ACE Engineering Academy
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ESE – 2019 MAINS
OFFLINE TEST SERIES

CIVIL ENGINEERING

TEST – 11
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) (i).

Sol: At point A, there will be a concentrated load (upward) ⇒ W_A = Height of the jump

⇒ $W_A = 4050 - 0 \Rightarrow W_A = 4050$ N.

At point C, there will be a concentrated load (downward) ⇒ W_C = Height of the jump

⇒ $W_C = -2550 - (-750) = -2550 + 750 = -1800$ N.

At point D, there will be a concentrated load (upward) ⇒ $W_D = 0 - (-2550) = 2550$ N.

Between B and C, there will be no load.

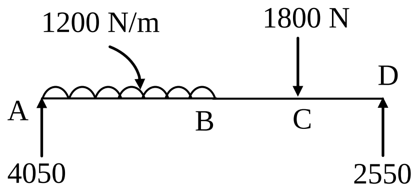
Between C and D, there will be no load.

Between A and B, there will be udl. Its intensity is given as

$$|w| = \frac{dF}{dx} = (\text{slope of SFD})_{AB}$$

$$|w| = \left|\frac{-750-4050}{4}\right| = \frac{4800}{4}$$

⇒ $|w| = 1200$ N/m



01 (a) (ii).

Sol: Given F.O.S = 3.5

$l = 2.5$ m

d = 60 mm

$E = 2.1 \times 10^5$ N/mm^2

The effective length of strut, $L = \frac{\ell}{\sqrt{2}}$

$$= \frac{2500}{\sqrt{2}} = 1767.76 \text{ mm}$$

Moment of inertia of the section, $I = \frac{\pi d^4}{64}$

$$= \frac{\pi \times (60)^4}{64} = 636172.51 \text{ mm}^4$$

Euler's crippling load $= P = \dfrac{\pi^2 EI}{\ell^2}$

$$= \frac{\pi^2 \times 2.1 \times 10^5 \times 636172.51}{(1767.76)^2}$$

= 421936.73 N

$\therefore$ Safe load $= \dfrac{P}{F.O.S} = \dfrac{421936.73}{3.5}$

= 120553.35 N

= 120.55 kN

01(b).

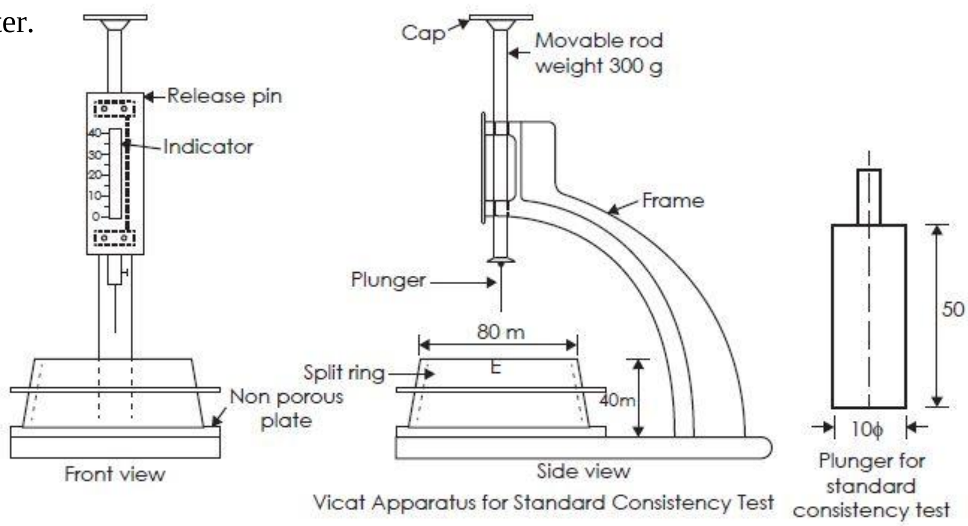
Sol: The Normal Consistency or Standard Consistency of a cement is defined as the percentage of water required to make a workable cement paste.

It can also be defined as the percentage of water required to make a cement paste which will permit a Vicat plunger to penetrate a depth of 33 to 35 mm from the top (5 to 7 mm from the bottom) of the mould of the Vicat Apparatus.

It is determined using Normal Consistency or Standard Consistency Test:

Test Apparatus:

The apparatus used for performing this test is the Vicat apparatus assembly with of a plunger having 10 mm diameter and 50 mm length and a mould which is 40 mm in height and 80 mm in diameter.



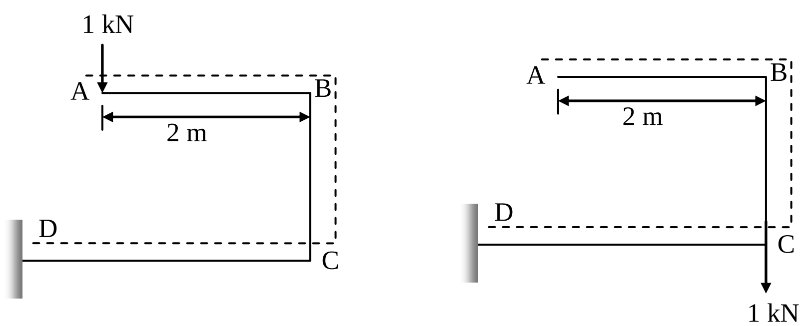
Vicat Apparatus for Standard Consistency Test

Test Procedure:

1. A cement paste is prepared by adding water 24% by weight of cement for the first trial.
2. The time of mixing should not be less than 3 minutes and should not be more than 5 minutes.
3. Now the paste is filled in the Vicat mould and the top is levelled off.
4. The mould is now placed under the plunger and is brought down to touch the surface of the paste.
5. Now the plunger is suddenly released and allowed to sink into the cement paste by its own weight.
6. The depth of penetration of the plunger is noted down.
7. The whole experiment is repeated with increments of water cement ratio until such a time when the plunger penetrated to a depth of 33 to 35 mm from the top, or 5 to 7 mm from the bottom, of the mould.
8. The percentage of water at which the plunger penetrated to a depth of 33 to 35 mm from the top, or 5 to 7 mm from the bottom, of the mould is known as the Normal Consistency or Standard Consistency of the cement, which is denoted as P.

01(c).

Sol:



The bending moment expressions for M due to given load, m_1 due to unit vertical load at A and m_2 due to unit vertical load at C are tabulated first (Note: moment carrying tension on dotted side is taken positive.)

Calculation table for Example

Portion	AB	BC	CD
Origin	A	B	C
Limit	0 – 2	0 – 2	0 – 3
M	$10x^2$	40	40 – 130x
m_1	x	2	2 – x
m_2	0	0	−x
Flexural Rigidity	EI	EI	EI

$$EI\Delta_A = \int_0^2 10x^2.xdx + \int_0^2 80dx + \int_0^3 (40-130x)(2-x)dx$$

$$= \left[\frac{10x^4}{4}\right]_0^2 + [80x]_0^2 + \int_0^3 (80-300x+130x^2)dx$$

$$= \frac{10(2^4)}{4} + 80(2) + \left[80x - 300\frac{x^2}{2} + \frac{130x^3}{3}\right]_0^3$$

$= 260$

Now, $E = 240$ GPa $= 240 \times 10^9$ N/m^2

$I = 150 \times 10^4 mm^4 = 150\times10^4\times10^{-12}\times m^4 = 150\times10^{-8} m^4$

$$\therefore \Delta = \frac{260}{240\times10^9\times150\times10^{-8}} = 7.222\times10^{-4} m = 0.722 \text{ mm}$$

$$EI\Delta_c = \int Mm_2 dx$$

$$= 0 + 0 + \int_0^3 (40-130x)(-x)dx$$

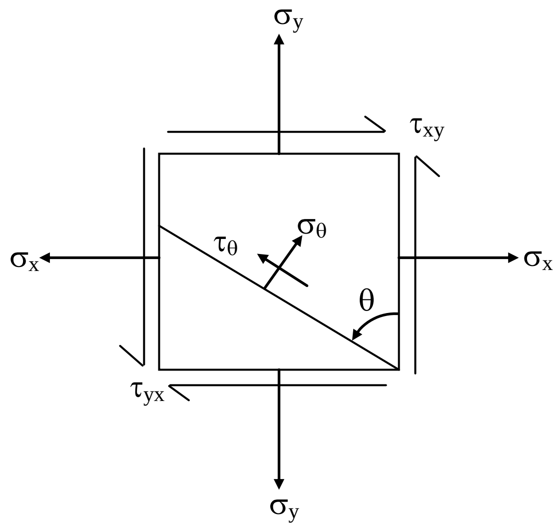
$$= \int_0^3 \left(-40x + 130x^2\right)dx$$

$$= \left[-20x^2 + 130\frac{x^3}{3}\right]_0^3 = 990$$

$$\Delta_C = \frac{990}{240\times10^9\times150\times10^{-8}} = 2.75\times10^{-3} m = 2.75 \text{ mm}$$

01(d).

Sol: We know that $\sigma_{0^o} + \sigma_{90^o} = \sigma_{30^o} + \sigma_{120^o}$



$\Rightarrow \sigma_x + \sigma_y = \sigma_{30^o} + \sigma_{120^o} = 20 + (-80) = -60$

$\Rightarrow \sigma_x = -\sigma_y - 60 \rightarrow$ (i)

Further we have

$$\sigma_\theta = \left(\frac{\sigma_x + \sigma_y}{2}\right) + \left(\frac{\sigma_x - \sigma_y}{2}\right)\cos 2\theta + \tau_{xy}\sin 60^o$$

For $\theta = 30^o$,

$$\sigma_{30^o} = -\frac{60}{2} + \left(\frac{-\sigma_y - 60 - \sigma_y}{2}\right)\cos 60^o + \tau_{xy}\sin 60^o$$

$$\Rightarrow 20 = -30 - (\sigma_y + 30) \times \frac{1}{2} + \tau_{xy}\frac{\sqrt{3}}{2}$$

$$\Rightarrow 65 = \frac{-\sigma_y + \tau_{xy}\sqrt{3}}{2} \Rightarrow -\sigma_y + \tau_{xy}\sqrt{3} = 130 \rightarrow \text{(ii)}$$

For $\theta = 60^o$

$$\sigma_{60^o} = \frac{-60}{2} + \left(\frac{-\sigma_y - 60 - \sigma_y}{2}\right)\cos 120^o + \tau_{xy}\sin 120^o$$

$$30 = -30 - (\sigma_y + 30)\left(\frac{-1}{2}\right) + \tau_{xy}\left(\frac{\sqrt{3}}{2}\right)$$

$$45 = \frac{\sigma_y + \tau_{xy}\sqrt{3}}{2} \Rightarrow \sigma_y + \tau_{xy}\sqrt{3} = 90$$

Solving equation (ii) and (iii) we get

$\tau_{xy} = 63.51$ MPa and $\sigma_y = -20$ MPa

Now, from equation (i), we have

$\sigma_x + \sigma_y = -60 \Rightarrow \sigma_x - 20 = -60$

$\Rightarrow \sigma_x = -40$ MPa

Thus, we have

$\sigma_x = -40$ MPa

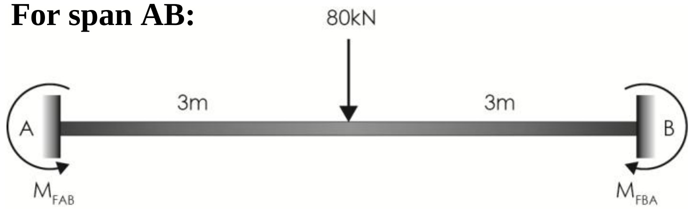
$\sigma_y = -20$ MPa

$\tau_{xy} = 63.51$ MPa

01. (e)

Sol: Fixed end moments:

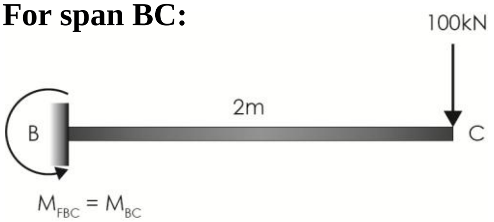
For span AB:



$$M_{FAB} = \frac{-WL}{8} = \frac{-80\times 6}{8} = -60\text{kN}-\text{m}$$

$$M_{FBA} = \frac{+WL}{8} = 60\text{kN}-\text{m}$$

For span BC:



$M_{FBC} = M_{BC} = -100 \times 2 = -200$ kN-m

Boundary conditions:

$\theta_A = 0$ [Fixed support]

Slope deflection equation:

$$M_{AB} = M_{FAB} + \frac{2EI}{L}(2\theta_A^{\nearrow 0} + \theta_B)$$

$$= -60 + \frac{2EI}{6}\theta_B \quad \text{…. (1)}$$

$$M_{BA} = M_{FBA} + \frac{2EI}{L}(2\theta_B + \theta_A)$$

$$= 60 + \frac{4EI\theta_B}{6} \quad \text{…. (2)}$$

Joint equilibrium equation:

$\sum M_B = 0$

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

$$60 + \frac{4EI\theta_B}{6} - 200 = 0$$

$$\theta_B = \frac{210}{EI}$$

Final moments:

$$M_{AB} = -60 + \frac{2EI}{6}\left[\frac{210}{EI}\right]$$

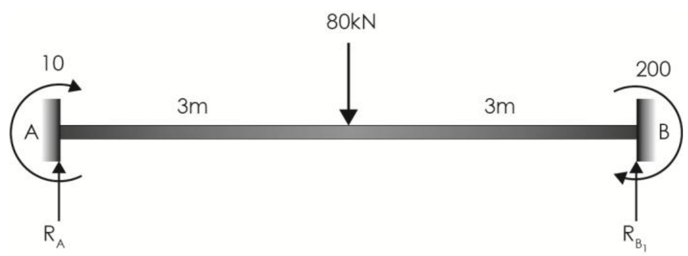
$= 10$ kN-m

$$M_{BA} = 60 + \frac{4EI}{6}\left[\frac{210}{EI}\right]$$

$= 200$ kN-m

$M_{BC} = -200$ kN-m

$M_{CB} = 0$



Support reactions:

$\sum V = 0$

$R_A + R_{B1} = 80$

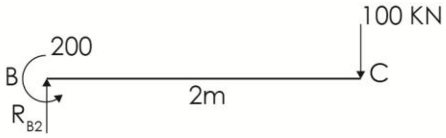
Taking moment about 'A'

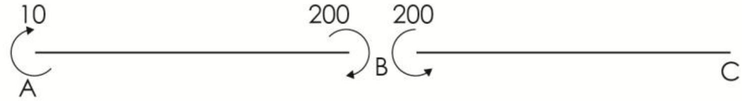
$\sum M_A = 0$ (↺ –ve ↻ +ve)

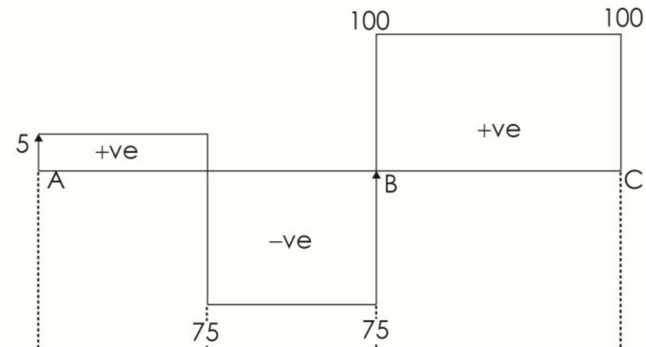
$-R_{B1} \times 6 + 200 + 80 \times 3 + 10 = 0$

$R_{B1} = 75$ kN (↑)

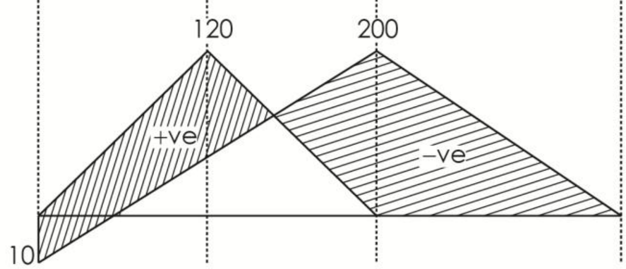
$R_A = 5$ kN (↑)







Shear Force Diagram



Bending moment diagram

02(a)(i).

Sol: W = 35 kN

Speed, V = 3.6 kmph = 1 m/s

Rope length = L = 60 m = 60×10^3 mm

d = 40 mm, E = 2.1×10^5 N/mm^2

$$A = \frac{\pi}{4}d^2 = \frac{\pi}{4}(40)^2 = 1256.6 \text{ mm}^2$$

As the chain get jammed, K.E of wagon is transformed into S.E of rope

$\therefore$ K.E of wagon $= \frac{1}{2}mV^2 = \frac{1}{2}\left(\frac{W}{g}\right)V^2$

$$= \frac{1}{2} \times \left(\frac{35\times10^3}{9.81}\right)1^2$$

$$= 1783.9 \text{ N-m} = 1783.9 \times 10^3 \text{ N-mm}$$

S.E in rope = $U = \frac{\sigma^2}{2E} \times \text{volume} = \frac{\sigma^2}{2E} \times A \times L$

$$= \frac{\sigma^2 \times 1256.6 \times 60 \times 10^3}{2 \times 2.1 \times 10^5}$$

$$= 179.52\sigma^2$$

$$1783.9 \times 10^3 = 179.52\ \sigma^2$$

$$\sigma = 99.68 \text{ N/mm}^2$$

$\therefore$ Instantaneous elongation = $\delta\ell_{inst} = \frac{\sigma L}{E} = \frac{99.68 \times 60 \times 10^3}{2.1\times10^5}$

$$= 28.48 \text{ mm}$$

02(a)(ii).

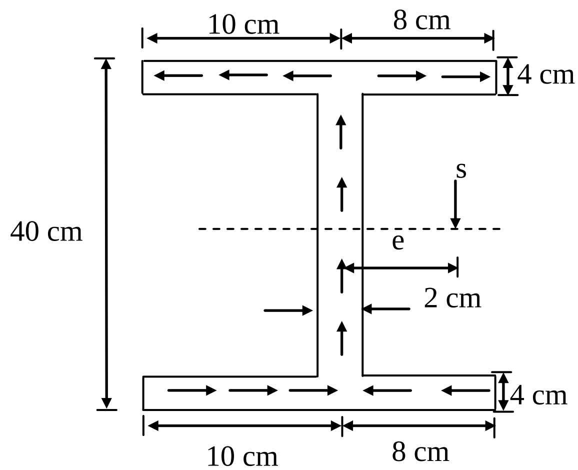
Sol: Given:

$t_1 = 4$ cm

$b_1 = 8$ cm

$b_2 = 10$ cm

$h = 40 - 4 = 36$ cm



$$I_{xx} = \frac{18\times40^3}{12} - \frac{16\times32^3}{12}$$

$$= 52309.33 \text{ cm}^4$$

$\therefore$ We have $e = \frac{t_1h^2\left(b_2^2 - b_1^2\right)}{4\ I_{xx}}$

$$e = \frac{4\times36^2\left(10^2 - 8^2\right)}{4\times52309.33} = 0.8919 \text{ cm}$$

02(b).

Sol: Fixed end moments:

$$M_{FAB} = +\frac{M}{4} = \frac{100}{4} = 25\text{kN}-\text{m}$$

$$M_{FBA} = +\frac{100}{4} = 25\text{kN}-\text{m}$$

$$M_{FBC} = -\frac{w\ell^2}{12} - \frac{w_1a_1b_1^2}{\ell^2} - \frac{w_2a_2b_1^2}{\ell^2}$$

$$= -\frac{10\times12^2}{12} - \frac{40\times3\times9^2}{12^2} - \frac{40\times9\times3^2}{12^2}$$

$$= -120 - 67.5 - 22.5 = -210 \text{ kN-m}$$

$$M_{FCB} = +\frac{w\ell^2}{12} + \frac{40\times3^2\times9}{12^2} + \frac{40\times9^2\times3}{12^2}$$

$$= 120 + 22.5 + 67.5 = 210 \text{ kN-m}$$

$$M_{FCD} = \frac{-Wab^2}{\ell^2} = -\frac{20\times6\times3^2}{9^2} = -80\text{kN}-\text{m}$$

$$M_{FDC} = \frac{wa^2b}{\ell^2} = \frac{120\times6^2\times3}{9^2} = 160\text{ kN}-\text{m}$$

Joint	Members	Relative stiffness (RS)	TS	DF
B	BA	$\frac{I}{6} = \frac{2I}{12}$	$\frac{2I}{12} + \frac{I}{12}$	$\frac{2I/12}{I/4} = \frac{2}{3}$
	BC	$\frac{I}{12}$	$= \frac{3I}{12} = \frac{I}{4}$	$\frac{I/12}{I/4} = \frac{1}{3}$
C	CB	$\frac{I}{12}$	$\frac{I}{12} + \frac{I}{12}$	$\frac{I/12}{I/4} = \frac{1}{2}$
	CD	$\frac{3}{4}\times\frac{I}{9} = \frac{I}{12}$	= 2I/12 = I/6	$\frac{I/12}{I/6} = \frac{1}{2}$

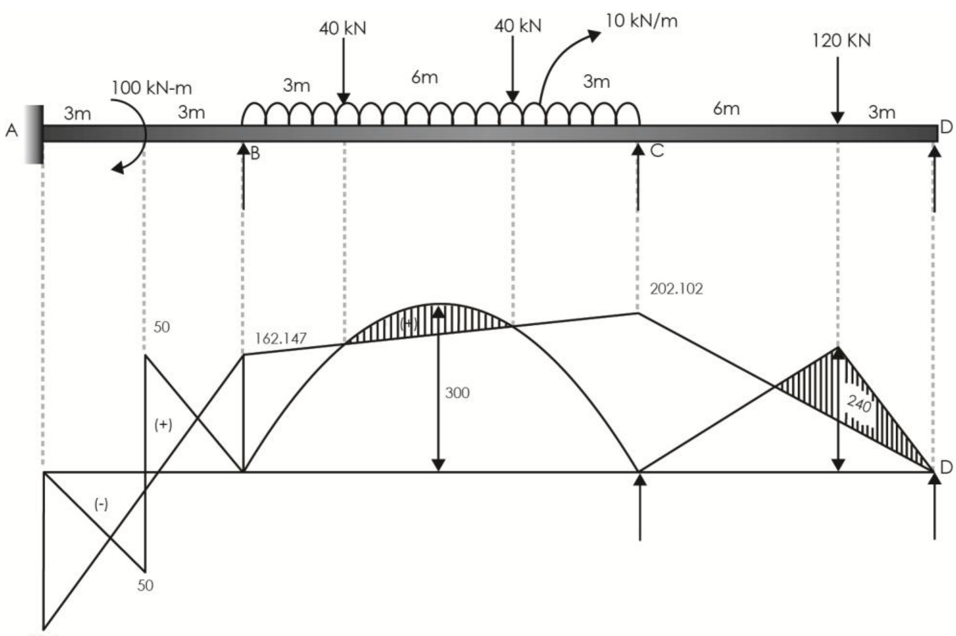
		2/3	1/3		½	1/2
	A	B		C		D
Fixed end moment	25	25	–210	210	–80	160
Release 'D' & carry over to C					–80 ← (1/2)	–160
Initial moments	25	25	–210	210	–160	0
Balance		123.333	61.67	–25	–25	
Carry over	61.666	–12.5	–12.5	30.835		
Balance		+8.33	4.167	–15.417	–15.417	
Carry over	4.165		–7.708	2.083		
Balance		+5.138	+2.569	–1.041	–1.041	
Carry over	2.569		–0.520	1.284		
Balance		+0.346	+0.173	–0.642	–0.642	
Final moments	93.4	162.147	–162.149	202.102	–202.1	0

Final Moments:

M_{AB}= 93.4 kN-m, M_{BA} = 162.147 kN-m

M_{BC} = –162.149 kN-m, M_{CB} = 202.102 kN-m

M_{CD} = –202.1 kN-m, M_{DC} = 0



02(c):

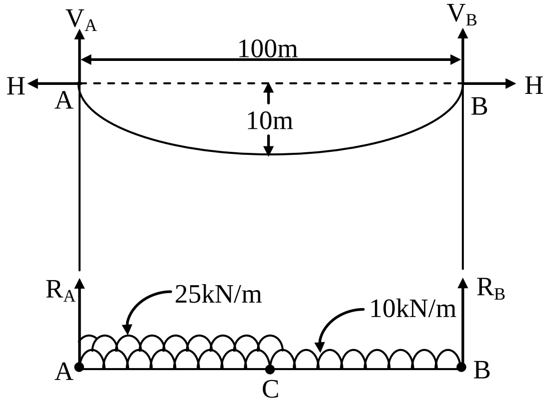
Sol: Width of roadway = 5 m

Dead load on the bridge = 4 kN/m^2

$\therefore$Dead load on each suspension girder/m = $\frac{(5\times1)\times4}{2} = 10\text{kN}/\text{m}$

Similarly, live load on the bridge = 10 kN/m^2

$\therefore$ Live load on each girder/m length = $\frac{(5\times1)(10)}{2} = 25\text{kN}/\text{m}$



Step-1: (Computation of R_A and R_B due to external loads)

Taking moments about A by considering the stiffening girder as a simply supported beam, we get

$$R_B(100) + 10\times100\times\frac{100}{2} + 25\left(50\times\frac{50}{2}\right) = 0$$

$\Rightarrow R_B$ = 812.50 kN

$R_A + R_B$ = 10 × 100 ÷ × 50 = 2250 kN

$\Rightarrow R_A$ = 2250 – 812.5 = 1437.5 kN

Step – 2: (Computation of Horizontal thrust H)

Taking moments about C and considering RHS, we get

$$-R_B(50) + 10\times50\times\frac{50}{2} + H(10) = 0$$

R_B = 812.5

$\Rightarrow$ H = 2812.5 kN

Step – 3: (Computation of BM at 25 m from left end due to external load)

BM due to external loads at 25 m from LHS = $R_A(25) - 10 \times 25 \times \frac{25}{2} - 25 \times 25 \times \frac{25}{2}$

$= 1437.5 \times 25 - 10 \times 25 \times \frac{25}{2} - 25 \times 25 \times \frac{25}{2}$

= 25000 kN-m

Step – 4: (Computation of depth of cable at 25 m from LHS)

$$y_{25} = \frac{4y_C}{l^2}(x)(l - x)$$

$$= \frac{4(10)}{100^2}(25)(75) = 7.5\text{m}$$

Step – 5: (Computation of BM due to H)

BM due to H at 25 m from LHS = $H(y_x)$

= 2812.5 (7.5) = 21093.75 kN-m

Step – 6: (Computation of net BM)

Net BM at 25 from LHS = 25000 – 21093.75

≈ 3906 kN-m

Step – 7: (Computation of equivalent udl w_e)

Let w_e be the equivalent uniformly distributed load transferred to the cable due to external loads.

Then $H = \frac{w_e l^2}{8y_C} \Rightarrow 2812.5 = \frac{w_e(100)^2}{8(10)}$

$\Rightarrow w_e = 22.5$ kN/m

Step – 8: (Computation of V_A and V_B due to w_e and T_{max})

$$V_A = V_B = \frac{w_e(l)}{2} = \frac{22.5(100)}{2} = 1125\,\text{kN}$$

∴ Max. tension the cable = $T_{max} = \sqrt{V^2 + H^2}$

$$T_{max} = \sqrt{1125^2 + 2812.5^2} \approx 3029\,\text{kN}$$

Step – 9: (Computation of SF at 25 m from LHS)

SF due to $w_C = V_A - w_e(25)$

$= 1125 - 22.5(25) = 562.5$ kN

SF due to external loads = $R_A - 10 \times 25 - 25 \times 25 = 1437.5 - 250 - 625 = 562.5$ kN

$\therefore$ Net SF at 25 m from LHS = 562.5 – 562.5 = 0

03(a)(i).

Sol: The different Non-Destructive Tests to determine the strength of concrete are as follows:

1. Rebound Hammer Test.
2. Ultrasonic Pulse Velocity Test.
3. Combined methods.
4. Surface Hardness Test.
5. Resonant Frequency Test.
6. Acoustic Emission Test.
7. Radioactive and Nuclear Methods.
8. Magnetic and Electrical Methods.

03(a)(ii).

Sol: The suitability of coarse aggregates based on their shape and size can be determined by the Flakiness Index and Elongation Index Tests.

Flakiness Index Test:

Flakiness Index is defined as the ratio of weight of aggregates whose least dimension (thickness) is less than 0.6 times of their mean dimension to the Weight of the sample taken.

$$\text{F.I} = \frac{\text{Weight of aggregates whose least dimension is less than 0.6 times of their mean dimension}}{\text{Weight of sample taken}} \times 100$$

Elongation Index Test:

Elongation index test must be performed only on non-flaky aggregates.

Elongation Index is defined as the ratio of weight of aggregates whose greatest dimension (length) is greater than 1.8 times of their mean dimension to the weight of the sample taken.

EI

$$= \frac{\text{Weight of aggregates whose greatest dimension is greater than 1.8 times of their mean dimension}}{\text{Weight of sample taken}} \times 100$$

Both F.I and E.I must be less than or equal to 30%.

03(a)(iii).

Sol: Ground Granulated Blast Furnace Slag:

- Molten blast furnace slag is a waste product produced in the manufacturing of pig iron.
- The rapid quenching of this molten blast furnace slag with water produces a glassy granular product called as granulated blast furnace slag which has oxides of Lime, Silica and Alumina. It increases the strength of concrete and also increases the resistance against chemical attack.

Fly Ash:

- It is the residue produced during the combustion of coal or any other form of coal.
- There are two ways in which fly ash can be used.

a) In the manufacture of PPC.

b) As admixture, which decrease segregation, beading and shrinkage effects in concrete.

Silica Fume:

- It is a byproduct of semi-conductor industry.
- It has high specific surface area and high silica content.
- Thus making it one of the most effective pozzolanic material.

Rice Husk Ash:

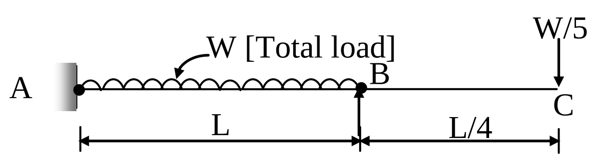
- It is produced by controlled burning of rice husk.
- It also possess pozzolanic properties.
- It is patented under the name Agro Silica.

Surkhi:

- Surkhi is nothing but powdered form of clay brick.
- Because of it being rich in silica, it is also used as a pozzolanic material.

03(b).

Sol:



- Static indeterminacy of beam

$D_S = r - s$ where,

r = No. of unknown reactions = 3

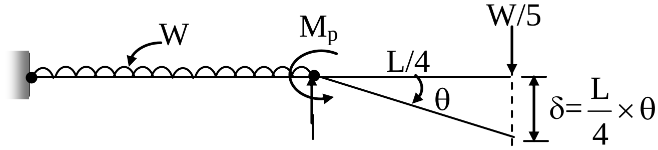
$D_S = 3 - 2 = 1$ s = No. of equilibrium equation = 2

- No of possible plastic hinges N = 3
- Minimum no. of plastic hinges required to form a mechanism 'n' = $D_s + 1 = 2$
- No. of independent mechanism $I = N - D_s$

$= 3 - 1$

$= 2$ [2 beam mechanism]

1st beam mechanism : Collapse of the cantilever part due to a plastic theory hinge at 'B'



External work done 'W_e' = load × Displacement under the load

$$W_e = \frac{W}{5} \times \frac{L}{4}\theta$$

Internal work done 'W_i' = Moment × Rotation = $M_p\theta$

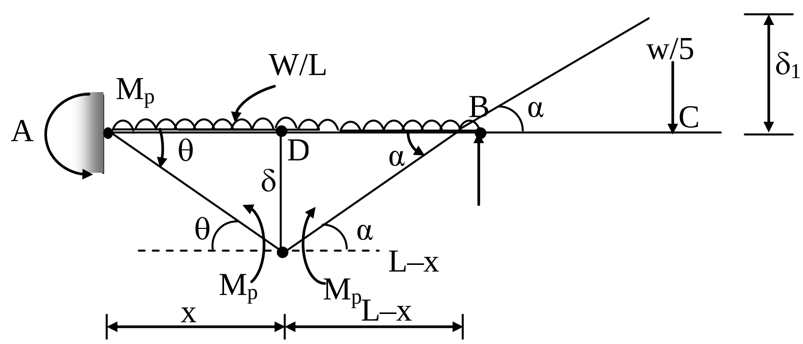
External work done = Internal work done

$$\frac{W}{5} \times \frac{L}{4}\theta = M_P\theta$$

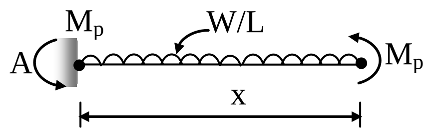
$$W = \frac{20M_P}{L}$$

2nd beam mechanism: Collapse of the span AB due to plastic hinges at the fixed end 'A' and at the section of maximum sagging moment.

- Let a plastic hinge be developed at A and at a section 'D' distance 'x' from support 'A'.



We know shear force at 'D' equal to zero for the equilibrium of AD, taking moment about 'A'

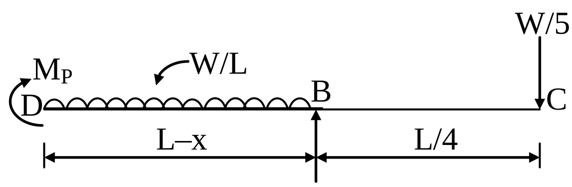


$\Sigma M_A = 0$ ↳–Ve ↲+Ve

$$-M_P + \frac{W}{L}\times\frac{x^2}{2} - M_p = 0$$

$$M_P = \frac{Wx^2}{4L} \rightarrow (1)$$

For the equilibrium of DBC, taking moment about B



$\Sigma M_B = 0$ ↳–Ve ↲+Ve

$$M_P - \frac{W}{L}\frac{(L-x)^2}{2} + \frac{w}{5}\times\frac{L}{4} = 0$$

$$M_P = \frac{W}{L}\frac{(L-x)^2}{2} - \frac{wL}{20} \qquad \rightarrow (2)$$

Equating (1) & (2)

$$\frac{Wx^2}{4L} = \frac{W(L-x)^2}{2L} - \frac{WL}{20}$$

Simplifying, we get $5x^2 - 20Lx + 9L^2 = 0$

$$x = \frac{20L - \sqrt{400L^2 - 180L^2}}{10} = 0.5168L$$

Substituting the above value of x in equation (1)

$$M_p = \frac{W(0.5168L)^2}{4L}$$

$$W = \frac{14.97M_p}{L}$$

Actual value of the collapse load is the lesser of the two values of W obtained above.

$\therefore$ Actual value of the collapse load 'W' = $\frac{14.97M_p}{L}$

03(c).

Sol: Reactions:

$\Sigma M_A = 0$

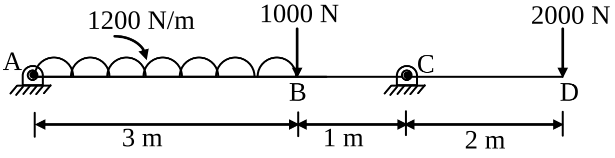
$\Rightarrow (1200 \times 3 \times 1.5) + (1000 \times 3) + (2000 \times 6) = 4R_C$

$\Rightarrow R_C = 5100$ N

$\Sigma F_y = 0$

$\Rightarrow R_A + R_C = (1200 \times 3) + 1000 + 2000$

$\Rightarrow R_A = 1500$ N



Shear Force Calculations:

AB: F = 1500 – 1200 x = Linear

$F_A = F_{@x = 0} = 1500$ N

$F_B = F_{@x = 3} = 1500 - 1200 \times 3 = -2100$ N

BC: F = 1500 – 1200 × 3 – 1000 = –3100 N

$F_B = F_C$ –3100 N = Constant

CD: F = 1500 – 1200 × 3 – 1000 + 5100

$\Rightarrow F = +2000$

$F_C + F_D = +2000$ N = Constant

Bending Moment Calculations:

AB : M = 1500 x – 1200x (x/2)

$M = 1500x - 600x^2$

$M_A = M_{@x = 0} = 0$

$M_B = 1500 \times 3 - 600 \times 3^2 = -900$ Nm

Location of point E $\Rightarrow$ F = 0

$\Rightarrow 1500 - 1200x = 0$

$\Rightarrow x = 1.25$ m from A

$M_E = M_{@x = 1.25} = 1500 \times 1.25 - 600 \times 1.25^2$

$\Rightarrow M_E = 937.5$ Nm

BC: $M = 1500\,x - 3600\,(x - 1.5) - 1000\,(x-3)$

$\Rightarrow M = -3100\,x + 5400 + 3000 = -3100x + 8400$

$M_{@C} = M_{@x = 4} = -400$ Nm

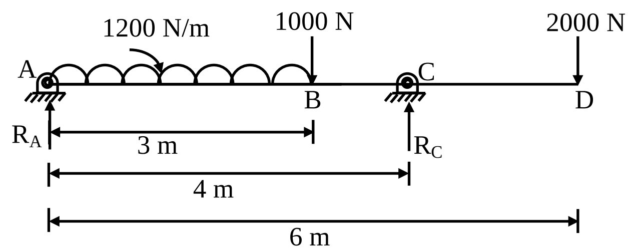
Location of point of Contraflexure:

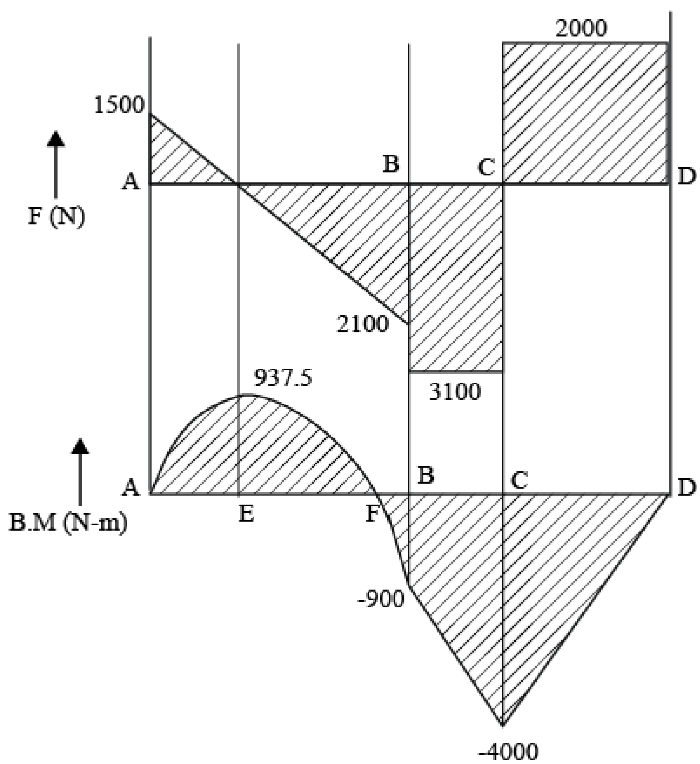
$M_F = 0 \Rightarrow 1500\,x - 600x^2 = 0$

$[1500 - 600x]x = 0$

$\Rightarrow x = 0$

$\Rightarrow x = 2.5$ m from A





04(a).

Sol: Given mix proportion

1:1.12:2.1 (weigh batching), w/c = 0.4

$w_{FA} = 1.12\ w_c$ $w_w = 0.4w_c$

$w_{CA} = 2.1\ w_C$

To prepare 1 m^3 of concrete

$$1 = \frac{w_w}{1\times 9.81} + \frac{w_c}{3.15\times 9.81} + \frac{w_{FA}}{2.6\times 9.81} + \frac{w_{CA}}{2.5\times 9.81}$$

$$1 = \frac{0.4w_C}{1\times 9.81} + \frac{w_C}{3.15\times 9.81} + \frac{1.12w_C}{2.6\times 9.81} + \frac{2.1w_C}{2.5\times 9.81}$$

$\Rightarrow w_C = 4.934$ kN = 503 kg $w_w = 0.4w_C$

$$V_C = \frac{4.934}{14.4}\,m^3 = 0.343\ m^3 \qquad = 0.4 \times 4.934\ \text{kN}$$

$= 1.974$ kN = 201 kg

$\Rightarrow w_{FA} = 1.12\ w_C$

$= 1.12 \times 4.934$

$= 5.526$ kN = 563 kg

$$V_{FA} = \frac{5.526}{16.2}\,m^3 = 0.341\ m^3$$

$\Rightarrow w_{CA} = 2.1\ w_C$

$= 2.1 \times 4.934$

$= 10.361$ kN = 1056 kg

$$V_{CA} = \frac{10.361}{15.8}\,m^3 = 0.656\ m^3$$

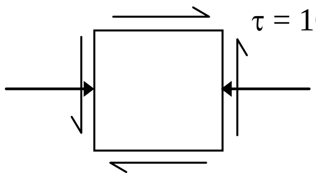
Mix proportion for this concrete based on volume batching will be

$$1 : \frac{0.341}{0.343} : \frac{0.656}{0.343}$$

= 1:0.99:1.91

04(b).

Sol:



Principal stresses are:

$$\sigma_{1,2} = \frac{\sigma_x}{2} \pm \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \tau_{xy}^2}$$

$$= -\frac{120}{2} \pm \sqrt{\left(-\frac{120}{2}\right)^2 + 100^2}$$

$= -60 \pm 116.60$

$\therefore \quad \sigma_1 = 56.60 \frac{N}{mm^2}$

$\sigma_2 = -176.60$ MPa

(i) Maximum principal stress theory:

$\sigma_1 \le S_{yt}$ (tension)

$\sigma_2 \le S_{yc}$ (Compression)

Here $\sigma_1, \sigma_2 \le 250$ MPa

$\therefore$ Material is safe.

(ii) Maximum principal strain theory:

Tension:

$$\varepsilon_1 \le \frac{S_{yt}}{E}$$

$\sigma_1 - \mu\sigma_2 \le S_{yt}$

$56.60 - (0.3)(-176.60) \le 250$

$109.58 \le 250$

$\therefore$ Material is safe.

Compression:

$\sigma_2 - \mu\sigma_1 \le S_{yt}$

$-176.60 - (0.3)(-56.60) \le 250$

$-193.56 \le 250$

$\therefore$ Material is safe.

(iii) Maximum shear stress theory:

$\sigma_1 - \sigma_2 \le S_{yt}$

$56.60 - (-176.60) \le 250$

$233.2 \le 250$

$\therefore$ Material is safe.

(iv) Maximum distortional energy theory:

$\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2 \le S_{yt}^2$

$(56.60)^2 + (-176.60)^2 - (56.60)(-176.60) \le 250^2$

$3.20 \times 10^3 + 31.187 \times 10^3 + 9.99 \times 10^3 \le 250^2$

$44.377 \times 10^3 \le 62.5 \times 10^3$

$44.377 \le 62.5$

$\therefore$ Material is safe.

04(c)(i).

Sol: Given both bars are of same length and material (L, E = constant)

d = 80 mm; a = 80 mm

$$U_1 = \frac{\sigma_1^2}{2E} \times A \times L = \frac{\sigma_1^2}{2E} \times \frac{\pi}{4} \times 80^2 \times L$$

$$U_2 = \frac{\sigma_2^2}{2E} \times A \times L = \frac{\sigma_2^2}{2E} \times 80^2 \times L$$

Both have same strain energy under axial forces

$\therefore U_1 = U_2$

$$\frac{\sigma_1^2}{2E} \times \frac{\pi}{4} \times 80^2 \times L = \frac{\sigma_2^2}{2E} \times 80^2 \times L$$

$$\therefore \frac{\sigma_1}{\sigma_2} = \sqrt{\frac{4}{\pi}} = 1.128$$

$$\therefore \frac{\sigma_1}{\sigma_2} = 1.128$$

04(c)(ii).

Sol: Given details:

k_1 = 2000 N/m, k_2 = 1500 N/m

k_3 = 3000 N/m, $k_4 = k_5$ = 500 N/m

f = 10 Hz.

The springs k_1, k_2 and k_3 are in series. Their equivalent stiffness

$$\frac{1}{k_{e1}} = \frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_3} = \frac{1}{2000} + \frac{1}{1500} + \frac{1}{3000}$$

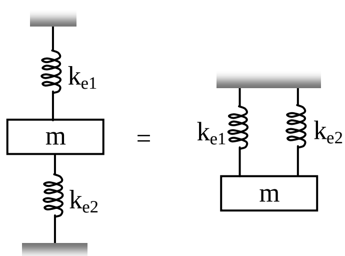
k_{e1} = 666.67 N/m

The two lower springs k_4 and k_5 are connected in parallel, so their equivalent stiffness

$k_{e2} = k_4 + k_5$ = 500 + 500 = 1000 N/m

Again these two equivalent springs are in parallel,

$k_e = k_{e1} + k_{e2}$ = 666.67 + 1000



$\Rightarrow k_e$ = 1666.67 N/m

$$f = \frac{\omega_n}{2\pi}$$

$\Rightarrow \omega_n = 2\pi f = 2\pi(10)$

$\Rightarrow \omega_n$ = 62.83 rad/s

But $\omega_n = \sqrt{\frac{k}{m}}$

$\Rightarrow \omega_n^2 = \frac{k}{m}$

$\Rightarrow m = \frac{k_e}{\omega_n^2} = \frac{1666.67}{(62.83)^2}$

$= 26.52$ kg

05.(a)

Sol: Step:1

Diameter of rivets:

Nominal diameter of rivet = 22 m

Gross diameter of rivet = 23.5 mm

Step:2

Rivet value (R):

Strength of power driven shop rivet in single shear $\left(\frac{\pi}{4} \times \frac{(23.5)^2 \times 100}{1000}\right) = 43.35$ kN

Assume that strength of rivet in bearing is greater than strength of rivet is single shear

Step:3

Direct shear in each rivet:

Number of rivets in bracket connection is 12

$F_1 = (P/12)$ kN

External moment acting on bracket connection M = (P × 250) kN-mm

Force in extreme rivet due to twisting (torsional) moment $F_2 = \left(\frac{P.250 r_n}{\Sigma r^2}\right)$kN

Distance of centre to extreme rivet from centre of gravity of group of rivets

$R_n = (50^2 + 200^2)^{1/2} = 206.155$ mm

Horizontal distance of each rivet from centre of gravity of group of rivets, = 50 mm vertical distance of rivets

Rivets in first row above centre of gravity = 40 mm

Rivets in second = 120 mm

Rivets in topmost row = 200 mm

Other rivets are symmetrically placed

$\sum r^2 = [4(5^2 + 4^2) + 4\,(5^2 + 12^2) + 4$

$(5^2 + 20^2) \times 100\ \text{mm}^2 = 2540 \times 100\ \text{mm}^2$

$$F_2\left(\frac{P\times 250\times 206.2}{2540\times 100}\right) = 0.205\ P\ \text{kN}$$

Resultant of two forces,

$F = [F_{12} + F_{22} + 2F_1F_2 \cos\theta]^{1/2}$

$\cos\theta = (50/206.2)$

$$\therefore F = \left[\left(\frac{P}{12}\right)^2 + (0.205P)^2 + 2\times\frac{P}{12}\times\frac{0.205P\times 50}{206.2}\right]^{1/2}$$

$$= 0.239\,P\ \text{kN}$$

As the resultant force in the extreme rivet is not to exceed the rivet value. Therefore the resultant force, F may be equated to rivet, R

$0.239\ P = 43.35$ kN

$P = 181.38$ kN

Hence the maximum load which can be applied is 181.38 kN

05(b).

Sol: The manufacturer would like to find the changes in objective function for changes made in the objective function coefficients. We can find out the range of an objective function coefficient given other coefficients so that the optimal point remains the same in figure, we can conclude that as long as the slope of objective function (line shown in broken dashes), i.e., ratio c_1/c_2 occupies a value in between the slopes of the two lines(line 1, line 2) , the optimal point won't change. In extreme cases, the c_1/c_2 value could be the slopes of either line. This situation would correspond to multiple solutions for the problem. We can also state the above condition in terms of c_2/c_1. Thus to assure that the optimal point is still at E(12,14) for the window-door problem, we can write the following conditions.

In terms of c_1/c_2

Slope of line $x_1 + 4x_2 = 68 \le c_1/c_2 \le$ slope of line $3x_1 + 2x_2 = 64$

i.e $\frac{1}{4} \le \frac{c_1}{c_2} \le \frac{3}{2}$

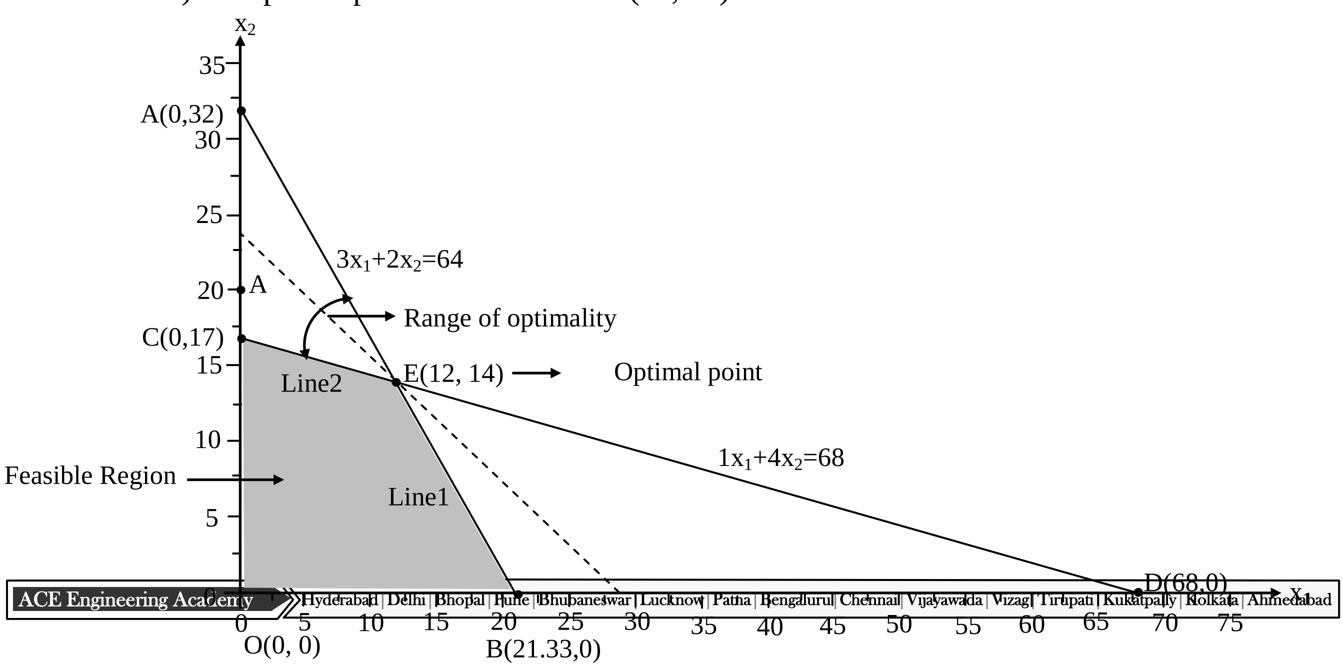
Now given the value of c_2, we can find out the range of values for c_1 which would still assure that the optimal point is still E(12, 14). Knowing the objective value coefficients, we can easily find the value of objective function. For example for $c_2 = 600$, the range of values for c_1 would be: $\frac{1}{4} \times 600 \le c_1 \le \frac{3}{2} \times 6600$, in other words, for any value of c_1 between 150 and 900 (both values included) the optimal point would still be E(12, 14).

In terms of c_2/c_1

Slope of line $3x_1 + 2x_2 = 64 \le c_2/c_1 \le$ slope of line $x_1 + 4x_2 = 68$

That is $\frac{2}{3} \le \frac{c_2}{c_1} \le \frac{4}{1}$

Similarly given the value of c_1, we can find out the range of values for c_2 which would still assure that the optimal point is still E(12, 14). Knowing the objective values coefficients, we can easily find the value of objective function. For example for $c_1 = 500$, the range of values for c_2 would be: $\frac{2}{3} \times 500 \le c_2 \le 4 \times 500$, in other words for any value of c_2 between 333.33 and 2000 (both values included) the optimal point would still be E(12, 14)



05(c).

Sol: Given b = 300 mm, D = 500 mm, effective cover = 40 mm

Effective depth d = 500 – 40 = 460 mm

A_{st} = 4 – 16 mm ϕ = 804.24 mm^2

Shear force V = 210 kN

f_{ck} = 25 MPa, f_y = 415 MPa

Step 1 :

Actual depth of neutral axis

$0.36\, f_{ck}\, bx_{act} = 0.87\, f_y\, A_{st}$

$$x_{act} = \frac{0.87 \times 415 \times 804.24}{0.36 \times 25 \times 300}$$

$= 107.54$ mm

Balanced depth of neutral axis $x_{bal} = 0.48$ d

$= 0.48 \times 460$

$= 220.8$ mm $> x_{act}$

$\therefore$ Under-reinforced section

Step – 2:

Moment of resistance:

MR = $0.87\, f_y\, A_{st}(d - 0.42\, x_{act})$

$= 0.87 \times 415 \times 804.24\, (460 - 0.42 \times 107.54)$

$= 120.45 \times 10^6$ Nmm = 120. 45 kNm

Factored shear force $V_u = 1.5 \times 210 = 315$ kN

Step 3 :

For M25 grade concrete and Fe 415 steel

$\tau_{bd} = 1.6(1.4) = 2.24$ MPa

Development length $L_d = \dfrac{\phi \sigma_{st}}{4\tau_{bd}}$

$$= \frac{16 \times 0.87 \times 415}{4 \times 2.24} = 644.73 \text{ mm}$$

Anchorage length:

$$L_d \le \frac{1.3M_1}{V} + L_o \rightarrow \text{for simply supported beam}$$

$\Rightarrow L_{d,\,actual} = L_d - A_V$

Assuming 90^o bend $\Rightarrow 644.73 - 8\,\phi$

$\Rightarrow L_{d,\,act} = 644.73 - 8 \times 16$

$= 516.73$ mm

$$\therefore L_o = 516.73 - \frac{1.3 \times 120.45 \times 10^6}{315 \times 10^3}$$

$= 19.63$ mm

$\therefore L_o = 19.63$ mm

05.(d)

Sol: Step:1

Size of Weld:

Consider one face, size of weld = 8 mm

Effective throat thickness = (0.7×8) = 5.6 mm

Step:2

Properties of Welds:

Let $\bar{x}$ be the distance of centroid 'G' of weld group from left hand edge of the plate

$$\bar{x} = \left[\frac{(2 \times 200 \times 5.6 \times 100) + (200 \times 5.63 \times 0)}{(2 \times 200 \times 5.6) + (200 \times 5.6)}\right]$$

$= 66.7$ mm

Moment of inertia of weld group about xx-axis

$$I_{xx} = [2 \times 20 \times 0.56 \times 100^2 + \frac{1}{6} \times 0.56 \times 20^3] \times 10^4 = 4480 \text{ mm}^4$$

Moment of inertia of weld group about yy-axis

$$I_{yy} = \left[\frac{1}{12} \times 2 \times 0.56 \times 20^3 + 2 \times 20 \times 0.56(20 - 6.67)^2 + 20 \times 0.56 \times 6.72^2\right] \times 10^4 \text{mm}^4$$

$= 1493.2 \times 10^4 \text{ mm}^4$

Polar moment of inertia of weld group

$I_{zz} = (I_{xx} + I_{yy}) = (4480 + 1493.2)$

$= 5973.2 \times 10^4 \text{ mm}^4$

Distance to the extreme weld from the centroid of weld group

$r = [100^2 + 133.2^2]^{1/2} = 166.64$ mm,

$\cos\theta = (13.33/16.65)$

Let 2P be the maximum load which can be placed on the bracket

Load transmitted by each face = P

Step:3

Stresses in Welds:

Direct shear stress

$$\tau_v = \left(\frac{P}{(2\times 200 + 200)\times 5.6}\right)$$

$$= \left(\frac{(0.0297P)}{100}\right) \text{N/mm}^2$$

Twisting moment resisted by the weld group,

$T = P \times (133.3 + 100) = 233.3$ P N-mm

Maximum shear stress due to twisting,

$$p_b = \left(\frac{233.3P\times 166.64}{5973.2\times 104}\right) = \left(\frac{0.0650P}{100}\right)$$

Combined stress in the weld group,

$$p = \left(\frac{P}{100}\right)$$

$$\frac{P}{100}\times[(0.0297)^2 + (0.650)^2 + 2\times 0.0297\times 0.0650\,(13.33/16.65)]^{1/2}$$

$$= \left(\frac{0.0905P}{100}\right) N/mm^2$$

Step:4

Check for Combined Stress:

Combined stress should not exceed maximum permissible stress 110 N/mm^2

$$\therefore \left(\frac{0.0905P}{100}\right) = 100,\ P = 121488\ N$$

$\therefore$ P = 121.488 kN, and 2P = 242.97 kN

Maximum load which may be placed is 242.97 kN

05(e).

Sol: Given : B = 150 mm; D = 300 mm, P = 200 kN

e = 100 mm, l = 6 m, w = 6 kN/m

A = BD = 150 × 300 = 45000 mm^2

$$Z = \frac{BD^2}{6} = \frac{150 \times 300^2}{6} = 2.25 \times 10^6 mm^3$$

At mid span:

Maximum bending moment = $\frac{w\ell^2}{8} = \frac{6 \times 6^2}{8} = 27$ kNm

Direct stress due to prestressing force= $\frac{P}{A} = \frac{200 \times 10^3}{45000} = 4.44$ MPa

Flexural stress due to prestressing load = $\frac{M}{Z} = \frac{27 \times 10^6}{2.25 \times 10^6}$ = 12 MPa

Flexural stress due to prestress = $\frac{Pe}{Z} = \frac{200 \times 10^3 \times 100}{2.25 \times 10^6} = 8.88$ MPa

Resultant stress at

Top of midspan = 4.44 + 12 – 8.88 = 7.56 MPa

Bottom of midspan = 4.44 – 12 + 8.88 = 1.33 MPa

Shift of pressure line from cable line = $\frac{M}{P} = \frac{27 \times 10^6}{200 \times 10^3}$ =135 mm

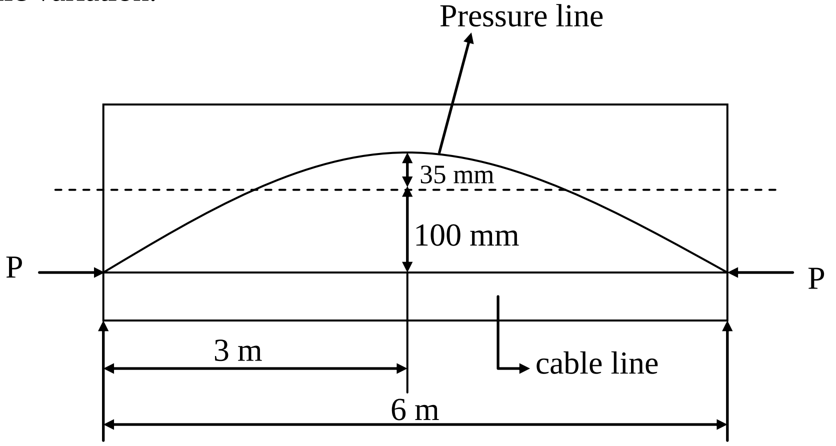
i.e., 135 – 100 = 35 mm above centroid of beam

At supports :

M = 0; P = 200 kN

i.e., shift of p lime from cable line = $\frac{M}{P} = 0$

Since 'P' is constant, 'M' follows parabolic variation, shift of 'p' line from cable line follows parabolic variation.



06(a)(i).

Sol:

The different types of losses are as below :

Pretensioning:

1. Elastic deformation concrete.   .
2. Relaxation of stress in steel.
3. Shrinkage of concrete.
4. Creep of concrete.

Post tensioning:

1. No loss due to elastic deformation if all the wires are simultaneously tensioned. If the bars are successively tensioned, there will be loss of prestress due to elastic deformation of concrete
2. Relaxation of stress in steel.
3. Shrinkage of concrete.
4. Creep of concrete.
5. Friction.

6. Anchorage slip.

In addition there may be losses due to sudden change in temperature especially in steam curing of pretensioned units.

1. Elastic deformation of concrete: When the prestress is applied to the concrete, an elastic shortening of concrete takes place. This results in an equal and simultaneous shortening of the prestressing steel.

Mathematically, loss due to elastic deformation = $m.f_c$

Where m = modular ratio = (E_s/E_c)

f_c = prestress in concrete at the level of steel.

$$= \frac{P}{A} + \frac{Pe}{I}(e)$$

2. Loss due shrinkage of concrete :-

Shrinkage is defined as change in volume of concrete members. It is dependent on humidity in atmosphere with passage of time but is unrelated to application of load.

Factors affecting shrinkage:-

1. Type of cement and aggregate.
2. Method of curing.

Loss of stress due to shrinkage = $\varepsilon_{cs} \times E_s$

Where

ε_{cs} = Total residual shrinkage strain having values of 3×10^{-4} for pre tensioned member

$$\frac{2\times10^{-4}}{\log_{10}(t+2)}$$ for post tensioning

t = age of concrete at transfer in days.

E_s = modulus of elasticity of steel.

3. Loss due to creep of concrete:-

Creep is the property of concrete by which it continuous to deform with time under sustained loads at unit stresses with in the accepted elastic range. *This in elastic deformation increases at a decreasing rate* during the time of loading and its total magnitude maybe several times as large as the short term elastic deformation.

- The strain due to creep vary with the magnitude of stress.

- It is a time dependent phenomenon.

Creep of concrete results in loss in steel stress.

Loss of stress due to creep can be calculated by the following two methods

1. **Ultimate creep strain method :-**

$\Delta\sigma_c = \varepsilon_{cc}\ E_s$

ε_{cc} = Ultimate creep strain for a sustained unit stress.

2. **Creep coefficient method :-**

$\Delta\sigma = \varepsilon_c . E_s = \phi . m f_c$

creep coefficient, $\phi = (\varepsilon_c/\varepsilon_e)$, $\varepsilon_c = \phi.\varepsilon_e$

ε_c = ultimate creep strain

ε_e = elastic creep strain

4. **Loss due to anchorage slip**:

(The loss during anchorage)

In most tensioning systems, when the cable is tensioned and the jack is released to transfer prestress to concert, the friction wedges employed to grip the *wires, slip over a small distance before the wires* are finally housed between the wedges. The magnitude of slip depends upon the type of wedge.

$\Delta\sigma = (E_s./L).\ \delta$

δ = slip of anchorage, mm

E_s = Mod. Elasticity of steel, N/mm^2

L = length of cable, mm

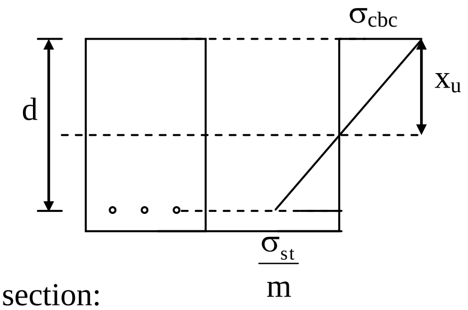
% loss is higher for short members than for comparatively longer ones.

No similar loss in RCC:

In the case of prestressed concrete, comprising of concrete and high tensile steel as basic components, both steel and concrete are stressed prior to the application of external loads. If such induced pre-stress in concrete is of compressive nature, it will balance the tensile stress produce in concrete surrounding steel due to external loads. Due to the induce of initial stress losses can be taking place in PSC which is not in case of RCC.

06(a)(ii).

Sol: For a reinforcement concrete beam of size 'b', effective depth 'd' stress variation is linear as per working stress method as shown below.



For a balanced section:

Both concrete and steel reach permissible stress simultaneously

From similar triangles: $\dfrac{d-x_u}{\dfrac{\sigma_{st}}{m}} = \dfrac{x_u}{\sigma_{cbc}}$

$$\Rightarrow \frac{d-x_u}{x_u} = \frac{\sigma_{st}}{m\sigma_{cbc}}$$

$$\Rightarrow \frac{d}{x_u} - 1 = \frac{\sigma_{st}}{m\sigma_{cbc}}$$

$$\Rightarrow \frac{d}{x_u} = 1 + \frac{\sigma_{st}}{m\sigma_{cbc}}$$

$$\Rightarrow x = \frac{d(m\sigma_{cbc})}{m\sigma_{cbc} + \sigma_{st}}$$

But $m = \dfrac{280}{3\sigma_{cbc}}$

$$\Rightarrow x = \frac{d\left(\dfrac{280}{3}\right)}{\left(\dfrac{280}{3} + \sigma_{st}\right)}$$

$$= \frac{d}{1 + \dfrac{3\sigma_{st}}{280}}$$

i.e., independent of grade of concrete

Hence proved

06(b).

Sol: Step 1:

Given data:

ISMB 350

Section Modulation $Z_x = 778.9\ cm^3$

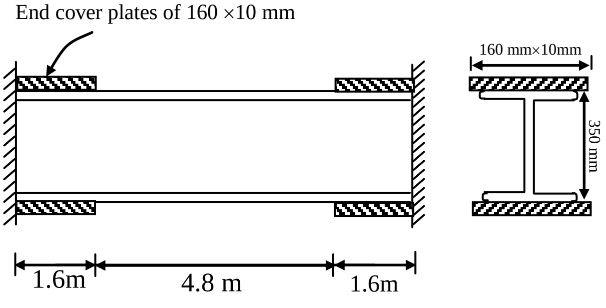
$= 778.9 \times 10^{-6}\ m^3$

Assume shape factor(S) = 1.12

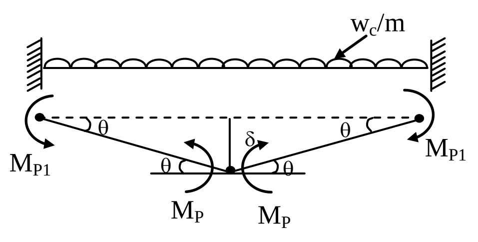
Yield stress $(f_y) = 250\ N/mm^2$

$= 250 \times 10^6\ N/m^2$

The entire data is shown in the fig. below



Step 2: Let the plastic moments at fixed end and at centre of beam be M_{P1} and M_P respectively.



Calculation of M_P(central 3m portion)

$M_P = f_y.Z_p$

$= (250 \times 10^6)(778.9 \times 10^{-6} \times 1.12)$

= 218092 N-m

Calculation of M_{P1} for end 1m length

$M_{P1} = f_y.Z_{P1}$

$Z_{P1} = Z_P$ of plates + Z_P of I-section

$$= \frac{[(16\times1)(18)(2)+778.9\times1.12]}{10^6}$$

$$= \left(\frac{576+872.37}{10^6}\right)m^3 = \left(\frac{1448.37}{10^6}\right)m^3$$

$$\therefore \quad M_{P1} = 250\times10^6\times\frac{1448.37}{10^6}$$

= 362092.5 N-m

Using Mechanism Method:

Internal Energy W_i

$$= M_{P1}.\theta + M_{P1}.\theta + M_P.\theta + M_P.\theta$$

$$= 2\theta[M_{P1}+M_P]$$

External Work W_e

$$= (w_c\times8)\left(\frac{0+\delta}{2}\right) = \frac{8w_c}{2}\times4\theta = 16\ w_c.\ \theta$$

According to virtual work principle

$w_e = w_i$

$$16\,w_c.\theta = 2\theta(M_{P1}+M_P)$$

$$w_c = \frac{2(M_{P1}+M_P)}{16}$$

$$= \frac{2(362092.5+218092)}{16}\times10^{-3} = 72.523\ \text{kN/m}$$

Hence, the collapse load is = 72.523 kN/m

06(c).

Sol: Given: b = 400 mm, D = 550 mm; l = 5 m

LL = 20 kN/m

f_{ck} = 25 MPa, f_y = 415 MPa

Step 1: Load calculation:

Dead load = γ BD

$= 25 \times 0.40 \times 0.55 = 5.5$ kN/m

Live load = 20 kN/m

Total load = 25.5 kN/m

Factored load = $1.5 \times 25.5 = 38.25$ kN/m

Step 2: Moment calculation:

Maximum Bending moment = $\frac{w\ell^2}{2}$ for cantilever beam

$M_u = 38.25 \times \frac{5^2}{2} = 478.125$ kNm

Effective depth = 550 – 50 = 500 mm

Moment of resistance of singly reinforced beam

$M_{u\,lim} = 0.138\, f_{ck}\, bd^2$

$= 0.138 \times 25 \times 400 \times 500^2 = 345$ kNm

$M_u > M_{u,\,lim}$

$\therefore$ Doubly reinforced section is required

Step 3: Reinforcement calculation:

Area of tensile reinforcement for $M_{u,\,lim}$, is A_{st1}

$M_{u,lim} = 0.87\, f_y\, A_{st1}\, (d - 0.42 \times x_{u\,max})$

$\Rightarrow 345 \times 10^6 = 0.87 \times 415 \times A_{st1}\, (500 - 0.42 \times 0.48 \times 500)$

$\Rightarrow A_{st,\,1} = 2393.65$ mm^2

Balance BM = $M_u - M_{u,\,lim}$

$M_u = 478.125 - 345 = 133.125$ kNm

Addition steel required is $A_{st,2}$

$M_{u2} = 0.87\, f_y\, A_{st}(d - d_e')$

$\Rightarrow 10^6 \times 133.125 = 0.87 \times 415 \times A_{st2}\, (500 - 50)$

$\Rightarrow \quad A_{st2} = 819.37$ mm^2

$\therefore$ Total Area of steel in tension zone = $A_{st1} + A_{st2}$

$= 2393.65 + 819.37 = 3213.02$ mm^2

$\therefore$ Number of 25 mm bars required = $\dfrac{3213.02}{\frac{\pi}{4}\times 25^2} = 6.54 \approx 7$ No's

Step 3:

Compression reinforcement:

Steel required in compression zone A_{sc}

$M_{u,2} = f_{sc}\,A_{sc}(d - d)$

$$\varepsilon_{sc} = 0.0035\left(1 - \frac{d'}{x_a\ max}\right)$$

$$= 0.0035\left(1 - \frac{50}{0.48\times 500}\right)$$

$= 2.77\times 10^{-3}$

$= 0.00277$

From table f_{sc} = 351.8 MPa

$\therefore$ 133.125 × 10^6 = 351.8 × A_{sc}(500-50)

$\therefore$ A_{sc} = 840.91 mm^2

Using 20 mm bars, number of bars required is

$$= \frac{840.91}{\frac{\pi}{4}\times 20^2} = 2.676 \approx 3 \text{ No's}$$

Step 4:

Minimum reinforcement:

$$\frac{A_{st\,min}}{bd} = \frac{0.85}{f_y}$$

$\Rightarrow A_{st,\,min} = \dfrac{0.85}{415}\times 400\times 500 = 409.63\ mm^2 < 3213.02\ mm^2$

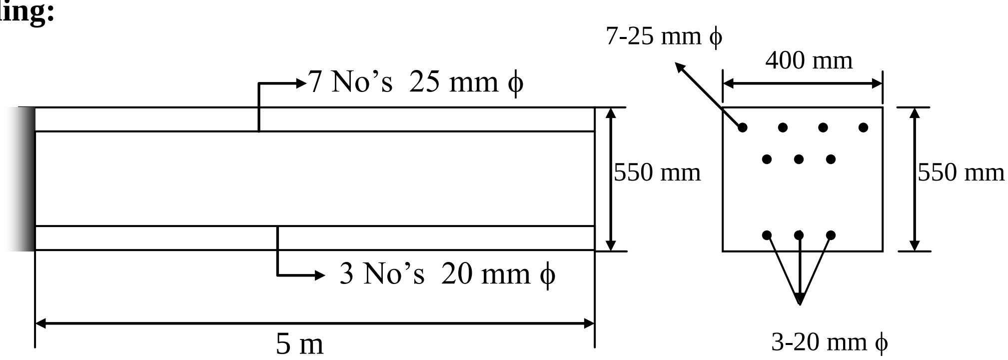
Hence ok

Maximum reinforcement = 0.04 × bD

= 0.04 × 400 × 550 = 8800 mm^2 > 3213.02 mm^2

Hence ok

Detailing:



07(a).

Sol: **Resource leveling** is defined as "A technique in which start and finish dates are adjusted based on resource limitation with the goal of balancing demand for resource's with the available supply". Resource leveling problem could be formulated as an optimization problem.

When performing project planning activities, the manager will attempt to schedule certain taken simultaneously. When more resources such as machines or people are needed than are available, or perhaps a specific person is needed in both tasks, the tasks will have to be rescheduled concurrently or even sequentially to manage the constraint. Project planning resource leveling is the process of resolving these conflicts. It can also be used to balance the workload of primary resource over the course of the project(s), usually at the expense of the traditional triple constraints (time, cost, scope). Leveling could result in a later project finish date if the tasks affected are in the critical path.

Project management structure can be divided into categories such as

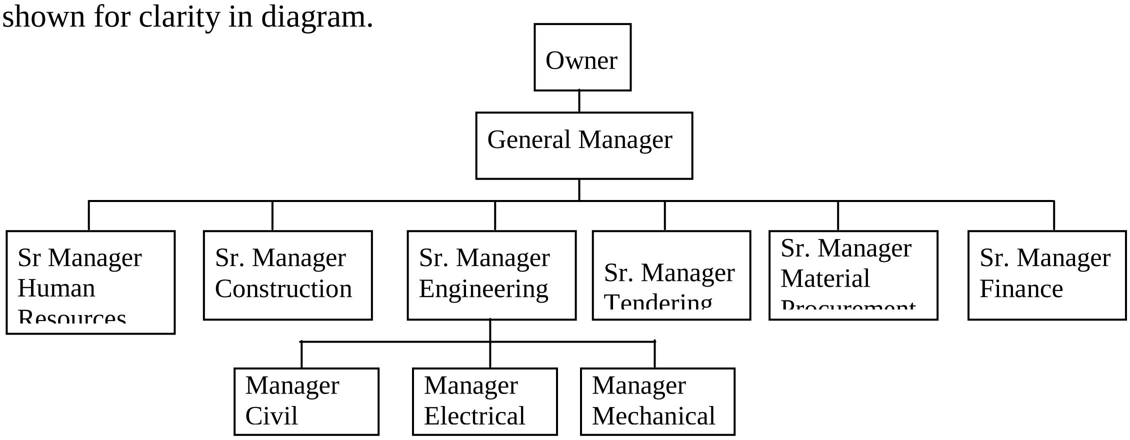
(i) Functional organisation

(ii) Pure project organisation

(iii) Matrix organization

(i) Functional organisation:

Traditionally, classical or functional organizations are marked by a vertical structure with long lines of communication and a long chain of command. A typical functional organization is shown in figure. In this form, each employee has one clear superior. For example, the general manager is reporting to the owner, the senior project manager is reporting to the general manager, and so on. The employees are grounded by specialty – for example, human resources development, construction, engineering, tendering, finance, and so on. Each of these specialty groups works under one executive. This groups are further subdivided into sections. For example, engineering can be subdivided into civil, electrical and mechanical sections. Sections under other groups are not shown for clarity in diagram.



Functional organization

In a functional organization, project related issues are resolved by the functional head. For example, construction-related issues would be sorted out by senior manager, construction. The functional organization structure assumes that the common bond supposed to be there between an employee and his superior would enhance the cooperation and effectiveness of the individual and the group.

Advantages:

- The degree of efficiency is high since the employees have to perform a limited number of activities
- There is a greater division of labor and, thus, the advantages of functional organizations are inherent in this structure.
- The specialized group can enhance the possibility of mass production.

Disadvantages :

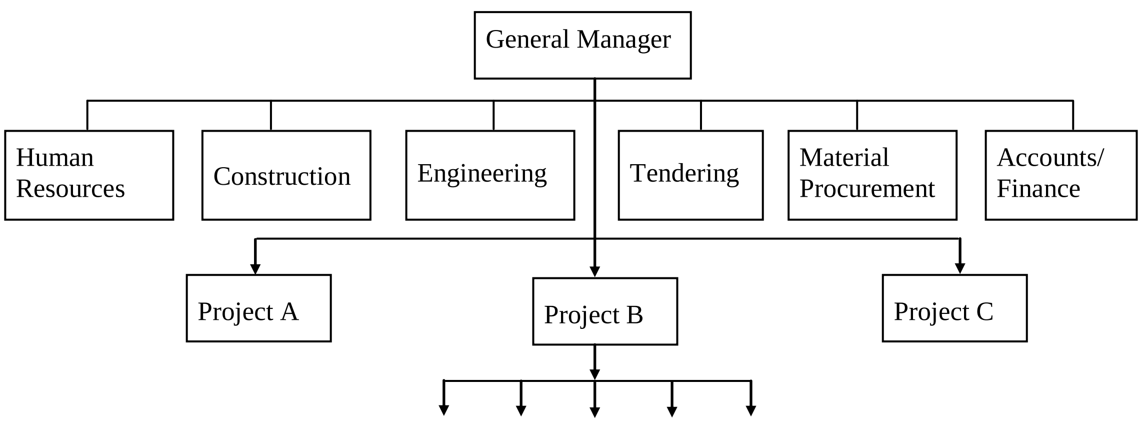
- The structure as such is unstable as it lacks disciplinary control.
- The structure is slightly complicated as it has several layers of sections.

- The responsibility for unsatisfactory results may be difficult to fix under such structure.
- There may be conflict among employees of equal rank.

In recent years, efforts have been made to reshape functional organizations having long lines of communication, by incorporating a horizontal structure with the aim of creating a shorter line of command. Such horizontally structured organizations are seen to be more flexible and able to quickly and effectively adapt to a changed environment.

(ii) Pure project organisation

Pure project or product organizations can be formed to support a steady flow of ongoing projects. One such typical pure project organization is shown in figure. In a project organization employees are grouped by project. The majority of the organizations resources is



Project organization

directed towards successful completion of projects. The project managers enjoy a great deal of independence and authority. In such structures, the different organizational units called departments either report directly to the project manager or provide supporting roles to the projects.

Advantages:

- The project manager maintains complete authority over the project and has maximum control over the project.

- The lines of communication are strong and open, and the system is highly flexible and capable of rapid reaction times. Thus, the structure can react quickly to the special and changing project needs.
- The project is the only real concern of the project employees. The pure project structure provides a unity of purpose in terms of effectiveness. It brings together all the administrative, technical and support personnel needed to bring a project from the early stages of development through to operational use.
- The appraisal of employee is based upon the performance of the project.
- The focus of resources is towards the achievement of organization goals rather than the provision of a particular function.

Disadvantages :

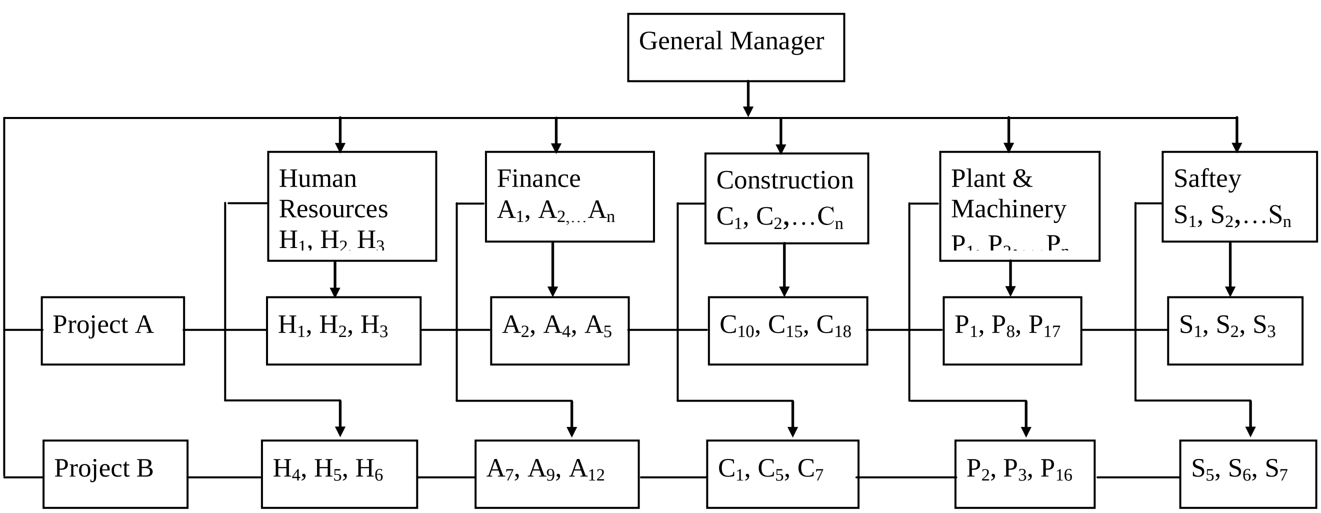
- There could be a duplication of efforts.
- It is very difficult to find a project manager having both general management expertise and diverse functional expertise.
- The administrative duties of a project manager may be demanding and the job could be quite stressful.
- Due to the fear of impediments in career growth, some employees may not prefer to leave their departments.
-

(iii) Matrix Organizations

The matrix organizations have evolved from the classical functional model. They combine the advantages of both the classical and the pure project/product structures. A matrix organization can take on a wide variety of specific forms, depending on which of the two extremes (functional or pure project) it most resembles.

In the matrix organization, the human resources are drawn from within various functional departments to form specific project teams. Every functional department has a pool of specialties. For example, the plant and machinery functional department may have the experts P_1, P_2, P_3, P_4, and so on. Similarly, the safety department may have experts S_1, S_2, S_3, S_4 and so on. When the project team for executing a project X is organized, the experts from different functional departments join together under the leadership of a project manager. For example, P_1 and S_1 may represent plant and machinery department and safety department, respectively. Similarly, depending on the requirement, personnel from different functional departments may join the project manager. The functional representative such as P_1 and S_1 are referred to as the project engineers. On completion of projects, the project engineers return to their functional units within the vertical organization structure.

The matrix organization recognizes the dynamic nature of a project and allows for the changing requirements. Such structures can cater to the varying workload and expertise demanded by the project. For example, if at any stage it is found that the services rendered by P_1 are inadequate and needs strengthening in the form of more persons, the project manager can request the functional head of plant and machinery department to send a few more persons such as P_2, P_3 and so on. Similarly, during less workload the project manager can request for demobilization of these project engineers. The project engineers P_1, P_2, P_3 and so on usually contact their parent functional departments for getting advice on complex technical matters or when they encounter unusual problems. Otherwise, for all practical purposes the project engineers are under the control of the project manager. One of the features of matrix organization is that the knowledge gained by the project engineers P_1, P_2 and others while executing projects can be shared vertically upwards for the benefit of future projects.



Typical matrix organization

Advantages:

- The structure facilitates quick response to changes, conflicts and project needs.
- There is a flexibility of establishing independent policies and procedures for each project, provided that they do not contradict company policies and procedures.
- There is a possibility of achieving better balance between time, cost and performance than is possible with the other structure such as functional or project forms.
- The project manager has authority to commit company resources provided the schedule does not cause conflicts with other projects.
- The strong base of technical expertise is maintained.

Disadvantages :

- Successful matrix authority application trends to take years to develop, especially if the company has never used dual authority relationships before.
- Initially, more effort and time is needed to define policies, procedures, responsibilities and authority relationships.
- The balance of power between functional and project authority must be carefully monitored.
- Functional managers may be biased according to their own set of priorities.
- Reaction times in a fast – changing project are not as fast as in the pure project authority structures.

The financial models are:

1. Payback Period
2. Discounted Cash Flow Models: The discounted cash-flow (DCF) technique takes into consideration the time value of money.

(i) Net present Value (NPV)

(ii) Internal Rate of Return (IRR)

1. Payback Period

The payback period is the time taken to gain a financial return equal to the original investment. The time period is usually expressed in years and months.

Consider the example where a company wishes to buy a new machine for a four year project. The manager has to choose between machine A or machine B, so it is a mutually exclusive situation.

Although both machines have the same initial cost (Rs.50,000) their cash-flows perform differently over the five year period. To calculate the payback period, simply work out how long it will take to recover the initial outlay (see table).

Year	Cash flow Machine A	Cash flow Machine B
0	Rs. 50000	Rs.50000
1	Rs 25000	Rs 15000
2	Rs. 15000	Rs 15000
3	Rs.10000	Rs.10000
4	Rs.10000	Rs.10000
5	Rs 10000	Rs.5000

Machine A will recover its outlay one year earlier than machine B. Hence, machine A is selected in preference to machine B.

(i) Net present value (NPV)

The NPV is a measure of the value or worth added to the company by carrying out the project. If the NPV is positive the project merits further consideration. When ranking projects, preference should be given to the project with the highest NPV.

Advantages:

- It introduces the time value of money.
- It expresses all future cash-flows in today's values, which enables direct comparisons.
- It allows for inflation and escalation. It looks at the whole project from start to finish.
- It can simulate project what-if analysis using different values.

Disadvantages:

- Its accuracy is limited by the accuracy of the predicted future cash-flows and interest rates.
- It is biased towards short run projects.
- It excludes non-financial data e.g. market potential.
- It uses a fixed interest rate over the duration of the project

(ii) Internal Rate of Return (IRR)

The internal rate of return is also called DCF yield or DCF return on investment. The IRR is the value of the discount factor when the NPV is zero. The IRR is calculated by either a trial and error method or plotting NPV against IRR. It is assumed that the costs are committed at the end of the year and these are the only costs during the year.

The IRR analysis is a measure of the return on investment, therefore, select the project with the highest IRR. This allows the manager to compare IRR with the current interest rates. One of the limitations with IRR is that it uses the same interest rate throughout the project, therefore as the project's duration extends this limitation will become more significant.

07(b).

Sol: Given SBC of soil = q_u = 300 kN/m^2

= 0.3 MPa

Dimensions of column A = 400 mm × 400 mm

B = 300 mm × 300 mm

c/c distance between columns = 3 m

Working load on column A = 2000 kN

B = 800 kN

Step – 1:

Base dimensions:

Assuming combined weight of footing + Back fill as 10% of column load

i.e., W' = 0.15 (2000 + 800) = 420 kN

$$\therefore \text{ Area of footing required} = \frac{2000+800+420}{300} = 10.733 \text{ m}^2$$

For soil, distribution to be uniform, the line of action of resultant load must pass through centroid of footing

Let centroid of footing is at a distance $\bar{x}$ from centre column 'B' projection of footing beyond column B = $\frac{300}{2} = 150$ mm

Factored column loads = $P_1 = 1.5 \times 2000 = 3000$ kN

$P_2 = 1.5 \times 800 = 1200$ kN

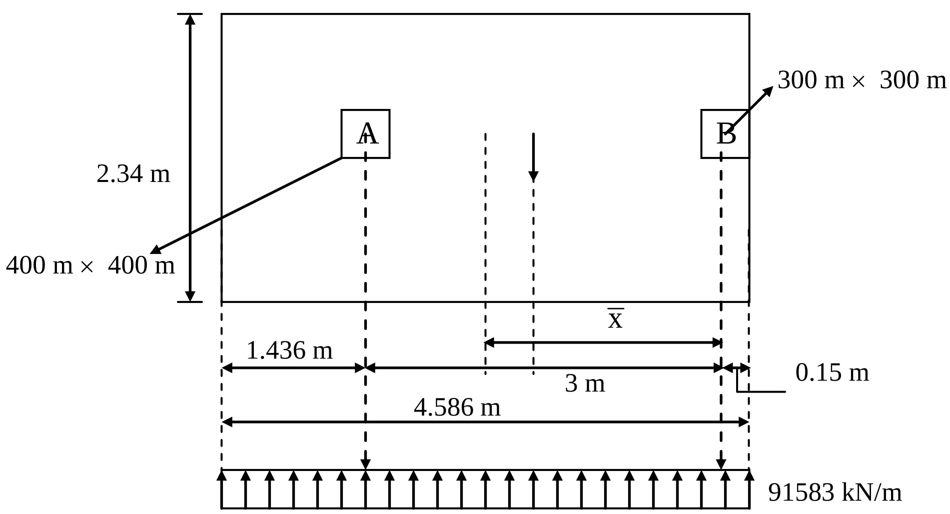
$$\therefore \bar{x} = \frac{3000 \times 3}{3000 + 1200} = 2.1428 \text{ m}$$

Also, if 'L' is length of footing

$$\frac{150}{1000} + \bar{x} = \frac{L}{2}$$

L = 2(0.15 + 2.1428)

= 4.586 m



Width of footing = $\frac{10.733}{4.586} = 2.34$ m

$\therefore$ Provide L = 4.586 m; B = 2.34 m

Projection beyond centre of column A = 4.586 – 3 – 0.15

= 1.436 m

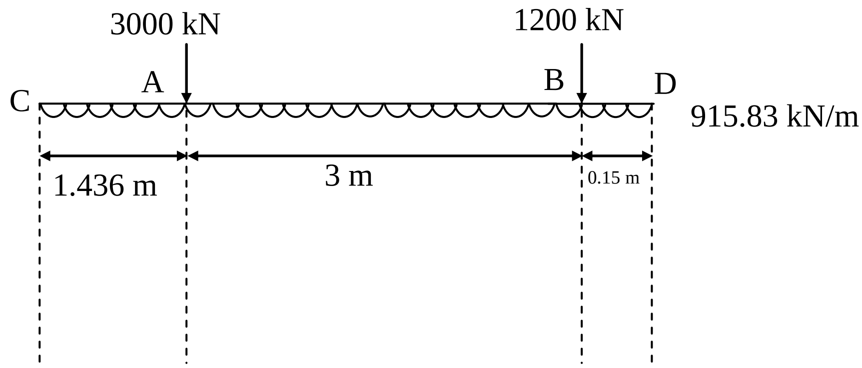
Step – 2:

Bending moment diagram:

Treating the footing as a wide beam (B = 2400 mm) in longitudinal direction, uniformly distributed load (acting upwords) is given by

$$\frac{P_1 + P_2}{L} = \frac{3000 + 1200}{4.586}$$

W = 915.83 kN/m



In AC: at a section x from 'C'

$$M_x = 915.8\frac{x^2}{2}$$

At C: x = 0; M_C = 0

A: x = 1.45 m; $M_A = 915.8 \times \frac{1.436^2}{2} = 944.26$ kNm

In AB: at a section 'x' from 'A'

$$M_x = 915.83\frac{(x + 1.436)^2}{2} - 3000x$$

At A; x = 0 ⇒ M_A = 944.26 kNm

At B ; x = 3m ⇒ M_B = 10.89 kNm

In BD : at a section 'x' from 'D'

$$M_x = 915.83\frac{x^2}{2}$$

At D; x = 0 ⇒ M_D = 0

At B; x = 0.15 m ⇒ M_B = 10.30 kNm

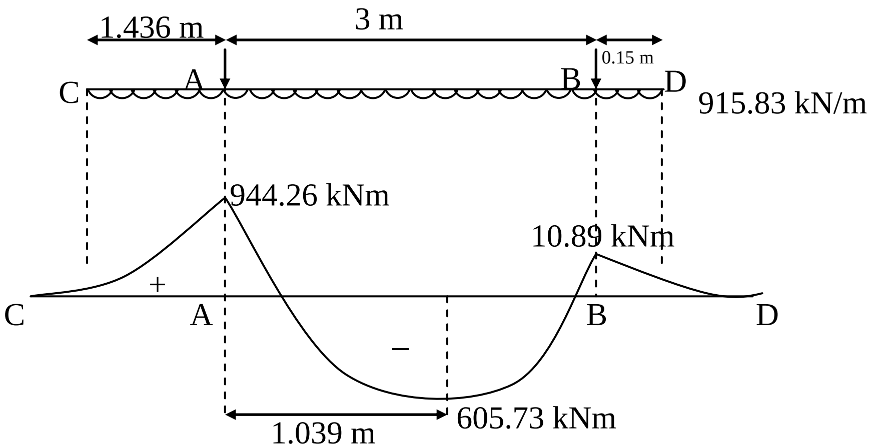
Mamximum negative BM in AB: $\frac{dM}{dx} = 0$

⇒ 915.83 (x+1.436) – 3000 = 0

$\Rightarrow$ x = 1.839 m from 'A'

$$\therefore \text{BM} = 915.83\frac{(1.839+1.436)^2}{2} - 3000(1.839) = -605.73 \text{ kNm}$$

BMD:

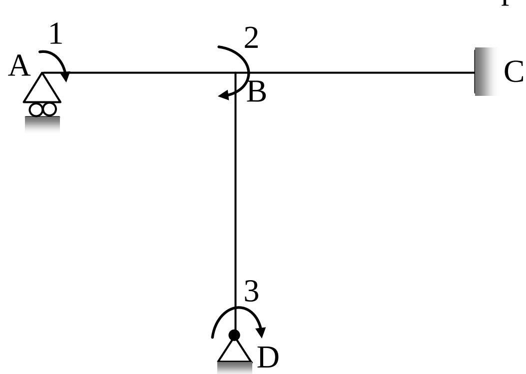


07(c).

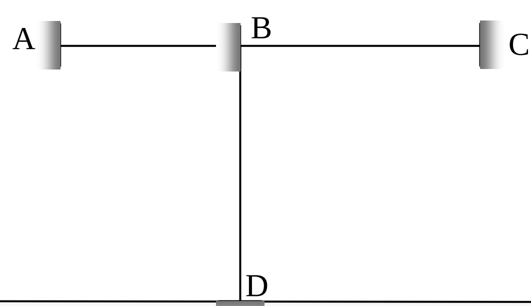
Sol: Determine the degree of kinematic indeterminacy (D_k)

$D_k = 3$ [θ_A , θ_B and θ_D] Neglecting axial deformations

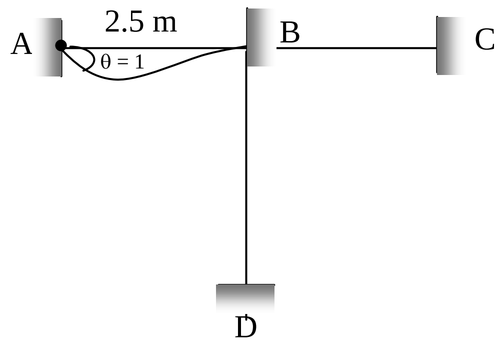
Assigning the coordinate numbers to the unknown displacements.

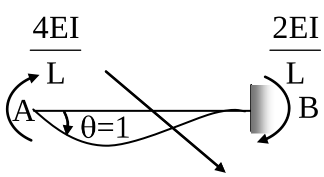


Impose restraints in all coordinates directions to get a fully restrained structure.



Determining the stiffness matrix (k) by giving unit displacement to the restrained structure in each of the coordinate directions and find the forces developed in all the coordinates directions. Applying unit displacement in coordinate direction 1.





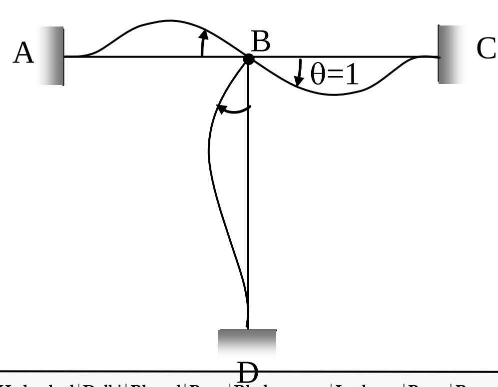
$$K_{11} = \frac{4EI}{L} = \frac{4EI}{2.5}$$

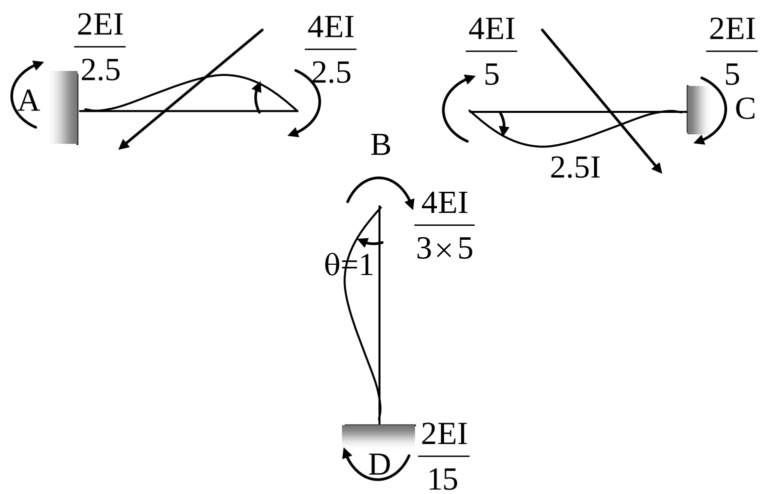
$K_{11} = 1.6$ EI

$$K_{21} = \frac{2EI}{L} = \frac{2EI}{2.5} = 0.8EI$$

$K_{31} = 0$

Applying unit displacement in coordinate direction '2'



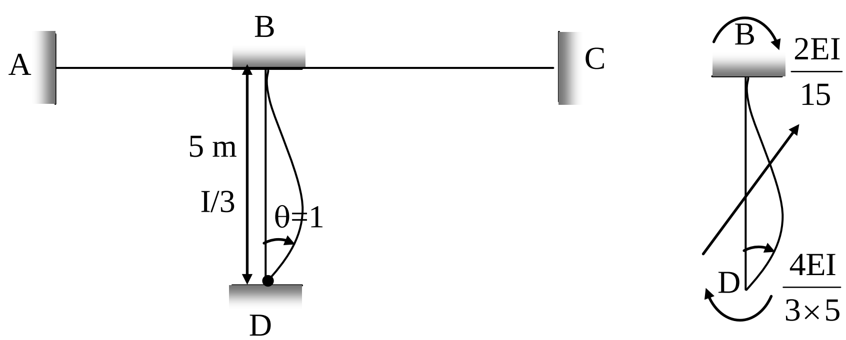


$$K_{12} = \frac{2EI}{2.5} = 0.8EI$$

$$K_{22} = \frac{4EI}{2.5} + \frac{4EI}{5} + \frac{4EI}{15} = 2.67\,EI$$

$$K_{32} = \frac{2EI}{15} = 0.13\,EI$$

Applying unit displacement in coordinate direction '3'



$K_{13} = 0$

$$K_{23} = \frac{2EI}{15} = 0.13EI$$

$$K_{33} = \frac{4EI}{15} = 0.267\,EI$$

$$\text{Stiffness matrix 'K'} = \begin{bmatrix} 1.6EI & 0.8EI & 0 \\ 0.8EI & 2.67EI & 0.13EI \\ 0 & 0.13EI & 0.267EI \end{bmatrix}$$

Find end moments:

$M_{FAB} = M_{FBA} = 0$

$$M_{FBC} = \frac{-WL^2}{12} = \frac{-20\times 5^2}{12} = -41.67 \text{ kN-m}$$

$M_{FBD} = M_{FDB} = 0$

Determining the forces developed in each of the coordinates directions of a fully restrained structure (P_L)

$$P_L = \begin{bmatrix} P_{L_1} \\ P_{L_2} \\ P_{L_3} \end{bmatrix}$$

$P_{L_1} = M_{FA} = M_{FAB} = 0$

$P_{L_2} = M_{FB} = M_{FBA} + M_{FBC} + M_{FBD}$

$= 0 - 41.67 + 0$

$= -41.67$

$P_{L_3} = M_{FD} = M_{FDB} = 0$

Observing the final forces in various coordinates direction (p)

$$P = \begin{bmatrix} P_1 \\ P_2 \\ P_3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

$\therefore$ As there are no external forces at coordinates 1, 2 & 3

Stiffness equation

$[K]\,[\Delta] = [P - P_L]$

$\Delta = [K]^{-1}\,[P - P_L]$

$$\begin{bmatrix} \Delta_1 \\ \Delta_2 \\ \Delta_3 \end{bmatrix} = \begin{bmatrix} 1.6EI & 0.8EI & 0 \\ 0.8EI & 2.67EI & 0.13EI \\ 0 & 0.13EI & 0.267EI \end{bmatrix}^{-1} \left[\begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} - \begin{Bmatrix} 0 \\ -4167 \\ 0 \end{Bmatrix} \right]$$

$$= \frac{1}{EI}\begin{bmatrix} 0.73 & -0.226 & 0.110 \\ -0.226 & 0.453 & -0.22 \\ 0.110 & -0.22 & 3.85 \end{bmatrix}\begin{bmatrix} 0 \\ 41.67 \\ 0 \end{bmatrix}$$

$$\Delta_1 = \frac{-9.41}{EI} = \theta_A$$

$$\Delta_2 = \frac{18.87}{EI} = \theta_B$$

$$\Delta_3 = \frac{-9.16}{EI} = \theta_D$$

The end moments in the members are obtained by using the slope deflection equations.

$$M_{AB} = \cancelto{0}{M_{FAB}} + \frac{2EI}{L}\left[2\theta_A + \theta_B\right]$$

$$= \frac{2EI}{2.5}\left[2 \times \frac{-9.41}{EI} + \frac{18.87}{EI}\right]$$

$$= 0$$

$$M_{BA} = \cancelto{0}{M_{FAB}} + \frac{2EI}{L}\left[2\theta_B + \theta_A\right]$$

$$= \frac{2EI}{2.5}\left[2 \times \frac{18.87}{EI} - \frac{9.41}{EI}\right]$$

$$= 22.66 \text{ kN-m}$$

$$M_{BC} = M_{FBC} + \frac{2EI}{L}\left[2\theta_B + \cancelto{0}{\theta_C}\right]$$

$$= (-)41.67 + \frac{2EI}{5}\left[\frac{18.87}{EI} \times 2\right] = -26.574 \text{ kN-m}$$

$$M_{CB} = M_{FCB} + \frac{2EI}{L}\left[\cancelto{0}{2\theta_C} + \theta_B\right]$$

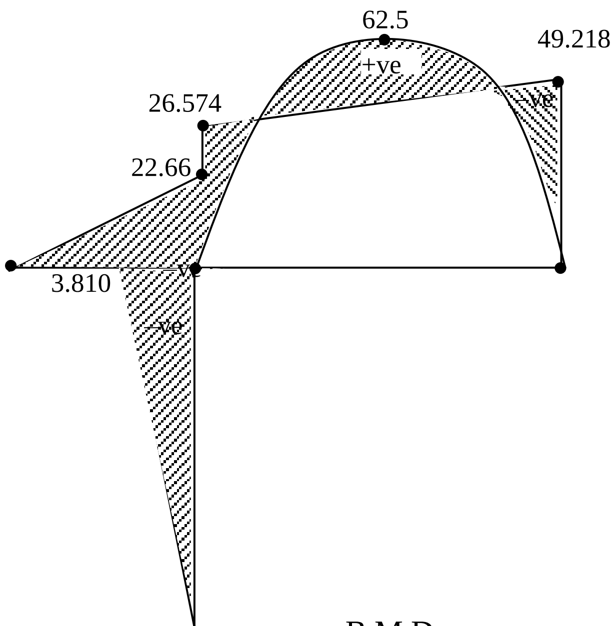
$$= 41.67 + \frac{2EI}{5}\left[\frac{18.87}{EI}\right]$$

$$= 49.218 \text{ kN-m}$$

$$M_{BD} = M_{FBD} + \frac{2EI}{L}[2\theta_B + \theta_D]$$

$$= \frac{2EI}{3\times 5}\left[2\times\frac{18.87}{EI} - \frac{9.16}{EI}\right] = 3.810 \text{ kN-m}$$

$M_{DB} = 0$



B.M.D

08(a)(i).

Sol:

I. Classification based on Geological Formation:

Building stones are obtained from rocks. Rocks are majorly classified on the basis of the mode of their occurrence, also referred as Geological classification

- **Igneous rocks:** Rocks that are formed by cooling of Magana or lava (molten or pasty rocky material) are known as igneous rocks. Ex: Granite, Basalt and Dolerite etc.
- **Sedimentary rocks:** These rocks are formed by the consolidation of the products of weathering obtained from the pre-existing rocks. Ex: gravel, sandstone, limestone, gypsum, lignite etc.
- **Metamorphic rocks:** These rocks are formed by the change in character of the pre-existing rocks when subjected to great heat and pressure. The process of their transformation is called metamorphism. Ex: Quartzite, Schist, Slate, Marble and Gneisses.

II. Physical Classification:

- **Stratified rocks:** These rocks possess planes of cleavage or stratification along which they can be split. Sedimentary rocks usually possess this property.
- **Un-stratified rocks:** The structure may be crystalline granular or compact granular. Examples: Igneous rocks.
- **Foliated rocks:** These rocks have a tendency to split up in a definite direction only. Ex: Metamorphic rocks.

III. Chemical Classification:

- **Siliceous rocks:** In these rocks, silica predominates. These rocks are hard; durable and not easily affected by weathering agencies. Ex: Granite, Quartzite, etc. Lime prevents shrinkage of raw bricks.
- **Argillaceous Rocks:** In these rocks, clay predominates. These rocks may be dense and compact or may be soft. Ex: Slates, Laterites etc.
- **Calcareous rocks:** Calcium carbonate is the main constituent in these rocks. The durability to these rocks will depend upon the constituents present in surrounding atmosphere. Ex: Lime Stone, marble, etc.

08(a)(ii).

Sol: Given data:

Number of working days/year = 300

No. of shifts/day = 2

No. of working hours/shift = 4hours

Production target = 30 million tonnes/year

$= 30 \times 10^6$ tonnes/year

Bulk density of the muck soils = 3.00 tonne/m^3

Step 1:

$\therefore$ Production target (in m^3/hr)

$$= \frac{30 \times 10^6 \text{ tonnes}}{3.00 \text{ tonnes}/\text{m}^3} \times \frac{1}{300 \times 2 \times 4(\text{Hours})}$$

$= 4166.667\ m^3/hr$

Step 2:

$\therefore$ O/P production capacity of shovel

$$= \frac{\text{Bucket capacity of shovel} (m^3)}{\text{Cycle time of shovel (seconds)}} \times 3600 \times k_b$$

Given bucket capacity of shovel = 15 m^3

Cycle time of shovel operations (like digging, lifting swinging and unloading etc) = 44 seconds

K_b = Bucket fill factor = 0.85

$\therefore$ Output production capacity of shovel $= \frac{15(m^3) \times 3600}{44(\text{seconds})}\left(\frac{m^3}{hr}\right) \times 0.85$

$= 1043.182\ m^3/hr$

Step 3:

Number of shovels required to target production

$$= \frac{\text{Total targeted production } (m^3/hr)}{\text{Production capacity of a shovel} (m^3/hr/shovel)}$$

$$= \frac{4166.667}{1043.182} = 3.994 \text{ Say 4 hovels}$$

08(b).

Sol: Let P be the factored load

Twisting moment M = P.e = 250P kN-mm

f_u = 410 MPa; f_y = 250 MPa

For grade M20 bolt of 4.6

f_{ub} = 400 MPa : f_{yb} = 240 MPa

Partial safety factors

$\gamma_{mb} = 1.25$, $\gamma_{mo} = 1.10$ & $\gamma_{ml} = 1.25$

Shank diameter of bolt d = 20 mm;

Diameter of bolt hole d_o = 22 mm;

Pitch p = 100 mm and

End distance e = 50 mm

The maximum forced bolt is one, which is farthest from C.G. of bolt group [i.e. r → maximum (1, 5, 6, & 10)] and which are close to the applied load line.

[θ → minimum (i.e. 1 and 5)]

Hence critical bolts are bolt no. 1 and bolt no. 5

Vertical shear force in any rivet due to P is F_a; $F_a = \frac{P}{n} = \frac{P}{10} = 0.1P\ kN$

Shear force in critical bolt due to M is F_m; $F_{m1} = F_{m5} = \frac{Mr_1}{\Sigma r^2}$

$r_1 = r_5 = r_6 = r_{10} = \sqrt{\left(\frac{120}{2}\right)^2 + 200^2}$

$= 208.80$ mm

$r_2 = r_4 = r_7 = r_9 = \sqrt{\left(\frac{120}{2}\right)^2 + 100^2} = 116.62$ mm

$r_3 = r_8 = \frac{120}{2} = 60\ mm$

$\Sigma r^2 = (r_1^2 + r_5^2 + r_6^2 + r_{10}^2) + (r_2^2 + r_4^2 + r_7^2 + r_9^2) + (r_3^2 + r_8^2)$

$= 4(208.8)^2 + 4(116.62)^2 + 2(60)^2$

$= 236000\ mm^2$

$$F_m = F_{m1} = F_{m5} = \frac{Mr_1}{\Sigma r^2} = \frac{250P \times (208.8)}{236000} = 0.2211\ P$$

$\cos\theta_1 = \frac{60}{208.8} = 0.287$

Maximum resultant shear force in critical bolt $F_{Rmax} = F_{R1} = F_{R5}$

$$F_{R\,max} = \sqrt{F_a^2 + F_{m1}^2 + 2F_aF_{m1}\cos\theta_1}$$

$$= \sqrt{(0.1P)^2 + (0.2211P)^2 + 2(0.1P)\times(0.2211P)\times 0.287}$$

$F_{Rmax} = 0.267P$

For safety of bolt group as per LSD of IS800:2007

$F_{Rmax} \le$ Design strength of one bolt (V_{db})

Design strength of one bolt V_{db}

V_{db} = Minimum of V_{dsb} and V_{dpb}

Design shear strength of one bolt (V_{dsb})

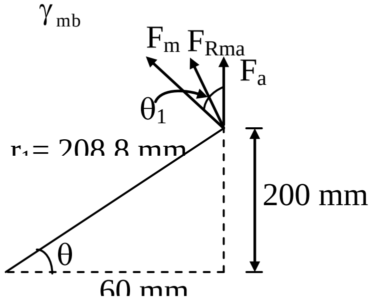
$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3}\times 1.25}\left(1\times 1\times 0.78\times \frac{\pi}{4}(20)^2 + 0\right)$$

$$= 45.26 \times 10^3 \text{ N} = 45.26 \text{ kN}$$

Design bearing strength of one bolt (V_{dpb})

$$V_{dpb} = \frac{2.5 k_b\, dt \cdot f_{ub}}{\gamma_{mb}}$$



k_b is a bearing factor is lesser of

- $\frac{e}{3d_o} = \frac{50}{3\times 22} = 0.75$
- $\frac{p}{3d_o} - 0.25 = \frac{100}{3\times 22} - 0.25 = 1.26$
- $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$
- 1.0

Hence bearing factor k_b = 0.75

$$V_{dpb} = 2.5\times k_b \times d\times t\cdot\times\frac{f_{ub}}{\gamma_{mb}}$$

$$= 2.5\times 0.75\times 20\times 7.8\times\frac{400}{1.25}$$

$$= 93.6 \times 10^3 \text{ N} = 93.6 \text{ kN}$$

Design strength of one bolt V_{db} = 45.26 kN

Equating 0.267 P = 45.26 $\Rightarrow$ $P = \frac{45.26}{0.267}$ = 169.51kN

Factored load P = 169.51 kN

08(c).

Sol: Given:

P = 2000 kN

D = 500 mm

f_{ck} = 25 MPa, f_y = 415 MPa

$\therefore$ Design load = P_u = 1.5 P = 1.5 × 2000 = 3000 kN

The column is effectively held in portion at both ends and restrained against rotation i.e., fixed at both ends.

As per, IS 456 – Cl 25.2, $l_{eff} = 0.65\, l$, for fixed column

$$\lambda = \frac{\ell_{eff}}{D} = \frac{2.990}{400} = 0.65 \times 4.6 = 2.99 \text{ m} < 12 \Rightarrow \text{short column}$$

Step – 1:

Minimum Eccentricity:

$$e_{mx} = \max\left\{\frac{\ell_{eff}}{500} + \frac{D}{30},\ 20 \text{ mm}\right\}$$

$$= \max\left\{\frac{2.990}{500} + \frac{500}{30}, 20 \text{ mm}\right\}$$

= max {22.64, 20}

= 22.64

0.05 D = 0.05 × 500 = 25 mm

$\therefore e_{min} \ngtr 0.05$ D

Step – 2: $\therefore e_{min} \ngtr 0.05$ D;

For circular column with helical reinforcement

$$\rho_u = 1.05[0.4 f_{ck} A_C + 0.67 f_y A_{SC}]$$

$$\Rightarrow 3000 \times 10^3 = 1.05\left[0.4 \times 25 \times \left(\frac{\pi}{4} \times 500^2 - A_{SC}\right) + 0.67 \times 415 \times A_{SC}\right]$$

$\Rightarrow 2857.14 \times 10^3 = 1963495.41 + A_{SC}(268.05)$

$\Rightarrow A_{SC} = 3333.88 \text{ mm}^2$

Using 20 mm diameter bars, number of bars required

$$= \frac{3333.88}{\frac{\pi}{4}\times 20^2} = 10.61 \approx 11 \text{ No's}$$

Step 3:

Design of Helical reinforcement:

(a) Diameter of ties required is maximum of

(i) $\nless \frac{1}{4}(\phi_{max}) = \text{i.e.,} \nless \frac{1}{4}(20)$

$\Rightarrow \ngtr 5$ mm

(ii) $\ngtr 6$ mm

$\therefore$ Provide 8 mm ϕ.

(b) For helical reinforcement

$$0.36\frac{f_{ck}}{f_y}\left(\frac{A_g}{A_c}-1\right) \le \frac{V_h}{V_c}$$

$$A_g = \frac{\pi}{4}\times 500^2 = 196349.54 \text{ mm}^2$$

Core diameter $D_c = D_g - 2$ (clear cover)

$= 500 - 2\ (40)$

$= 420$ mm

$$A_C = \frac{\pi}{4}\times D_C^2 = \frac{\pi}{4}\times 420^2 = 138544.238 \text{ mm}^2$$

$$V_h = \frac{1000}{P}\times(\pi\times D_h)\times\left(\frac{\pi}{4}\times\phi_h^2\right)$$

$$D_h = D_c - \phi_h = 420 - 8 = 412 \text{ mm}^2$$

$$\Rightarrow V_h = \frac{1000}{P}\times\pi\times 412\times\frac{\pi}{4}\times 8^2$$

$$= \frac{65534.17\times 10^3}{P}$$

$V_C = 1000 \times A_C = 1000 \times 196349.54 \text{ mm}^2$

$$\therefore \frac{0.36\times 25}{415}\left(\frac{196349.54}{138544.238}-1\right)\le \frac{65534.17\times 10^3}{P\times 196349.54\times 10^3}$$

$$\Rightarrow 9.048\times 10^{-3}\le \frac{0.333}{P}$$

$\Rightarrow P \le 36.88$ mm

Also: Pitch $\ngtr$ 75 mm

$$\ngtr \frac{1}{6}D_c\left(\frac{1}{6}\times 420\right)=70\text{ mm}$$

i.e., $\ngtr$ 70 mm

$\nless$ 25 mm

$\nless 3\,\phi_h$ ($3\times 8 = 24$ mm)

$\therefore$ Provide pitch @ 30 mm.

Detailing: