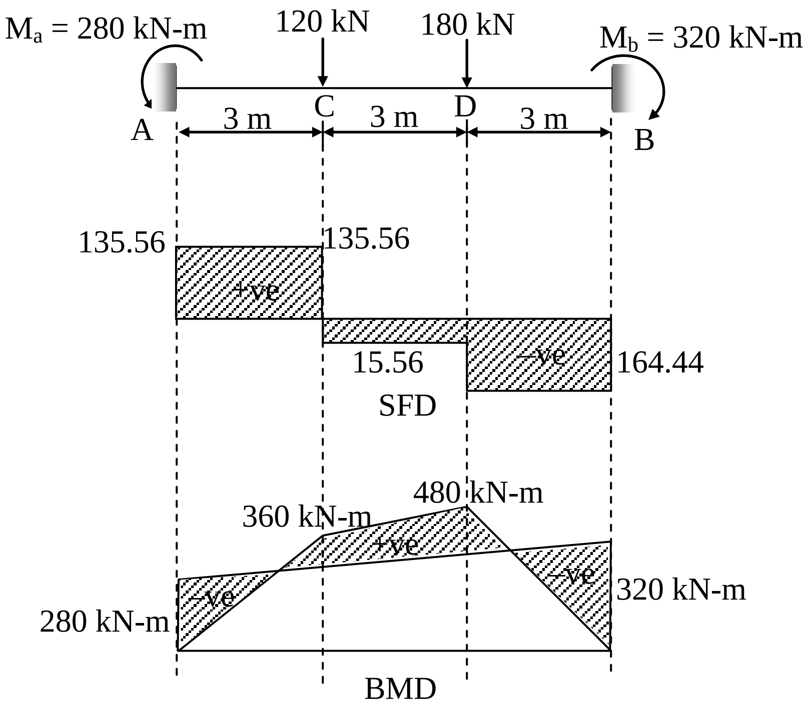
Since $M_b > M_a$ the reaction V at B is upward and reaction V at A is downward



Final reaction at A = $V_a = V_a - V$ = 140 – 4.44 = 135.52 kN

Final reaction at B = $V_b = V_b + V$ = 160 + 4.44 = 164.44 kN

By considering force and fixed BMD, the final BM diagram can be drawn.

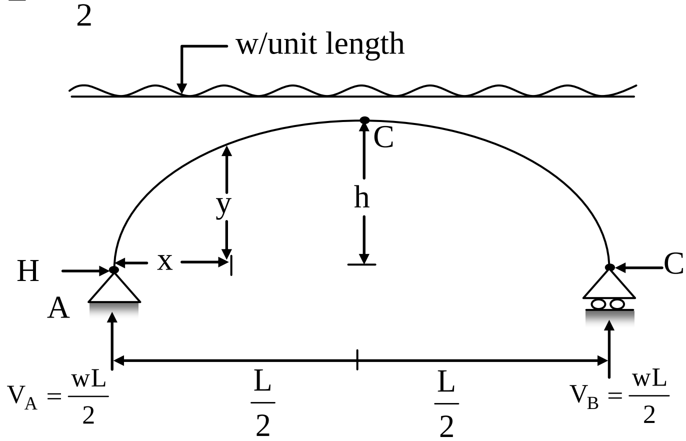


05(d).

Sol: Let the span of the arch be L and rise be h as shown in figure. Due to symmetry

$$V_A = V_B = \frac{1}{2} \times \text{total load}$$

$$= \frac{1}{2} w \times L = \frac{wL}{2}$$



Taking moment about C, we get

$$0 = V_A \times \frac{L}{2} - Hh - w \times \frac{L}{2} \times \frac{L}{4}$$

$$= \frac{wL}{2}\frac{L}{2} Hh - \frac{wL^2}{8}$$

Or, $H = \frac{wL^2}{8}$

At any section distance x from A,

$$M = V_A x - Hy - \frac{wL^2}{2}$$

But in a parabolic arch, $y = \frac{4hx(L-x)}{L^2}$

$$\therefore \quad M = \left(\frac{wL}{x} x\right) - \frac{wL^2}{8} \times \frac{4hx(L-x)}{L^2} - \left(\frac{wL^2}{2}\right)$$

$$= \left(\frac{wLx}{2}\right) - \frac{w}{2} x(L-x) - \left(\frac{wL^2}{2}\right) = 0$$

Thus, for a parabolic arch subjected to a uniformly distributed load over its entire span, the bending moment at any section is zero. Hence, the parabolic shape is a funicular shape for a three-hinged arch subjected to uniformly distributed load over entire span.

05(e).

Sol: The arch is shown in figure. Due to symmetry

$$V_A = V_B = \frac{1}{2} \times \text{total load}$$

$$= \frac{1}{2} \times 2\left(\frac{1}{2} \times 15 \times 40\right)$$

$$= 300 \text{ kN}$$

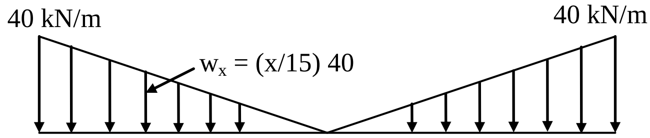
$\Sigma M_C = 0$, gives

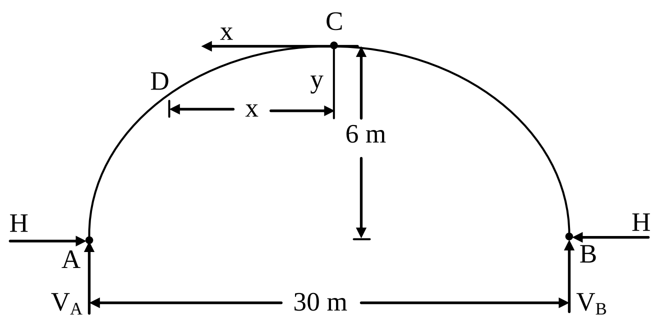
$$V_A \times 15 - H \times 6 - \frac{1}{2} \times 15 \times 40 \times 10 = 0$$

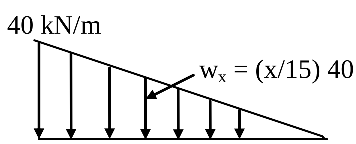
$H = 250$ kN

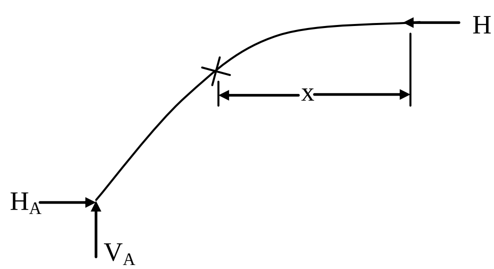
Taking origin at crown point C, the equation of parabola is given by

$$\frac{x^2}{y} = \text{constant} = a$$









At x = 15 and y = 6

$$a = \frac{15^2}{6} = 37.5$$

or x^2 = 37.5 y or $y = \frac{x^2}{37.5}$

Reaction at crown is the horizontal force H. Here, due to symmetry, there is no vertical shear.

$$\therefore\ M_x = H_y - \frac{1}{2} x \times W_x \times \frac{2x}{3}$$

$$= 250 \times \frac{x^2}{3.75} - \frac{1}{2} \times x \times \frac{x}{15} \times 40 \times \frac{2x}{3}$$

$$= \frac{250x^2}{37.5} - \frac{40x^3}{45}$$

For maximum bending moment,

$$\frac{dM_x}{dx} = 0 = \frac{250}{37.5} 2x - \frac{40}{45} 3x^2$$

$$\therefore \qquad x\left(\frac{500}{37.5} - \frac{40}{15} x\right) = 0$$

The above equation has two solutions: x = 0 and $\frac{500}{37.5} - \frac{40}{15} x = 0$

x = 0 gives the crown point, where moment is minimum, i.e., zero. Hence, the point of maximum moment is given by

$$x = \frac{500}{37.5} \times \frac{15}{40} = 5$$

$$M_{max} = \left(\frac{250x^2}{37.5}\right) - \left(\frac{40x^3}{45}\right)$$

$$= \left(\frac{250 \times 25}{37.5}\right) - \left(\frac{40 \times 125}{45}\right) = 55.555 \text{ kNm}$$

At x = 5 m from crown, i.e., at 10 m from abutment.

06(a).

Sol: Fixed end moment:

$FEM_{AB} = FEM_{BA} = 0$

$$FEM_{BC} = \frac{-100 \times 4^2 \times 2}{6^2} = -88.89 kNm$$

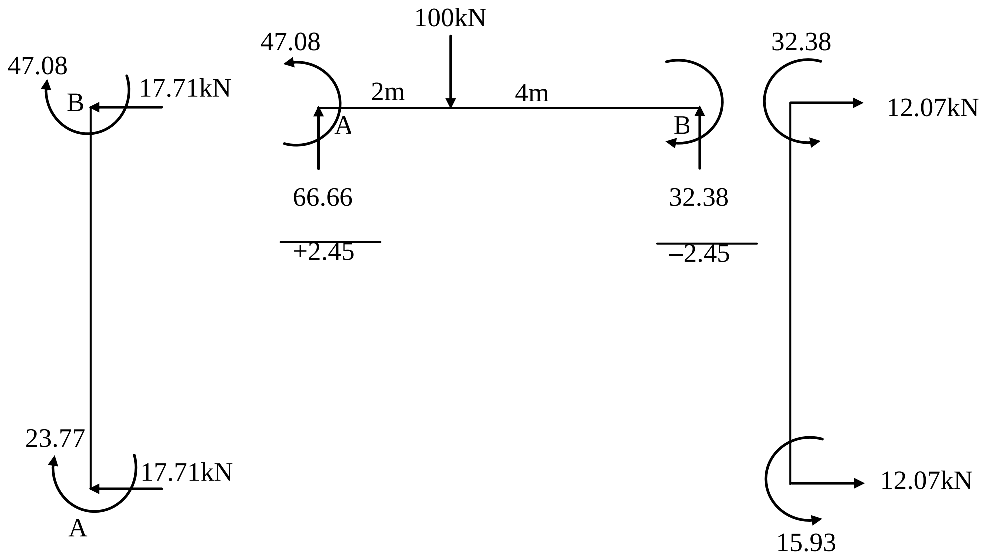
$$FEM_{CB} = \frac{100 \times 4 \times 2^2}{6^2} = 44.44 kNm$$

$FEM_{CD} = FEM_{DC} = 0$

Distribution factors:

Joint	Member	Relative stiffness	Total relative stiffness	Distribution Factor
B	BA	$\frac{I}{4}$	$\frac{7I}{12}$	$\frac{3}{7}$
	BC	$\frac{2I}{6}$		$\frac{4}{7}$
C	CB	$\frac{2I}{6}$	$\frac{7I}{12}$	$\frac{4}{7}$
	CD	$\frac{I}{4}$		$\frac{3}{7}$

Joint	A		B		C	D
Member	AB	BA	BC	CB	CD	DC
DF	—	$\frac{3}{7}$	$\frac{4}{7}$	$\frac{4}{7}$	$\frac{3}{7}$	—
FEM	0	0	–88.89	44.44	0	0
		38.09	50.80	–25.40	–19.04	
	19		–12.70	25.54		–9.5
		6.44	7.26	–14.51	–10.89	
	3.22		–7.25	3.63		5.45
		3.1	4.15	–2.07	–1.56	
	1.55		–1.04	2.07		–0.78
		0.45	0.59	–1.18	–0.89	
	23.77	47.08	–47.08	32.38	–32.38	–15.93



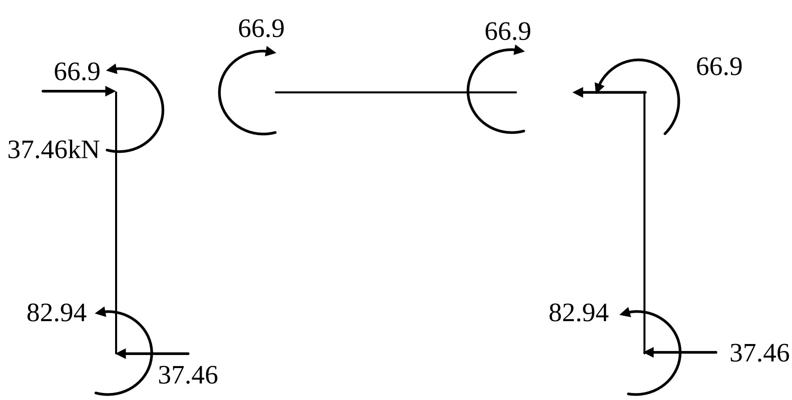
Sway force = – 17.71 + 13.07 = – 4.64 kN

Actual sway force = 4.64 kN(→)

Applying fixed end moments:

$FEM_{AB} = FEM_{BA} = FEM_{CD} = FEM_{DC} = -100$ kNm

Joint	A		B		C		D
Member	**AB**	**BA**	**BC**	**CB**	**CD**	**DC**
DF	—	$\frac{3}{7}$	$\frac{4}{7}$	$\frac{4}{7}$	$\frac{3}{7}$	—
FEM	–100	–100	0	0	–100	–100
		42.85	57.15	57.15	42.85	
	21.43		28.57	28.57		21.43
		–12.24	–16.33	–16.33	–12.24	
	–6.12		–8.16	–8.16		–6.12
		3.49	4.67	4.67	3.49	
	1.75		2.34	2.34		1.75
		–1	–1.34	–1.34	–1	
	–82.94	–66.9	66.9	66.9	–66.9	–82.94



Total sway causing force = 37.46 × 2

= 74.92 kN

Correction factor = $\dfrac{4.64}{74.92} = 0.062$

M_{AB} = 23.27 + (0.062 × (–82.94)) = 18.12 kNm

M_{BA} = 47.08 – 66.9 × 0.062 = 42.93 kNm

M_{BC} = –47.08 + 66.9 × 0.062 = – 42.93 kNm

M_{CB} = 32.38 + 66.9 × 0.062 = 36.52 kNm

M_{CD} = – 36.52 kNm

M_{DC} = – 15.93 – 82.94 × 0.062 = 21.07 kNm

06(b).

Sol: The effective length of column kL = 6 m

= 6000 mm

Angle of inclination θ = 45°

Size of flat lacing bar = 60 mm × 10 mm

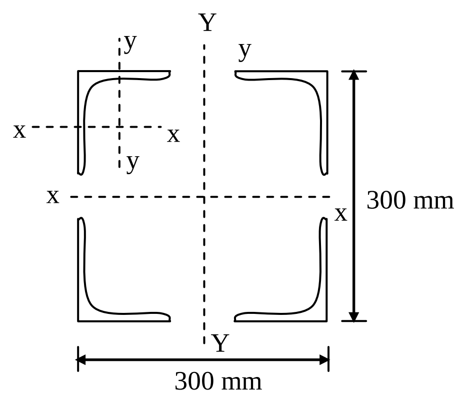
Properties of ISA 100 × 100 × 10

A = 1903 mm^2

$I_{xx} = I_{yy} = 177 \times 10^4$ mm^4

$C_{xx} = C_{yy}$ = 28.4 mm

r_v = 19.1 mm; g = 55 mm



Factored load on the column

$P_d = f_{cd}. A_e$

A_e = effective sectional area of built up column

= 4 × 1903 = 7612 mm^2

f_{cd} = Design stress in axial compression

Effective slenderness ratio = $\dfrac{KL}{r_{min}}$

Where r_{min} is minimum radius of gyration of built up column

Moment of inertia of Built up column about buckling axis

$$I_{XX} = I_{YY} = 4\left[177 \times 10^4 + 1903 \times \left(\frac{300}{2} - 28.4\right)^2\right]$$

$$= 119.63 \times 10^6 \text{ mm}^4$$

$$r_{min} = r_{XX} = r_{YY} = \sqrt{\frac{I_{min}}{A_e}} = \sqrt{\frac{119.63 \times 10^6}{7612}}$$

$$= 125.36 \text{ mm}$$

Effective slenderness ratio of laced built up column should be increased by 5% as per IS 800 : 2007 specifications

$$1.05\left(\frac{KL}{r_{min}}\right) = 1.05\left(\frac{6000}{125.36}\right) = 50.25$$

For

$$\frac{KL}{r} = 50.25, f_{cd} = 150 - \frac{(167 - 150)}{(70 - 60)}(50.25 - 50)$$

$$= 149.57 \text{ N/mm}^2$$

Factored load on the column

$P_d = f_{cd}.\ A_e$

$= 149.57 \times 7612$

$= 1138.52 \times 10^3$ N = 1138.52 kN

Using single flat lacing bars making an angle 45^o with the longitudinal axis of the column: Let 'L' & '*l*' be the spacing of lacing and length of lacing member respectively

Centre to centre distance

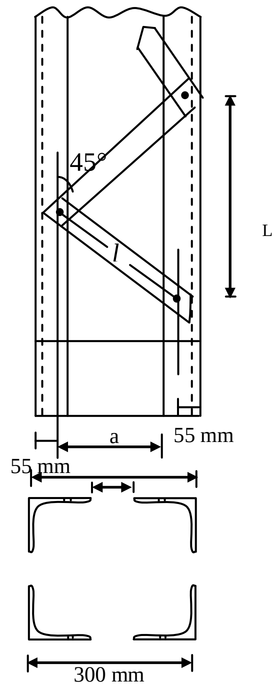
a = 300 – 55 – 55 = 190 mm

$$\tan 45^o = \frac{L/2}{a} t$$

L = 2a. tan 45^o = 2 × 190 = 380 mm

$$\sin 45^o = \frac{a}{\ell} = \frac{190}{\ell}$$

$$\ell = \frac{190}{\sin 45} = 268.70 \text{ mm}$$



Check for local buckling:

$$\frac{L}{r^c \min} = \frac{380}{r_{vv}} = \frac{380}{19.4} = 19.58 \leq 50$$

$$\leq 0.7 \times \left(\frac{KL}{r}\right) = 0.70 \times 47.86$$

$$= 33.50$$

Hence single lacing is sufficient

Check for buckling of flat lacing bar:

Effective slenderness ratio of lacing bar

$$= \frac{K\ell}{r_{min}}$$

Effective length of lacing bar Kl = 268.7 mm

Minimum radius of gyration of flat bar

$$= r_{min} = \sqrt{\frac{60 \times 10^3}{12} \times \frac{1}{60 \times 10}}$$

$$= 2.88 \text{ mm}$$

$$\frac{K\ell}{r_{min}} = \frac{268.7}{2.88} = 93.08 \leq 145$$

Hence lacing member is safe

06(c). (i)

Sol:

Refractory bricks are also known as fire bricks, as they are made from a special kind of clay called as fire clay. Fire clay has high percentages of silica and alumina. The percentages of alumina vary from 25 to 35 percent and that of silica varies from 65 to 75 percent. Other compounds such as lime, magnesia, iron oxide and alkalis together should not be more than 5 percent. These bricks are always burnt in Hoffman's kiln.

The following are the properties of refractory bricks:

1. The process of manufacture of these bricks is same as that of ordinary clay bricks.
2. The burning and cooling of fire bricks are done gradually.
3. These bricks are usually white or yellowish white in colour.
4. These bricks are around 30 to 35 N in weight.
5. These bricks can resist high temperature without softening or melting.
6. These bricks are used for lining of interior surfaces of furnaces, chimneys, kilns, ovens, etc.
7. The compressive strength of these bricks is around 200 to 225 N/mm2.
8. The percentage of water absorption of these bricks varies from 5 to 10 percent.

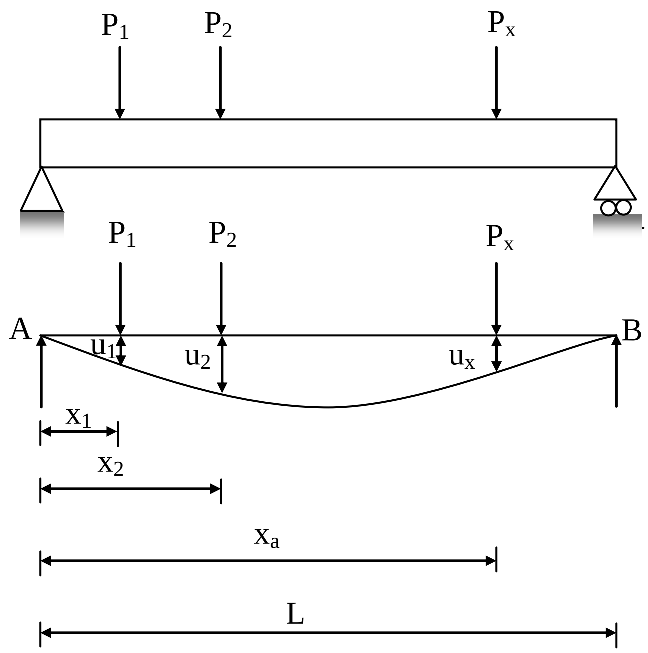
06(c) (ii)

Sol:

S.No	Thermoplastics	Thermosetting Plastics
1.	These solidify when cooled below a particular temperature.	These solidify when heated above a particular temperature.
2.	These have weak forces of interaction among chains.	The whole mass of polymers is well connected with strong covalent bonds.
3.	Average molecular weight can be defined.	Average molecular weight cannot be defined.
4.	Expensive	Cost-effective
5.	Highly recyclable.	Cannot be recycles.
6.	These melt when heated	More resistant to high temperatures

07(a) (i)

Sol: Catigliano's First Theorem: For linearly elastic structure, where external forces only cause deformations, the complementary energy is equal to the strain energy. For such structures, the Castigliano's first theorem may be stated as the first partial derivative of the strain energy of the structure with respect to any particular force gives the displacement of the point of application of that force in the direction of its line of action.



Let P_1, P_2, …., P_n be the forces acting at x_1, x_2, ……., x_n from the left end on a simply supported beam of span L. Let u_1, u_2, …., u_x be the displacements at the loading points P_1, P_2,…….., P_n respectively as shown in Fig. Now, assume that the material obeys Hooke's law and invoking the principle of superposition, the work done by the external forces is given by

$$W = \frac{1}{2}P_1u_1 + \frac{1}{2}P_2u_2 + ... + \frac{1}{2}P_nu_n \qquad ……………. (1)$$

Work done by the external forces is stored in the structure as strain energy in a conservative system. Hence, the strain energy of the structure is,

$$U = \frac{1}{2}P_1u_1 + \frac{1}{2}P_2u_2 + ... + \frac{1}{2}P_nu_n \qquad ……………… (2)$$

Displacement u_1 below point P_1 is due to the action of P_1, P_2,…..P_n acting at distances x_1,x_2,….,x_n respectively from left support. Hence, u_1 may be expressed as,

$$u_1 = a_{11}P_1 + a_{12}P_2 + ……. + a_{1n}P_n \qquad ………… (3)$$

In general,

$u_i = a_{i1}P_1 + a_{i2}P_2 + \ldots\ldots + a_{in}P_n \qquad i = 1,2\ldots.n \quad \ldots\ldots\ldots (4)$

where a_{ij} is the flexibility coefficient at I due to unit force applied at j. Substituting the values of $u_1, u_2, \ldots\ldots u_n$ in equation(2) from equation (4), we get

$$U = \frac{1}{2}P_1[a_{11}P_1 + a_{12}P_2 + \ldots..] + \frac{1}{2}P_2[a_{21}P_1 + a_{22}P_2 + \ldots..] + \ldots\ldots + \frac{1}{2}P_n[a_{n1}P_1 + a_{n2}P_2 + \ldots..] \quad \ldots\ldots.. (5)$$

We know from Maxwell-Betti's reciprocal theorem $a_{ij} = a_{ji}$. Hence, equation (5) may be simplified as

$$U = \frac{1}{2}\left[a_{11}P_1^{\ 2} + a_{12}P_2^{\ 2} + \ldots..+ a_{mn}P_n^{\ 2}\right] + \left[a_{12}P_1P_2 + a_{13}P_1P_3 + \ldots..+ a_{an}P_1P_n\right] + \ldots\ldots (6)$$

Now, differentiating the strain energy with any force P_1 gives,

$$\frac{\partial U}{\partial P_1} = a_{11}P_1 + a_{12}P_2 + \ldots\ldots + a_{1n}P_n \quad \ldots\ldots\ldots.. (7)$$

It may be observed that equation (7) is nothing but displacement u_1 at the loading point.

In general,

$$\frac{\partial U}{\partial P_n} = u_n \quad \ldots\ldots\ldots (8)$$

Hence, for determinate structure within linear elastic range the parties derivative of the total strain energy with respect to any external load is equal to the displacement of the point of application of load in the direction of the applied load, provided the supports are unyielding and temperature is maintained constant.

07(a). (ii)

Sol:

S.No.	Determinate Structures	Indeterminate Structures
1	Equilibrium conditions are fully adequate to analyse the structure.	Conditions of equilibrium are not adequate to fully analyse the structure.
2	Bending moment or shear force any section is independent of the material property of the structure.	Bending moment or shear force at any section depends upon the material property.
3	The bending moment or shear force at any section is independent of the cross-section or moment of inertia.	The bending moment or shear force at any section depends upon the cross-section or moment of inertia.
4	Temperature variations do not cause stresses.	Temperature variations cause stresses.
5	No stresses are caused due to lack of fit	Stresses are caused due to lack of fit
6	Extra conditions like compatibility of displacements are not required to analyse the structure.	Extra conditions like compatibility of displacements are required to analyse the structure along with the equilibrium.

07(b).

Sol: Assume yield stress for structural steel to be used is f_y = 250 N/mm^2 and compression flange of the beam is restrained by floor construction.

Given load is uniformly distributed load due to dead load (self weight) and live load.

Partial safety factor for DL + LL, γ_L = 1.50

Design (or) factored load = 1.5 × 60 = 90 kN-m

Maximum factored moment occur at centre

$$M_{max} = \frac{wL^2}{8} = \frac{90(15)^2}{8} = 2531.25 \text{ kN} - \text{m}$$

Maximum factored shear force

$$V_{max} = \frac{wL}{2} = \frac{90(15)}{2} = 675 \text{ kN}$$

Moment of resistance of beam section required

$$M_d = \frac{\sigma_y . Z_p}{\gamma_{ms}}$$

where Z_p = plastic section modulus required

$$Z_{px} = \frac{2531.25 \times 10^6 \times 1.10}{250}$$

$= 11.137 \times 10^6 \text{ m}^3$

$Z_{px} = 11137 \times 10^3 \text{ mm}^3$

Elastic section modulus, (Assuming shape factor = 1.12)

$$\text{required } (Z_{xx})_R = \frac{Z_p}{1.12}$$

$$= \frac{11137 \times 10^3}{1.12}$$

$= 9944.19 \times 10^3 \text{ mm}^3$

For ISWB 600

$I_{xx} = 106198.5 \times 10^4 \text{ mm}^4$, d = 600 mm

$I_{yy} = 4702.5 \times 10^4 \text{ mm}^4$

$$Z_{xx} = \frac{I_{xx}}{y} = \frac{106198.5 \times 10^4}{(600/2)}$$

$= 3539.95 \times 10^3 \text{ mm}^3$

$(Z_{xx})_R > (Z_{xx})_B$, Hence plated beam to be used.

Section modulus of plates

$Z_p = 9944.19 - 3539.95 = 6404.24 \text{ cm}^3$

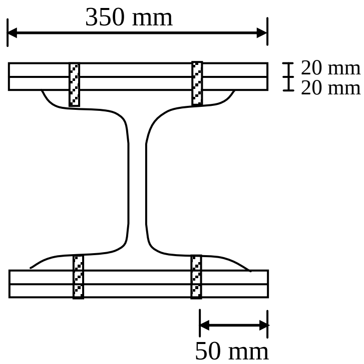
Let width of cover plate B_f = 350 mm

Area of each cover plate is A_p

$$A_p = \frac{6404.24 \times 10^3}{600} = 10673.33 \text{ mm}^2$$

Thickness of cover plate required = $t_p = \frac{10673.33}{350} = 30.49$ mm

Provide 2 No's 20 mm thick cover plates of width 350 mm attached with flange



Assume Given ISWB 600 @ 145.1 kg/m is classified as plastic section.

Design Moment of Resistance of the plated beam section (M_d)

Elastic section modulus of beam

$(Z_{xx})_B = 3539.95\ cm^4$

Moment of Inertia of built up beam.

$$I_{xx} = \left[106198.5 \times 10^4\right] + 2\left[\frac{350 \times 40^3}{12} + 350 \times 40 \times (320)^2\right]$$

$= 3932.91 \times 10^6\ mm^4$

Elastic section Modulus of built up beam

$Z_{xx} = 13109.72 \times 10^3\ mm^3$

Assuming shape factor = 1.12

Plastic section Modulus of built up beam

$Z_p = 11705.11 \times 10^3\ mm^3$

Design bending strength of built up beam

$$M_a = \frac{Z_p.f_y}{\gamma_{mo}} = 11705.1 \times 10^3 \times \frac{250}{1.10}$$

$= 2660.2 \times 10^6$ N-mm

Design bending moment due to external load (M = 2531.25 kN-m) is less than Design bending strength of built up beam section. Hence beam section is adequate for flexure.

Design shear strength of beam (V_d)

$$V_d = \frac{f_y}{\sqrt{3}.\gamma_{mo}} \times \text{Area of web}$$

$$= \frac{250}{\sqrt{3} \times 1.10} \times 600 \times 11.8$$

(thickness of web = 11.8 mm)

= 929.036 × 10^3 N = 929.03 k N,

Maximum factored shear force (V = 675 kN) is less than design shear strength of beam. (V_d = 929.03 kN).

Design for Deflection:

Assume for elastic cladding

$$\text{Limiting deflection} = \frac{\text{Span}}{240} = \frac{15000}{240}$$

= 62.5 mm

Maximum deflection under service load at middle

$$\Delta_{max} = \frac{5}{384} \frac{60(1500)^4}{20 \times 10^5 \times 3932.9 \times 10^6}$$

= 30.16 mm < 62.5 mm

Hence beam section is safe against deflection.

07(c)(i).

Sol: For M15 grade of concrete, the mix proportion based on nominal mix method is 1 : 2 : 4 (volume batching)

$\Rightarrow V_{FA}$ = 2VC Given W/C = 0.5

VCA = 4VC $\Rightarrow$ WW = 0.5 WC

To prepare 1m3 of the concrete

$$1m^3 = \frac{W_W}{1 \times 9.81} + \frac{W_C}{3.15 \times 9.81} + \frac{W_{FA}}{2.6 \times 9.81} + \frac{W_{EA}}{2.5 \times 9.81}$$

$$1 = \frac{0.5W_C}{9.81} + \frac{14.4V_C}{3.15 \times 9.81} + \frac{16.2V_{FA}}{2.6 \times 9.81} + \frac{15.8V_{CA}}{2.5 \times 9.81}$$

$$= \frac{0.5 \times 14.4 V_C}{9.81} + \frac{14.4 V_C}{3.15 \times 9.81} + \frac{16.2 \times 2 V_C}{2.6 \times 9.81} + \frac{15.8 \times 4 \times V_C}{2.5 \times 9.81}$$

$\Rightarrow$ VC = 0.198 m3

$\Rightarrow$ $$W_C = \frac{0.198 \times 14.4 \times 1000}{9.81}$$

= 291 kg

WW = 0.5 WC = 146 kg

$V_{FA} = 2V_C = 0.396$ m3

$$W_{FA} = \frac{0.396 \times 16.2 \times 1000}{9.81}$$

= 609 kg

VCA = 4VC = 0.792 m3

$$W_{CA} = \frac{0.792 \times 15.8 \times 1000}{9.81}$$

= 1276 kg

07(c)(ii).

Sol:

The important physical properties of Aluminium compared to other metals are as follows:

(i) Low specific gravity (Around 2.7).

(ii) High Corrosion Resistance.

(iii) High Electrical Conductivity.

(iv) High Thermal Conductivity.

(v) Low melting point.

(vi) High Ductility.

Since Aluminium has high corrosion resistance, no cleaning is required before melting it for recycling. And, since Aluminium has low melting point, fuel cost for melting is also less compared to other metals. Thus Aluminium has a very low recycling cost.

08 (a).(i)

Sol:

The different tests used to determine the properties of coarse aggregates are as follows:

Flakiness Index Test.

Elongation Index Test.

Aggregate Crushing Value Test.

Aggregate Impact Value Test.

Aggregate Abrasion Value Test.

Soundness Test.

The hardness of coarse aggregates is assessed based on the Aggregate Abrasion Value Test.

Aggregate Impact Value Test:

This test gives the Aggregate Abrasion Value (AAV), which is an index of the resistance of aggregates against surface wear and tear (i.e.,) Hardness.

The apparatus used for this test is called Los Angeles Machine.

The test specimen of aggregates is weighed (W1 kg) before transferring it to the machine along with Abrasive charge (Steel balls).

Now this sample is placed in the cylinder of the machine which is allowed to rotate @ 20 to 33 rpm for a total of 500 revolutions.

After 500 revolutions, the aggregates are sieved on a 1.70 mm IS sieve (As per IS:2386 Part IV) and the retained aggregates are weighed (say W2 kg).

Loss in weight as percentage indicates the percentage of wear.

Aggregate Impact Value (AAV) = (W2/W1) x 100.

08(a). (ii)

Sol:

Depending upon the nature, surface of the rock and the purpose for which stones are needed, quarrying is done by one or more of the following methods.

Excavation.

Wedging.

Heating.

Blasting.

Quarrying by Blasting:

Quarrying by Blasting is the quarrying method used to produce stones and aggregates on a large scale.

The operation of Quarrying by Blasting involves the following steps.

Boring: In this stage vertical bores or holes are drilled into the rock.

Charging: In this stage, the holes are dried completely and the required amount of charge is places inside the holes.

Tamping: In this stage, the charge places in the holes is compacted using a tamping rod.

Firing: The charged holes are ignited causing explosion, which breaks the rock into smaller pieces of different sizes.

08(b).

Sol: The truss is analyse by taking kN as unit of force and mm as unit for linear measurements.

P-Forces:

From the equilibrium of joint C

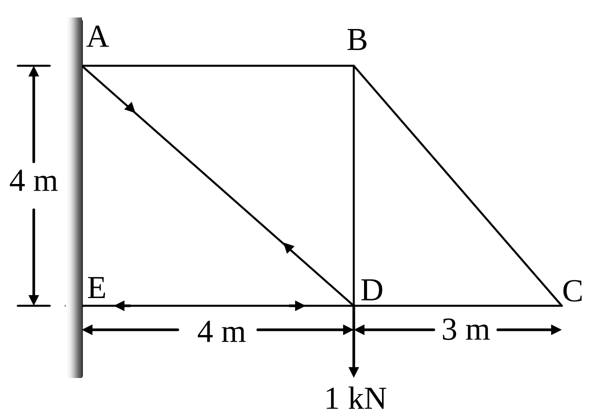
$P_{CB} \times 0.8 = 60$ or $P_{CB} = 75$ kN (tensile)

$P_{CB} \times 0.6 = P_{CD}$ or $P_{CD} = 45$ kN (compressive)

At joint B:

$P_{BD} = P_{CB} \times 0.8 = 75 \times 0.8 = 60$ kN (compressive)

$P_{BA} = P_{CB} \times 0.6 = 45$ kN (tensile)



At joint D:

$$P_{DA} \times \frac{1}{\sqrt{2}} = P_{BD} = 60 \text{ kN}$$

$$P_{DA} = 60\sqrt{2} \text{ kN(tensile)}$$

$$P_{DE} = P_{DA} \times \frac{1}{\sqrt{2}} + P_{DC} = 60 + 45 = 105 \text{ kN(compressive)}$$

k - forces due to unit load at D:

Referring to figure, from joint equilibriums C and B.

$$P_{CB} = P_{CD} = P_{BA} = P_{BD} = 0$$

From joint D:

$$P_{DA} \times \frac{1}{\sqrt{2}} = 1 \quad \text{or} \quad P_{DA} = \sqrt{2} \text{ kN(tensile)}$$

and $P_{DE} = P_{DA} \times \frac{1}{\sqrt{2}} = 1 \text{ kN(compressive)}$

Further calculations are carried out in table below.

Member	**Length**	**Area in mm**	**P-force in mm^2**	**k-force in kN**	$\frac{PkL}{AE}$
AB	4000	1000	45	0	0
BC	5000	1000	75	0	0
CD	3000	1000	– 45	0	0
DE	4000	1500	– 105	– 1	280
DB	4000	1000	– 60	0	0
AD	$4000\sqrt{2}$	1500	$-60\sqrt{2}$	$\sqrt{2}$	452.55

$$\sum \frac{PkL}{A} = 732.55$$

Deflection D = $\sum \frac{PkL}{AE}$

$$\Delta_D = \frac{1}{E}\sum \frac{PkL}{A}$$

$$\Delta_D = \frac{1}{200} \times 732.55$$

$$\Delta_D = 3.663 \text{ mm}$$

8(c).

Sol:

The increase in volume of sand due to the presence of free moisture in it is called as bulking of sand. This free moisture forms a thin film over the sand particles which keeps the neighbouring particles away due to the effects of surface tension. Thus resulting in bulking of the volume of the sand. The significance of surface tension forces and consequently how far the sand particles are pushed away will depend on the percentage of moisture present and also the size of the fine aggregate particles. Generally, this phenomenon is occurs in all sizes of aggregates, but it is significant mostly in fine sands and coarse silts. It is also to be noted that the effects of bulking increases with increase in moisture upto a certain limit and then with further increase in moisture content leads to decrease in the bulking effects. No bulking can be observed when the sand is completely saturated.

To estimate the extent of bulking of sand, a sample of moist fine aggregate is filled into a measuring cylinder. The level of the fine aggregate is noted as h1. Now water is poured into the measuring cylinder till the fine aggregate is completely inundated. Since, the fine aggregate is completely saturated, there are no bulking effects. Now, its level is noted down as h2. Now the percentage bulking can be calculates using the following formula.

% Bulking = (h1 – h2)/h2 x 100.

If the same experiment is repeated for different moisture contents and for different size of fine aggregates, we will get a set of curves known as bulking chart, which looks as follows. These bulking charts can be employed to assess the bulking nature of different sizes of aggregates.

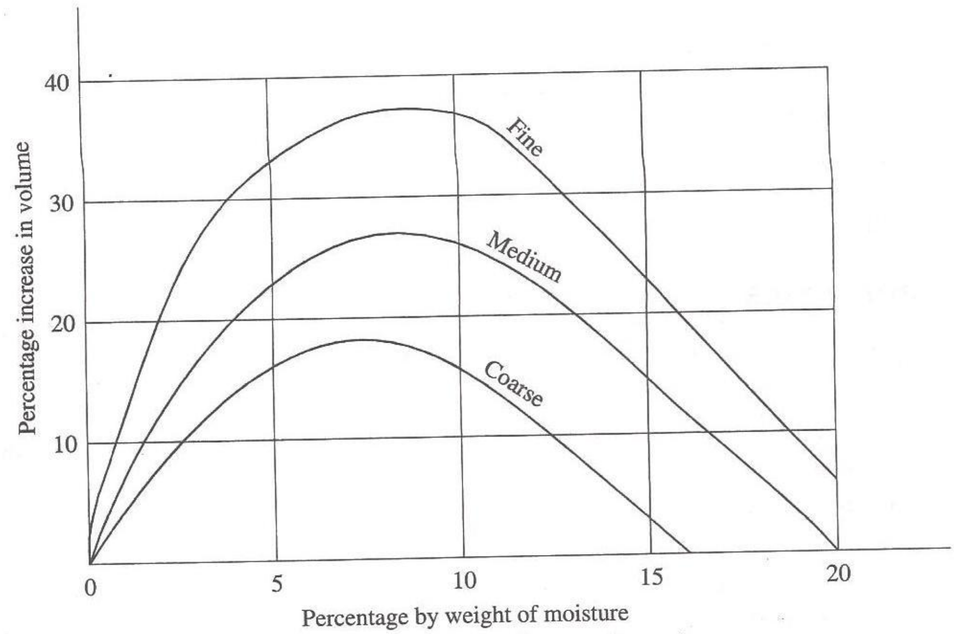


Chart showing bulking of sand