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ESE - 2018
MAINS EXAMINATION
Questions with Detailed Solutions

CIVIL ENGINEERING

PAPER - I

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CIVIL ENGINEERING

ESE _MAINS_2018_PAPER – 1

PAPER REVIEW

Except few questions from RCC & Steel, remaining questions in the paper can be easily attempted. Particularly in this paper selection of questions plays a vital role in securing a good score. For example Section-A is relatively easy than Section-B, so choosing 3 questions from Section-A will fetch you a big advantage.

SUBJECT WISE REVIEW

SUBJECT(S) SECTION-A	LEVEL	Marks
Strength of Materials	Easy	104
Structural Analysis	Moderate	84
Building Materials & Concrete Technology	Easy	52
SECTION-B	**LEVEL**	**Marks**
R.C.C. & P.S.C	Moderate	104
Steel Structures	Moderate	84
Construction Management & Equipment	Easy	52

Subject Experts,

ACE Engineering Academy

01. (a)

(i) List out at least eight tests required to determine the suitability of stone for engineering use. (8 M)

Sol:

Following are the tests conducted to determine the suitability of stone for engineering use

i) **Acid Test:** This test is carried out to understand the presence of $CaCO_3$ in building stone.

ii) **Attrition Test:** This test is done to find out the resisting power of stones against grinding action or wear and tear against the traffic.

iii) **Crushing Test:** This test is performed to find out the compressive strength of stones.

iv) **Impact Test:** This test is performed to find out resistance of stones against sudden loading.

v) **Smith's Test:** This test is performed to find out presence of soluble matter in stones.

vi) **Water Absorption Test:** This test is done to check how much water the stone may absorb.

vii) **Hardness Test:** This test is done to determine the surface abrasion resistance of stone.

viii) **Microscopic Test:** This test is performed to check grain size, texture, pores, shakes etc of stone.

(ii) Briefly explain the purpose and the procedure for Attrition Test on stone. (4 M)

Sol:

The main purpose of Attrition test is to check the resistance of stone against abrasion (ability to withstand grinding action) of stone.

The procedure of the test is discussed as follows:

- This test is carried out on 'Deval Attrition test machine'
- The test specimen of stone is weighed (say W_1 kg) before transferring it to the machine.
- The drum of machine is inclined at 30^o to horizontal and it is allowed to rotate @ 2000 revolutions per hour for 5 hours.
- After 5 hours, stone are sieved on a 1.70 mm IS sieve (As per IS:2386 Part IV) and the retained stone are weighed (say W_2 kg).
- Loss in weight as percentage indicates the percentage of wear.
- % loss in weight = $\frac{W_1 - W_2}{W_1} \times 100\%$
- A maximum of 40% is allowed for Water Bound Macadam in Indian conditions and 35% for bituminous concrete.

01. (b) A very long steel drill pipe got stuck in hard clay at an unknown depth. The drill pipe was applied a large upward force and observed that the drill pipe came out elastically by 500 mm. It was also observed that there was elongation of 0.04 mm in a gauge length of 200 mm. Estimate the depth of hard clay bed. Following consideration may be taken into account: Resistance offered by all material / elements may be taken as zero. (12 M)

Sol: Given that resistance offered by all material elements is taken as zero only the bar is deforming elastically.

l = Depth of pipe struck in clay

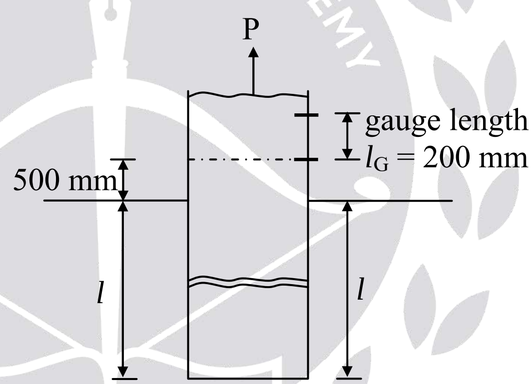
The elastic deformation of pipe, $\delta l_p = 500$ mm

[Which is due to elastic deformation over a length struck up in hard clay]

Elastic deformation observed in a gauge length of 0.04 mm is 200 mm

$$\therefore \delta\ell_G = \frac{P\ell}{AE} = \frac{P}{AE}(200) = 0.04$$

$$\frac{P}{AE} = \frac{0.04}{200}$$



For the entire pipe

$$\delta\ell_{pipe} = \frac{P\ell}{AE}$$

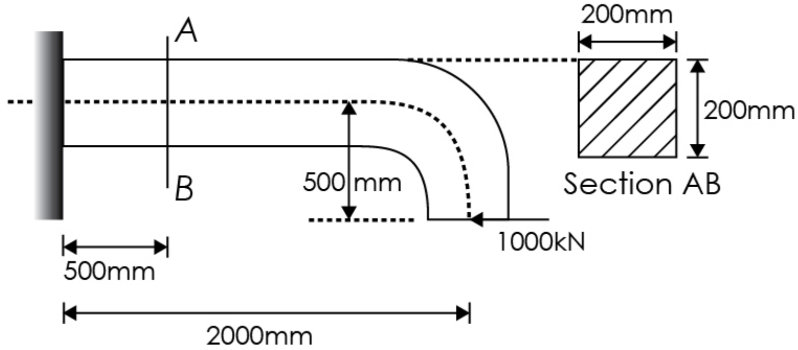
$$500\text{ mm} = \frac{P}{AE}(\ell)$$

$$500 = \frac{0.04}{200}(\ell)$$

Length of pipe got struck in hard clay = $l = \dfrac{500 \times 200}{0.04}$

$$l = 2500 \text{ m}$$

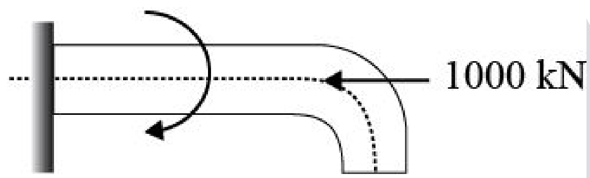
01. (c)



A bracket has been loaded as shown in figure. Compute the top and bottom fibre stress at section A-B. (12 M)

Sol:

BM = 1000 ×500 kN-m



Axial Force = –1000 kN = 10^6 N (compressive)

Bending moment = 1000 × 500 kN-mm = 500 × 10^6 N-mm

From the cross-section,

(i) Stress at top section

$$\sigma_{top} = \sigma_{bending} - \sigma_{axial}$$

$$= +\frac{500\times10^6\times100}{\frac{200^4}{12}} - \frac{10^6}{200^2}$$

$$\sigma_{top} = 375 - 25$$

$$= +350 \text{ MPa (Tensile)}$$

(ii) Stress at bottom layer:

$$\sigma_{bottom} = -\sigma_{bending} - \sigma_{axial}$$

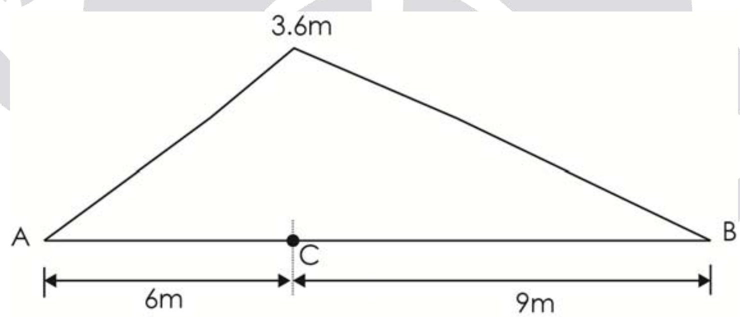
$$= -\frac{500\times10^6\times100}{\frac{200^4}{12}} - \frac{10^6}{200^2}$$

$$\sigma_{bottom} = -375 - 25 = -400 \text{ MPa}$$

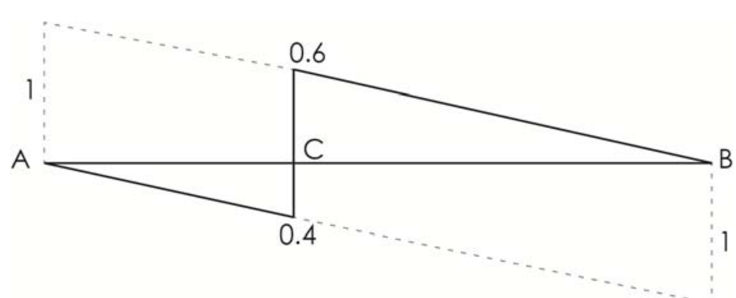
01. (d) A beam *AB* is simply supported over a span of 15 m. An u.d.l of 25 kN/m intensity and 5 m length moves over the beam from end *A* to *B*. Draw the influence line diagram for bending moment and shear force at section C located at 6 m from end *A*. Hence calculate the maximum bending moment and shear force at section C. (12 M)

Sol:

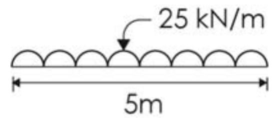
ILD for Bending Moment at C:

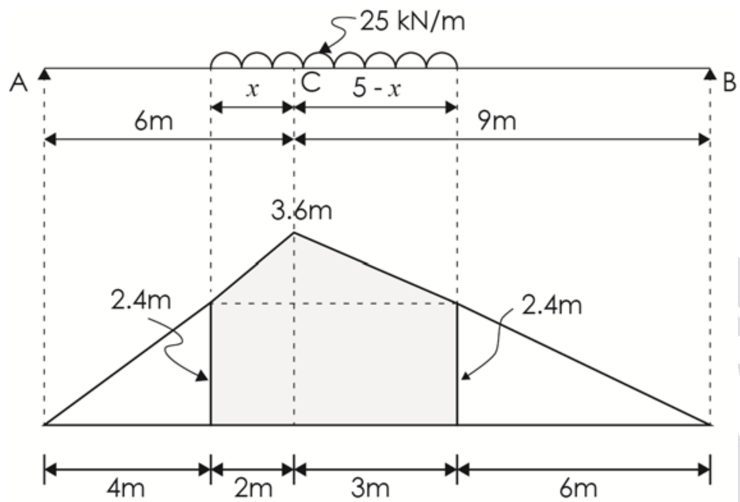


ILD for shear force at C:



Max bending moment at section 'C':





ILD for BM $_C$

Average load on AC = Average load on BC

$$\frac{x}{6} = \frac{5-x}{9}$$

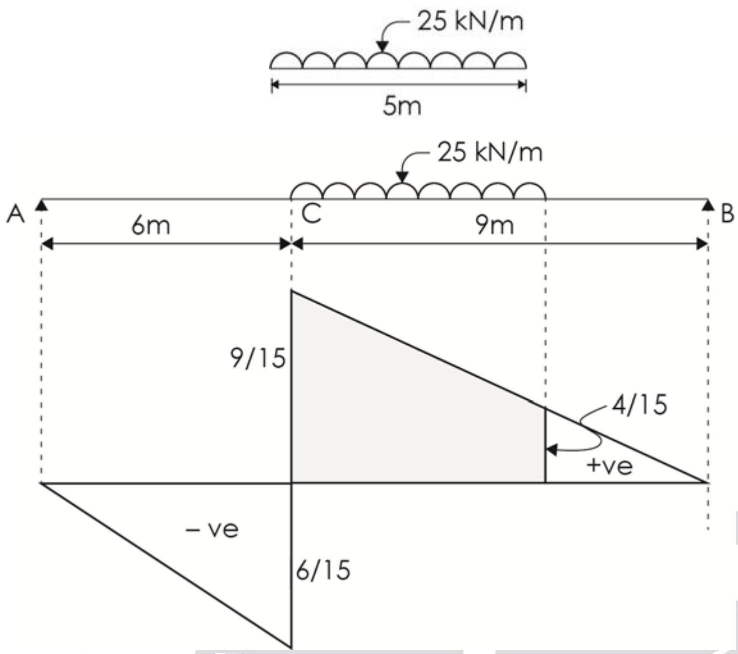
x = 2 m

Max BM at C = Intensity of ud*l* × Area of shaded ILD

$$= \left[25\left(\frac{2.4+3.6}{2}\right)2 + 25\left(\frac{2.4+3.6}{2}\right)3\right]$$

= 150 + 225 = 375 kN-m

Maximum shear force at section 'C':



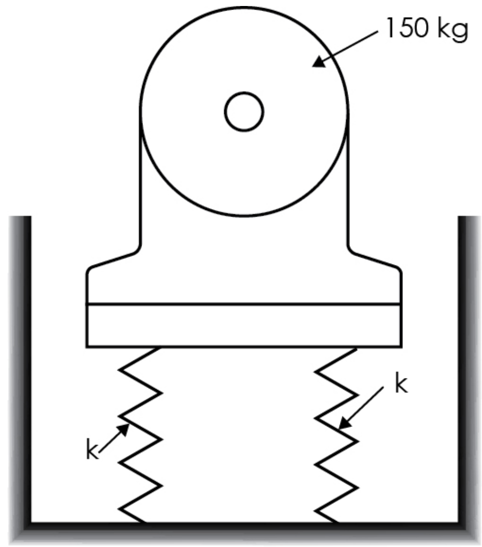
ILD for SF_C

Maximum SF at C = Intensity of ud*l* × Area of shaded ILD

$$= 25\left(\frac{\frac{9}{15}+\frac{4}{15}}{2}\right)5$$

= 54.167 kN

01. (e)



A motor of 150 kg mass is supported by four springs as shown in figure. Each of the springs has a stiffness of 120 kN/m. The unbalance of the rotor is equivalent to a mass of 40 g located at 150 mm from the axis of rotation. The motor is constrained to move vertically. Find

(i) the speed of motor at which resonance will occur

(ii) the amplitude of vibration of the motor when the speed is 1000 rev./m. (12 M)

Sol: $K = 4\times 120\times10^3 = 480\times10^3$ N/m

M = 150 kg

ω_n = undamped natural frequency $= \sqrt{\frac{K}{M}} = \sqrt{\frac{48\times10^4}{150}}$

= 56.569 rad/s

ω = 1000 rpm $= \frac{2\pi\times1000}{60}$

= 104.72 rad/s

m = unbalanced mass = 40 g = 0.04 kg

e = eccentricity of unbalanced mass = 150 mm = 0.15 m

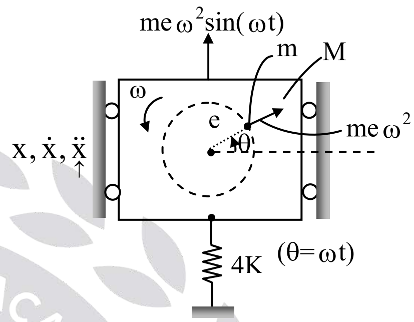
(i) Resonance occurs when speed of rotation coincides/equals natural frequency of the system.

$\therefore$ Rotation speed at which resonance occurs = ω_n = 56.569 rad/s

$$= \frac{56.569 \times 60}{2\pi}\,\text{rpm} = 540.2\,\text{rpm}$$

(ii) Equation of motion of the system is

$$M\ddot{x} + Kx = me\omega^2 \sin(\omega t)$$



Steady state solution is $x(t) = \pm\frac{me\omega^2}{K - M\omega^2}\sin(\omega t)$

Steady state amplitude = X = K – M ω²

$$X = \left|\frac{me\omega^2}{K - M\omega^2}\right| = \left|\frac{0.04 \times 0.15 \times 104.72^2}{480000 - 150 \times 104.72^2}\right|$$

= 0.0565 mm

02.(a)

(i) List out eight chemical ingredients of Portland cement and briefly explain their functions.

(12 M)

Sol:

The chemical ingredients of Portland cement are discussed as follows:

(i) **Lime (CaO) (62-67%):** It impart strength and soundness to cement. If it is in excess, it makes the cement unsound which will cause it to expand & disintegrate.
If it is deficient, the strength of cement reduces.

(ii) **Silica (17-25%):** It also imparts strength to cement.

- If inexcess, the strength of cement is increased, but also increases the setting time of cement.

(iii) **Alumina (3-8%):**

- It imparts quick setting property to the cement.
- It also acts as a flux in reducing the clinkering temperature.
- If inexcess, it reduces the strength of cement.

(iv) **Ca SO_4 (Gypsum) (3-4%):**

- $CaSO_4$ is added in the form of gypsum.
- It helps in increasing the initial setting time of cement.

(v) **Iron Oxide (Fe_2O_3) [3-4%]**

- It imparts strength, hardness and reddish brown tint to cement.
- If inexcess, cement appears too dark.

(vi) **Magnesia (1-3%]**

- It also imparts strength, hardness and yellowish tint to cement.
- If inexcess, it makes cement unsound and also cement appear too light.

(vii) **Sulphur (1-3%)**

- It is present as an impurity in cement.
- It is responsible for making the cement unsound and volume changes.
- Unsoundness due to sulphur cannot be measured experimentally.

(viii) **Alkalies (0.2-1%)**

- These are also present in cement as an impurity.
- These are responsible for whitish stains appeared on cement, called on efflorescence defect.

02. (a)

(ii) What are Bogue's compounds? Briefly mention their functions. (8 M)

Sol: During the burning process (clinkering) in the manufacture of cement, the chemical ingredients fuse together to form complex chemical compounds known as Bogue’s Compounds.

There are 4 Bogue’s compounds, which may have different formation time. These are briefly discussed as follows:-

Tri Calcium Aluminate (3 CaO. Al_2 O$_3$] or [C_3A]

- It is also called on celite (around 4-14%)
- It undergoes hydration within 24 hrs after addition of water.
- It is responsible for flash setting (quick setting) of Cement; and shrinkage which may lead to cracks.
- It releases maximum heat of hydration.

Tetra Calcuim Alumino Ferrate (4 CaOAl_2O_3. Fe_2O_3)or (C_4AF)

- It is also called felite (10-18%)
- It also undergoes hydration within 24 hrs after addition of water.
- It is observed to worst engineering property, and does not impart any desirable property to cement.

Tri-Calcium Silicate (3Cao.SiO_2) or C_3S:

- It is also called Alite (45-65%)
- It undergoes hydration within a week after addition of water to cement.
- It is responsible for development of early strength in cement. Hence for any construction where early gain of strength is required, C_3S proportion may be increased.

  Ex: Cold weather concreting, prefabricated construction, emergency repair work etc
- It posseses the best cementing property among all Bogue's compounds.

Dicalcium Silicate (2CaO. SiO_2) or (C_2S):

- It is also called Belite (15-35%)
- It undergoes complete hydration within a year after addition of water.
- It is responsible for gain in strength in later stages, resistance of cement against chemicals.
- Its proportion may be increased in constructions, where strength is required in later stages.

  Ex: Dam construction

  Bridge construction

  Long term hydraulic projects

02.(b) A square bar (50 mm × 50 mm cross-section) of 100 mm length is subjected to an axial compressive load of 10 kN. Calculate the change in volume of the bar, if all lateral strains of the bar are prevented by a uniform pressure on its four lateral faces. Calculate this pressure value and the change in volume. Also calculate the value of bulk modulus *K* and shear modulus *G* for the material of bar. Following parameters may be used if required.

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

2. $\mu = 0.25$

(20 M)

Sol: Given Data:

$E = 2 \times 10^5$ MPa, $\mu = 0.25$

$P_x = -10^4$N, L = 100 mm

C/s of square 50 × 50 mm

(i) Bulk Modulus,

$$k = \frac{E}{3\times(1-2\mu)}$$

$$= \frac{2\times10^5}{3\times(1-2(0.25))}$$

$$= 1.33 \times 10^5 \text{ N/mm}^2$$

(ii) Shear Modulus,

$$G = \frac{E}{2\times(1+\mu)} = \frac{2\times10^5}{2\times(1+0.25)} = 80 \times 10^3 \text{ N/mm}^2$$

(iii) Assume, lateral strains are restricted by applying σ_y and σ_z

$$\sigma_x = \frac{-10^4}{50^2} = -4 \text{ MPa}$$

$$\varepsilon_y = 0$$

$$\therefore \quad \varepsilon_y = \frac{\sigma_y - \mu(\sigma_x + \sigma_z)}{E} = 0$$

$$\sigma_y - \mu.(\sigma_x + \sigma_z) = 0$$

Similarly, as, $\varepsilon_z = 0 \quad \Rightarrow \sigma_Z - \mu(\sigma_x + \sigma_y) = 0$

On solving $\sigma_y = \sigma_z = \dfrac{\mu\sigma_x}{1-\mu}$

$$\sigma_y = \sigma_z = \frac{0.25\times(-4)}{1-0.25}$$

$$= -1.33 \text{ N/mm}^2$$

$\therefore$ Uniform pressure on lateral faces is 1.33 N/mm^2, compressive

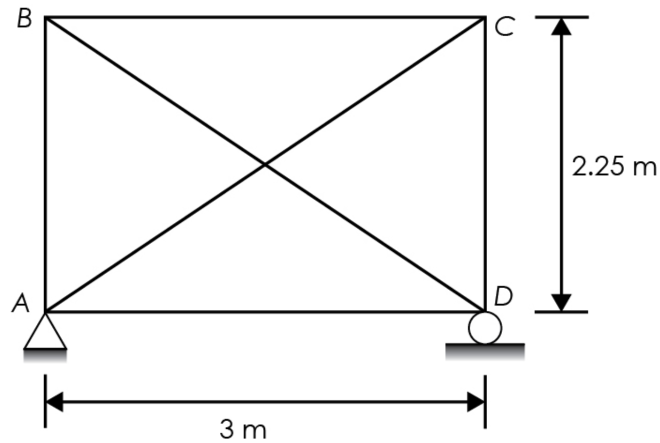
(iv) $\varepsilon_v = \dfrac{(\sigma_x + \sigma_y + \sigma_z)(1-2\mu)}{E}$

$$= \frac{(-4-1.33-1.33)(1-2\times 0.25)}{2\times 10^5}$$

$$\varepsilon_v = -1.665\times 10^{-5} = \frac{\Delta v}{v}$$

$\therefore$ $\Delta V = -1.665 \times 10^{-5} \times 100 \times 50^2 = -4.1625 \text{ mm}^3$

02. (c)



The frame shown in figure is fabricated using members with cross-sectional areas as given below :

Diagonal members = 2000 sq. mm

Other members = 1000 sq. mm

***E* for the material of members = 2×10^5 N/sq. mm**

Member *AC* was fitted last and the length of the member was 1 mm short. Determine the forces developed in the members when *AC* was pulled and fitted in position. (20 M)

Sol: Static indeterminacy $D_s = D_{se} + D_{si}$

External indeterminacy D_{se} = Number of reaction – Number of equilibrium equations

$D_{se} = 3 - 3 = 0$ (externally determinate structure)

Internal indeterminacy $D_{si} = m - (2j - 3)$

Where, m = No.of members = 6

j = No.of joints = 4

$D_{si} = 6 - (2 \times 4 - 3)$

= 1 (Internally indeterminate structure)

$D_s = D_{se} + D_{si} = 1$ (statically indeterminate structure)

Note:

1. In case of statically determinate frames if any member is not of exact length and it is forced in position, there are no stresses induced in the members of the frame.

2. In case of indeterminate frames, if the members are not of exact length, they will have to be fixed in position which will induce forces in the other members of the frame.

   Force in the member having lack of fit is 'X'.

$$X = \frac{\delta}{\sum \frac{k^2 L}{AE}}$$

Sign Conventions:

'δ' is taken to be positive if the members is short in length (so as to exert pull 'X' at the joints) and negative if the member is excess in length (so as to apply push at the joints).

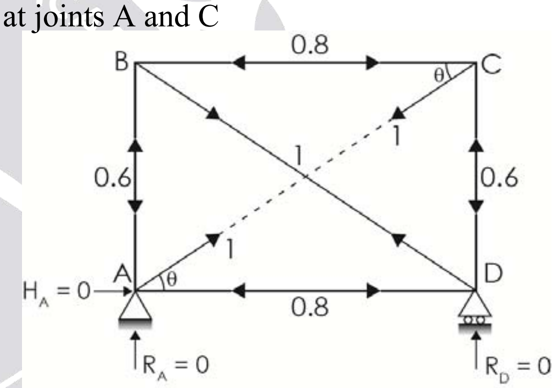
(i) To analyse the frame, it is made determinate by removing the member having lack of fit, unit forces are applied at the joints of the members having lack of fit.

(ii) Member 'AC' is removed and unit loads are applied at joints A and C

$$\cos\theta = \frac{3}{3.75} = 0.8$$

$$\sin\theta = \frac{2.25}{3.75} = 0.6$$

Compression: –ve Tension: +ve



Member	K	L (m)	A (mm²)	$\frac{K^2L}{A}$	Force in members due to lack of fit = K X (kN)
AD	–0.8	3000	1000	1.92	–17.372 (compression)
BC	–0.8	3000	1000	1.92	–17.372 (compression)
AB	–0.6	2250	1000	0.81	–13.029 (compression)
CD	–0.6	2250	1000	0.81	–13.029 (compression)
BD	1	3750	2000	1.875	21.715 (tension)
AC	1	3750	2000	1.875	21.715 (tension)
				9.21	

$$X = \frac{\delta}{\sum \frac{K^2 L}{AE}} = \frac{1}{\frac{9.21}{2\times10^5}}$$

$X = 21.715 \times 10^3$ N = 21.715 kN

03. (a)

(i) Briefly explain Thermoplastic and Thermosetting materials (4 M)

Ans: 1. Behaviour with respect to heating:

(i) **Thermo-plastic:** The thermo-plastic or heat non-convertible group is the general term applied to the plastics which become soft when heated and hard when cooled. It is possible to shape and reshape these plastic by means of heat and pressure. One important advantage of this variety of plastics is that the scrap obtained from old and warn-out articles can be effectively used again.

(ii) **Thermo-setting:** The thermo-setting or heat convertible group is the general term applied to the plastics which become rigid when moulded at suitable pressure and temperature. The thermo-setting plastics are soluble in alcohol and certain organic solvents, when they are in thermo-plastic stage. This property is utilized for making paints and varnishes from these plastics. The thermo-setting plastics are durable, strong and hard. They are available in a variety of beautiful colours. They are mainly used in engineering application of plastics.

03. (a)

(ii) List out six differences between them. (8 M)

Sol: The following are the differences between Thermoplastics and Thermosetting materials.

S. No.	Thermoplastics	Thermosetting Materials
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

03. (a)

(iii) Briefly explain manufacture of Aluminium and state at least six physical and mechanical properties of Aluminium. (8 M)

Sol: Industrial manufacture of Aluminium is accomplished in two phases.

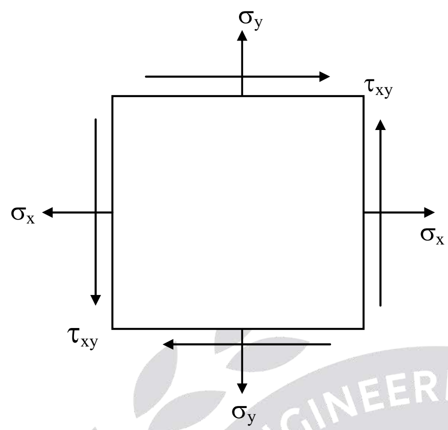
1. **Bayer process:** In this process the bauxite (ore for Aluminium) is refined to get alumina (Aluminium oxide).
2. **Hall-Heroult process:** In this process, smelting of aluminium oxide is done to produce pure aluminium.

The following are the different physical and mechanical properties of Aluminium.

1. Low specific gravity (Around 2.7).
2. High Corrosion Resistance.
3. High Electrical Conductivity.
4. High Thermal Conductivity.
5. Low melting point.
6. High Ductility.
7. High Recyclability.

03. (b) In a steel flat plate a state of plane stress available. Calculate σ_y and principal stresses if $\sigma_x = 140$ N/mm², $\mu = 0.25$, $\tau_{xy} = 40$ N/mm², $\epsilon_z = -3.6 \times 10^{-4}$ and $E = 2 \times 10^5$ N/mm². (20 M)

Sol:



Strain in z- direction (unaffected by τ_{xy}) because τ_{xy} only change shape, does not affect volume.

Data: $\sigma_x = 140$ N/mm²

$\mu = 0.25$

$\tau_{xy} = 40$ N/mm²

$\varepsilon_z = -3.6 \times 10^{-4}$

$E = 2 \times 10^5$ N/mm²

(i) For plane stress system

Plane stress condition

$\sigma_z = 0$ [stress perpendicular to the plane of area]

$$\varepsilon_z = \frac{\sigma_z}{E} - \mu\frac{\sigma_x}{E} - \mu\frac{\sigma_y}{E}$$

$$-3.6\times10^{-4} = \frac{0}{E} - 0.25\frac{140}{2\times10^5} - 0.25\frac{\sigma_y}{2\times10^5}$$

$$-3.6\times10^{-4} = \frac{0-0.25(140+\sigma_y)}{2\times10^5}$$

$\sigma_y = 148$ N/mm²

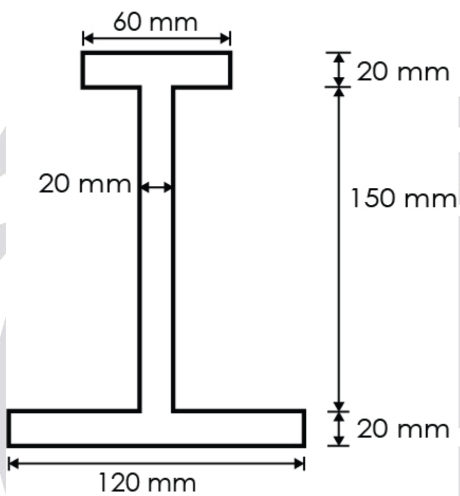
(ii) Principal stresses:

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

$$\sigma_{1,2} = \frac{140+148}{2} \pm \sqrt{\left(\frac{140-148}{2}\right)^2 + 40^2}$$

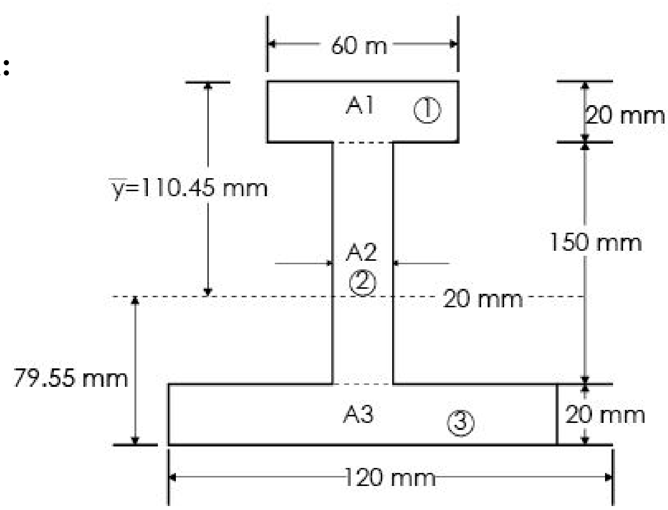
$\therefore\ \sigma_1 = 184.2$ MPa and $\sigma_2 = 103.8$ MPa

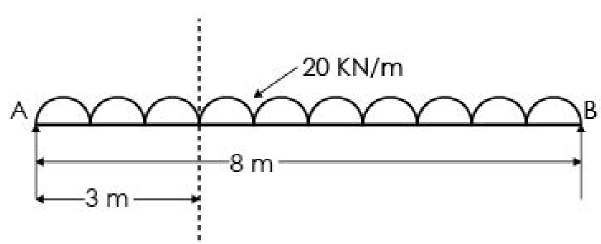
03. (c)



A beam of unsymmetrical I section shown in figure is simply supported over a span of 8 m. It carries a uniformly distributed load of 20 kN/m over the entire span. Draw the sketch for shear stress variation across the depth of the cross-section located at a distance of 3 m from left end *A*. (20 M)

Sol:





For the given cross section

Location of N.A from top edge is given by

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

$$= \frac{(60\times 20)\times 10 + (20\times 150)\times(20+75) + (20\times 120)\times\left(20+150+\frac{20}{2}\right)}{(60\times 20)+(20\times 150)+(20\times 120)}$$

$$= \frac{12000+285000+432000}{1200+3000+2400}$$

= 110.45 mm

Calculation of moment of inertia of c/s about axis

$$= \left[\left(\frac{60\times 20^3}{12}\right) + (60\times 20)\times(110.45-10)^2\right] + \left[\left(\frac{150^3\times 20}{12}\right) + (20\times 150)(110.45-95)^2\right]$$

$$+\left[\left(\frac{120\times 20^3}{12}\right) + (20\times 120)(79.55-10)^2\right]$$

$$I = (40000+12108243+5625000+716107.5+80000+11609286)$$

$I = 30178636.5\ \text{mm}^4$

At a distance '3m' from end A, shear force

F = 80 kN – 20 × 3

F = 20 kN

∴ Shear stress at junction of flange (1) and web (2) in flange (1)

$$a\bar{y} = (60\times 20)\times(110.45-10)$$

$$a\bar{y} = 120540$$

∴ $$\tau = \frac{Fa\bar{y}}{Ib}$$

$$= \frac{20\times 120540}{30178636.5\times 60} = 1.33\ \text{N/mm}^2$$

Shear stress at junction of flange (1) and web (2) in web

$$a\bar{y} = 120540$$

$$\tau = \frac{Fa\bar{y}}{Ib} = \frac{20\times 120540\times 10^3}{30178636.5\times 20} = 3.994\ \text{N/mm}^2$$

Shear stress at neutral axis:

$$a\bar{y} = (60\times 20)\times(110.45-10)+[20\times(110.45-20)\times(110.45-65.225)]$$

$= 120540 + 81812.025$

$= 202352.025$

$$\tau = \frac{20\times 202352.025\times 10^3}{30178636.5\times 20}$$

$= 6.705\ N/mm^2$

Shear stress at junction of web (2) and flange (3) in web

$$a\bar{y} = (20\times 120)(79.55-10)$$

$$a\bar{y} = 166920$$

$$\tau = \frac{Fa\bar{y}}{Ib}$$

$$= \frac{20\times 166920\times 10^3}{330178636.5\times 20}$$

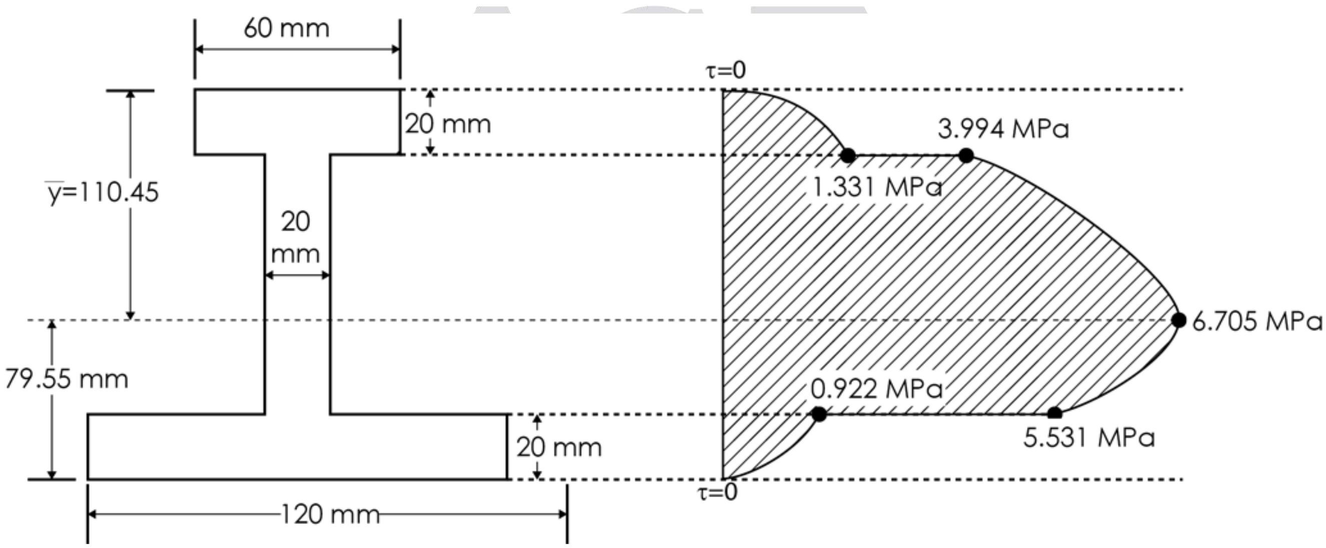
$= 5.531\ N/mm^2$

Shear stress of junction of web (2) and flange (3) in flange

$$a\bar{y} = 166920$$

$$\tau = \frac{20\times 166920\times 10^3}{30178636.5\times 120}$$

$= 0.922\ N/mm^2$



04. (a) A single solid circular shaft 400 mm diameter running at 200 RPM, is to be replaced by two hollow circular shafts of equal size running at 100 RPM and developing 50% additional power. The internal diameter of hollow shaft may be taken as one third of their external diameters. If the working stress of the new shaft is 30% greater than that of the former, find the external and internal diameters of the hollow shafts. (20 M)

Sol:

$$\text{Power } P = \frac{2\pi NT}{60} \qquad \therefore\ T = \frac{60.P}{2\pi N} \quad .... (1)$$

$$\text{Torsion, } \frac{T}{J} = \frac{\tau}{R} \qquad \therefore\ T = \tau \times \frac{J}{R}$$

$$\therefore\ \frac{J}{R} = \frac{T}{\tau} = \frac{60P}{2\pi N\tau}$$

$$\therefore\ \frac{P}{\tau} = \frac{2\pi}{60}\cdot N\cdot\frac{J}{R}$$

i) **Solid shaft**

$$\frac{P_1}{\tau_1} = \frac{2\pi}{60}\times 200\times\frac{\pi}{32}\times 400^4\times\frac{1}{200} = 263.19$$

ii) **Hollow shaft**

$$\frac{P_2}{\tau_2} = \frac{2\pi}{60}N_2\cdot\frac{J}{R} \quad ... (3)$$

Given $P_2 = 1.5\ P_1$ and $\tau_2 = 1.3\ \tau_1$

$$\frac{P_2}{P_1} = \frac{1.5}{1.3}\cdot\frac{P_1}{\tau_1} = \frac{1.5}{1.3}\times 263.19\times 10^6 = 303.68\times 10^6$$

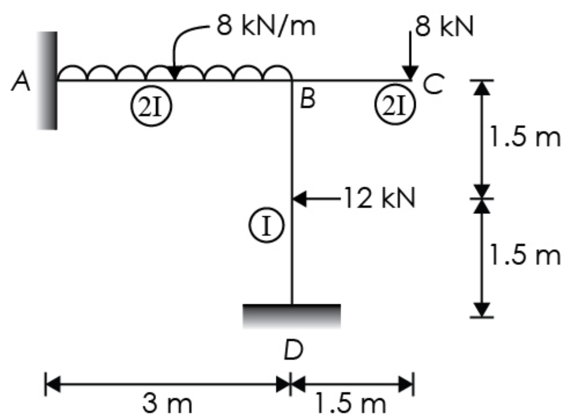
from (3) and (4)

$$303.68\times 10^6 = \frac{2\pi}{60}\times 100\times\frac{\pi}{324}\left(D_0^4 - D_1^4\right)\times\frac{2}{D_0}$$

$$147.69\times 10^6 = \frac{D_0^4 - Di^4}{D_o} = \frac{D_0^4 - (D_0/3)^4}{D_o} = D_0^3\times\left\{1-\left(\frac{1}{3}\right)^4\right\}$$

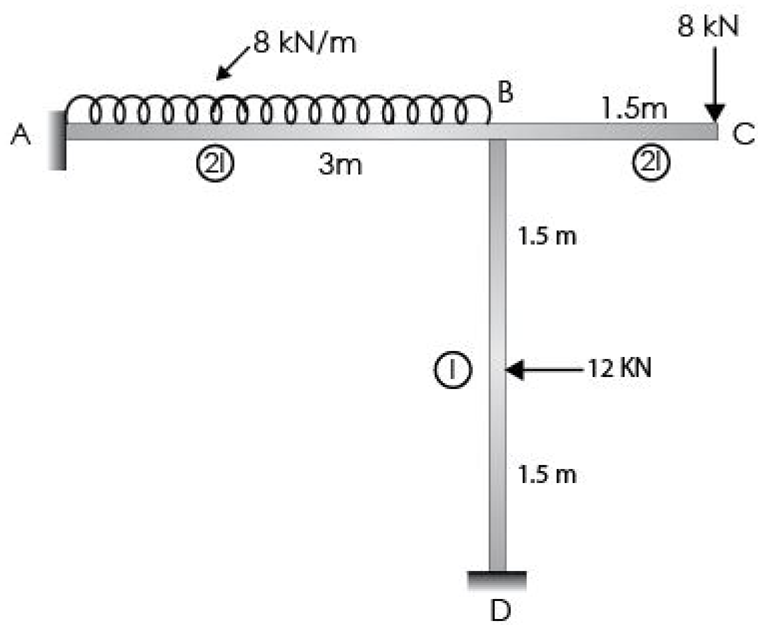
$D_0 = 530.78$ mm $\qquad D_i = 176.93$ mm

04. (b)



Analyse the frame shown in figure by slope deflection method and draw the B.M.D. and S.F.D. (20 M)

Sol:

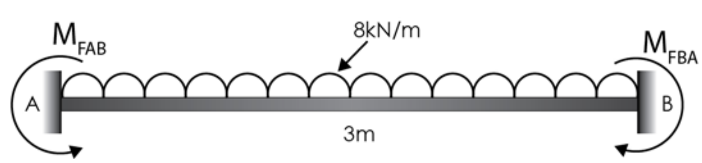


Step-1: Fixed end moments:

For span AB:

$$M_{FAB} = \frac{-wL^2}{12} = \frac{-8\times 3^2}{12} = -6\,\text{kN-m}$$

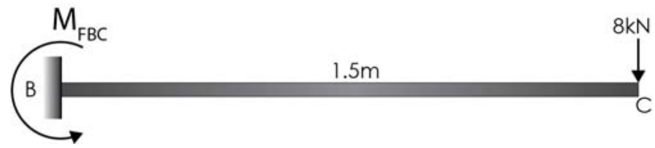
$$M_{FBA} = \frac{+wL^2}{12} = +6\,\text{kN-m}$$



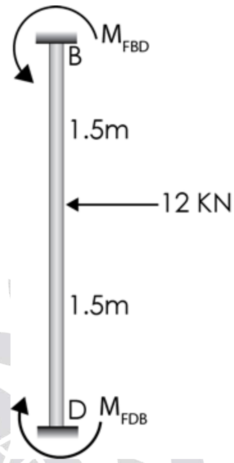
For span BC:

$M_{FBC} = M_{BC} = -8 \times 1.5$

$= -12$ kN-m



For span BD:



$$M_{FBD} = \frac{-WL}{8} = \frac{-12 \times 3}{8}$$

$= -4.5$ kN-m

$$M_{FDB} = +\frac{WL}{8} = \frac{+12 \times 3}{8}$$

$= +4.5$ kN-m

Step-2:Boundary conditions:

$\theta_A = \theta_D = 0$ (fixed)

$\delta = 0$ (non-sway frame)

Step-3: Slope-deflection equation:

$$M_{AB} = M_{FAB} + \frac{2EI}{L}\left[2\cancel{\theta_A}^{\,o} + \theta_B - \cancel{\frac{3\delta}{L}}^{\,o}\right]$$

$$= 6 + \frac{2E(2I)}{3}\theta_B$$

$$= -6 + \frac{4EI\theta_B}{3} \quad \text{\_\_\_\_\_\_ (1)}$$

$$M_{BA} = M_{FBA} + \frac{2EI}{L}\left[2\theta_B + \cancel{\theta_A}^{\,o} - \cancel{\frac{3\delta}{L}}^{\,o}\right]$$

$$= 6 + \frac{2E(2I)}{3}(2\theta_B)$$

$$= 6 + \frac{8EI\theta_B}{3} \quad \text{\_\_\_\_\_\_\_\_ (2)}$$

$M_{BC} = M_{FBC} = -12$ kN-m

$$M_{BD} = M_{FBD} + \frac{2EI}{L}\left[2\theta_B + \theta_D - \frac{3\delta}{L}\right]$$

$$= -4.5 + \frac{4EI\theta_B}{3} \qquad \_\_\_\_ (3)$$

$$M_{DB} = M_{FDB} + \frac{2EI}{L}\left[2\theta_D + \theta_B - \frac{3\delta}{L}\right]$$

$$= 4.5 + \frac{2EI\theta_B}{3} \qquad \_\_\_\_ (4)$$

Step-4: Joint equilibrium equation

$$M_{BA} + M_{BC} + M_{BD} = 0 \qquad \_\_\_\_\_\_ (5)$$

$$6 + \frac{8EI\theta_B}{3} - 12 - 4.5 + \frac{4EI\theta_B}{3} = 0$$

$$4EI\theta_B - 10.5 = 0$$

$$\theta_B = \frac{2.625}{EI}$$

Step 5: Final moments

By substituting 'θ_B' in equations (1) to (4), we get final moments

$$M_{AB} = -6 + \frac{4EI}{3}\left(\frac{2.625}{EI}\right) = -2.5 \text{ kN-m}$$

$$M_{BA} = 6 + \frac{8EI}{3}\left(\frac{2.625}{EI}\right) = 13 \text{ kN-m}$$

$$M_{BC} = -12 \text{ kN-m}$$

$$M_{BD} = -4.5 + \frac{4EI}{3}\left(\frac{2.625}{EI}\right) = -1 \text{ kN-m}$$

$$M_{DB} = 4.5 + \frac{2EI\theta_B}{3} = 4.5 + \frac{2EI}{3}\left(\frac{2.625}{EI}\right)$$

$$= 6.25 \text{ kN-m}$$

Final Moments:

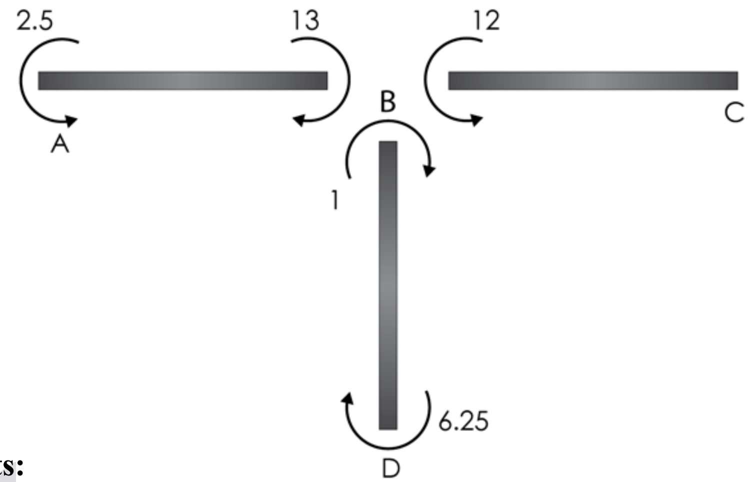
$M_{AB} = -2.5$ kN-m

$M_{BA} = 13$ kN-m

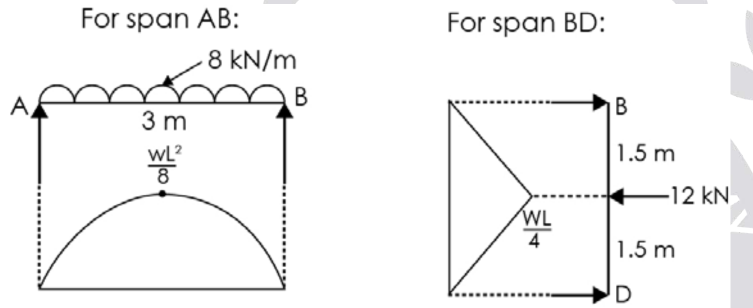
$M_{BC} = -12$ kN-m

$M_{BD} = -1$ kN-m

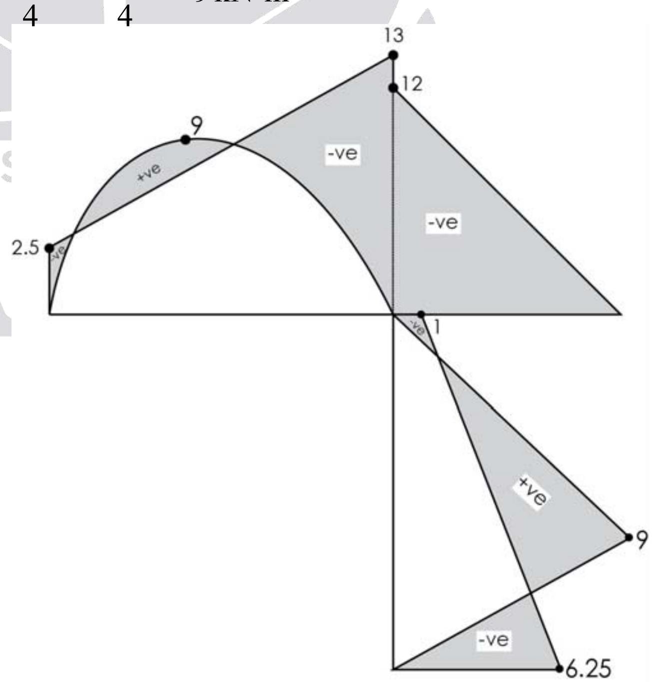
$M_{DB} = 6.25$ kN-m



Step 6: Simply supported moments:



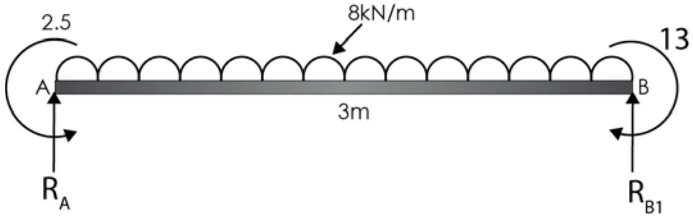
$$\frac{wL^2}{8} = \frac{8 \times 3^2}{8} = 9\,\text{kN-m} \qquad \frac{wL}{4} = \frac{12 \times 3}{4} = 9\,\text{kN-m}$$



Note: Bending moment plotted on tension side

Bending Moment Diagram

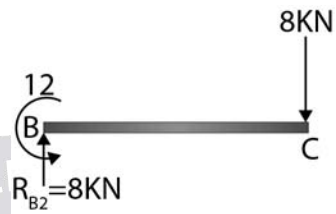
Step 7: Support reactions



$\Sigma v = 0$

$R_A + R_{B1} = 8 \times 3$

Taking moment about A



$\Sigma M_A = 0$ (-ve anticlockwise, +ve clockwise)

$$-R_{B1} \times 3 + 13 + 8 \times 3 \times \frac{3}{2} - 2.5 = 0$$

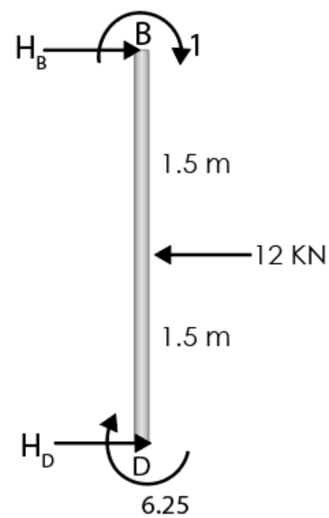
$R_{B1} = 15.5$ kN (↑)

$R_A = 8.5$ kN (↑)

$\Sigma H = 0$

$H_B + H_D = 12$

Taking moment abut 'B'



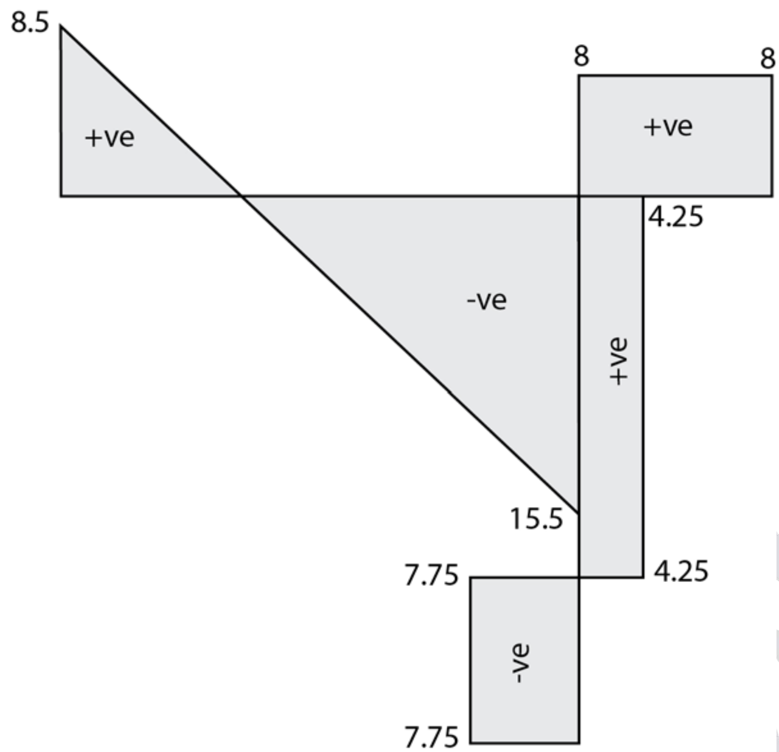
$\Sigma M_B = 0$ (-ve anticlockwise, +ve clockwise)

$-H_D \times 3 + 6.25 + 12 \times 1.5 - 1 = 0$

$H_D = 7.75$ kN (→)

$H_B = 12 - 7.75$

$= 4.25$ kN (→)



Shear Force Diagram

04. (c)

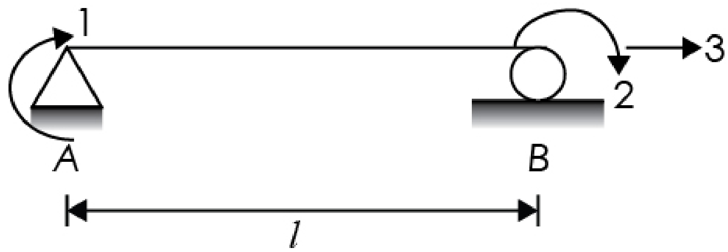
(i) Distinguish between Flexibility Method and Stiffness Method used for analysis of structures. (4 M)

Sol:

Step	Force method (flexibility or compatibility method)	Displacement method (stiffness or equilibrium method)
1	Determine the degree of static indeterminacy (degree of redundancy), n.	Determine the degree of kinematic indeterminacy, (degree of freedom), n.
2.	Choose the redundants	Identify the independent displacement components
3.	Assign coordinates 1, 2, ..., n to the redundants.	Assign coordinates 1, 2, ..., n to the independent displacement components.
4.	Remove all the redundants to obtain the release structure	Prevent all the independent displacement components to obtain the restrained structure.
5.	Determine (Δ_L), the displacements at the coordinates due to the applied loads acting on the released structure.	Determine (P'), the forces required at the coordinates in the restrained structure due to the loads other than those acting at the coordinates.
6.	Determine (Δ_R), the displacements at the coordinates due to the redundants acting on the released structure.	Determine (P_Δ), the forces required at the coordinates in the unrestrained structure to cause the independent displacement components (Δ).
7.	Compute the net displacements at the coordinates. $(\Delta) = (\Delta_L) + (\Delta_R)$	Compute the net forces at the coordinates. $(P) = (P') + (P_\Delta)$
8.	Use the conditions of compatibility of displacements to compute the redundnats $(P) = (\delta)^{-1}\{(\Delta) - (\Delta_L)\}$	Use the conditions of equilibrium of forces to compute the displacements. $(\Delta) = (k)^{-1}\{(P) - (P')\}$
9.	Knowing the redundants, compute the internal member forces by using equations of statics.	Knowing the displacements, compute the internal member forces by using slope-deflection equations.

04. (c)

(ii)



Briefly explain the procedure and then develop the stiffness matrix for the beam element shown in figure with respect to the degrees of freedom 1, 2 and 3. The cross-sectional area *A* and flexural rigidity *EI* are constant for the beam. (16 M)

Sol:

The following steps are to be taken to get the required solution by stiffness method.

(i) Determine the degree of kinematic indeterminacy

(ii) Assign the coordinate numbers to the unknown displacements

(iii) Impose restraints in all coordinate directions to get a fully restrained structure.

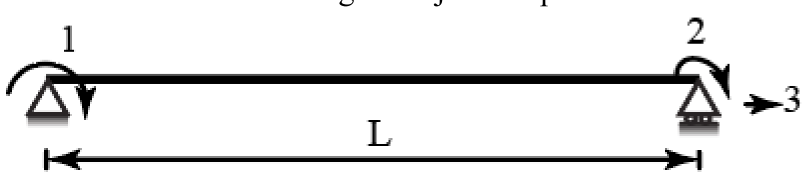
(iv) Determine the forces developed in each of the coordinate directions of a fully restrained structure. It is called as (P_L) .

(v) Determine 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 coordinate directions. For this, only structures approach is explained in this text.

(vi) Observing the final forces in various coordinate directions, note the final forces (P).

(vii) Solve the stiffness equations (K) (Δ) = (P – P_L) to get the displacements D in the coordinate directions.

(viii) Calculate the check it forces using these joint displacements.



$$K = \begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix}$$

Applying unit displacement in coordinate direction 1

$k_{11} = \frac{4EI}{L}$; $k_{21} = \frac{2EI}{L}$; $k_{31} = 0$

Applying unit displacement in coordinate direction '2'

$k_{12} = \frac{2EI}{L}$; $k_{22} = \frac{4EI}{L}$; $k_{32} = 0$

Applying unit displacement in coordinate direction 'c'

$k_{13} = 0$, $k_{23} = 0$, $k_{33} = \frac{AE}{L}$

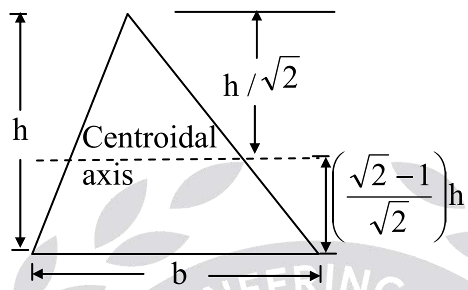
Stiffness matrix, $K = \begin{bmatrix} \frac{4EI}{L} & \frac{2EI}{L} & 0 \\ \frac{2EI}{L} & \frac{4EI}{L} & 0 \\ 0 & 0 & \frac{AE}{L} \end{bmatrix}$

SECTION – B

05. (a) Find the shape factor of a triangular section of base *b* and height *h* for bending about an axis parallel to the base. (12 M)

05. (a)

Sol:



Shape factor $= \dfrac{Z_P}{Z_e}$

Section modulus, $Z_e = \dfrac{I}{y_{max}}$

Moment of Inertia, about C.G

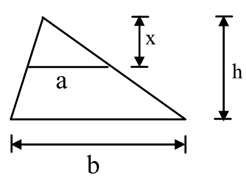
$$I = \frac{bh^3}{36}$$

$$y_{max} = \frac{2h}{3}$$

$$Z_e = \frac{bh^3}{36}\cdot\frac{3}{2h}$$

$$Z_e = \frac{bh^2}{24}$$

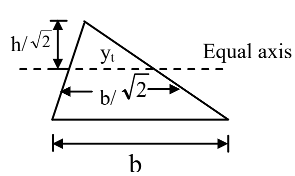
Plastic modulus, $Z_p = \dfrac{A}{2}(y_t + y_b)$



Given, $x = \dfrac{h}{\sqrt{2}}$

$$\frac{x}{a} = \frac{h}{b}$$

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



$$y_t = \frac{1}{3}\frac{h}{\sqrt{2}} = 0.236\,h$$

$$y_b = \frac{1}{3}\left(\frac{2b + \frac{b}{\sqrt{2}}}{b + \frac{b}{\sqrt{2}}}\right)\left(\frac{\sqrt{2}-1}{\sqrt{2}}\right)h$$

$y_b = 0.155$ h

$$Z_P = \frac{1}{2}\left(\frac{1}{2}b \times h\right)(0.236\,h + 0.155\,h)$$

$Z_P = 0.09775\ bh^2$

$$\text{Shape factor } = \frac{Z_p}{Z_e} = \frac{0.09775bh^2}{\frac{bh^2}{24}}$$

S = 2.34

05. (b) A construction project has the following characteristics :

Activity	Preceding Activity	Duration (weeks)
A	None	5
B	A	2
C	A	6
D	B	12
E	D	10
F	D	5
G	D	5
H	B	9
I	C, E	1
J	G	2
K	F, I, J	3
L	K	9
M	H, G	7
N	M	9

(i) Draw a network for this project

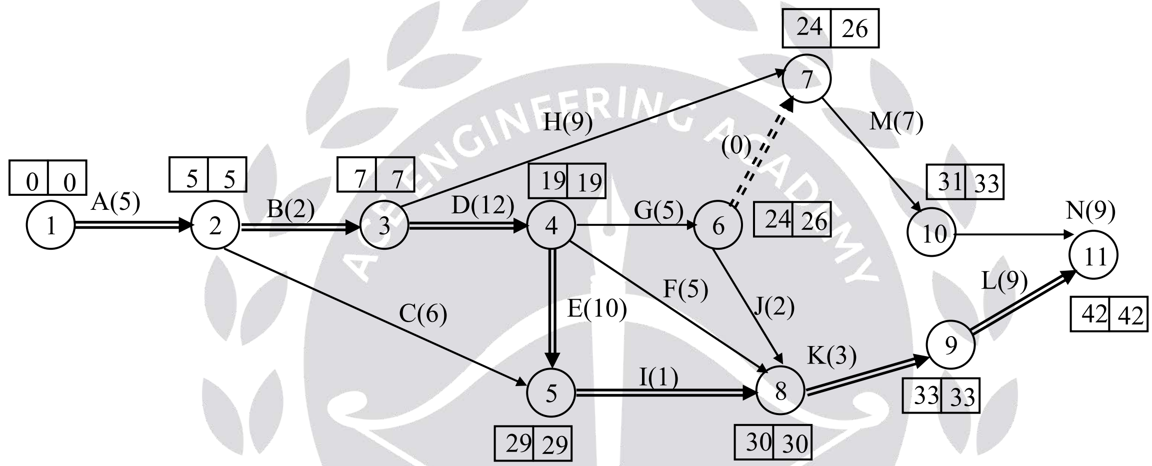
(ii) Find various paths and the critical path as well as the project completion time

(iii) Prepare an activity schedule showing Earliest Start time, Earliest Finish time, Latest Start time, Latest Finish time and float for each activity.

(iv) Will the critical path change if activity G takes 10 weeks instead of 5 weeks? If so, what will be the new critical path? (12 M)

Sol:

(i)



(ii)

Path	Duration
A-B-H-M-N	5+2+9+7+9 = 32
A-B-D-G-Dummy -M-N	5+2+12+5+0+7+9 = 40
A-B-D-G-J-K-L	5+2+12+5+2+3+9 = 38
A-B-D-F-K-L	5+2+2+5+3+9 = 36
A-B-D-E-I-K-L	5+2+12+10+1+3+9 = 42
A-C-I-K-L	5+6+1+3+9 = 24

Critical path: A-B-D-E-I-K-L

Project completion time = 42 weeks

(iii)

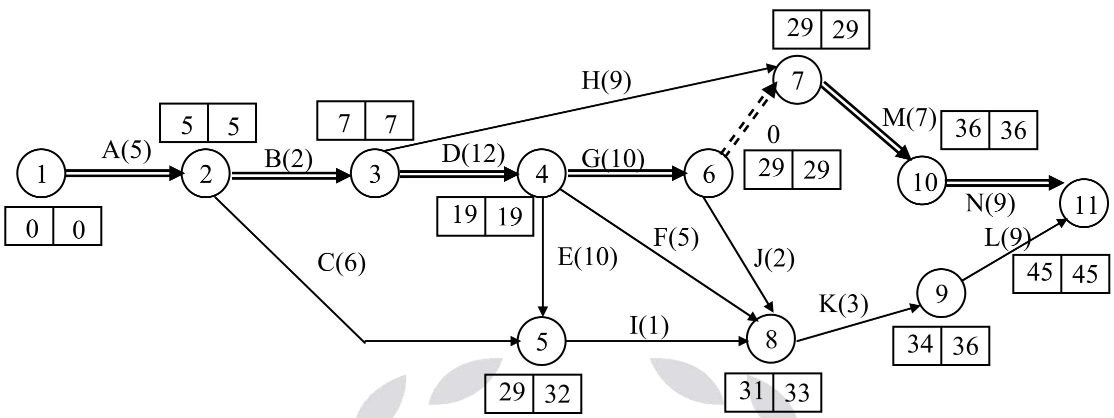
Activity	Duration (t_{ij})	EST (T_E^i)	EFT (EST + t_{ij})	LST (LFT – t_{ij})	LFT (T_L^j)	Float (LST – EST) (or) (LFT – EFT)
A	5	0	5	0	5	0
B	2	5	7	5	7	0
C	6	5	11	23	29	18
D	12	7	19	7	19	0
E	10	19	29	19	29	0
F	5	19	24	25	30	6
G	5	19	24	23	28	4
H	9	7	16	17	26	10
I	1	29	30	29	30	0
J	2	24	26	28	30	4
K	3	30	33	30	33	0
L	9	33	42	33	42	0
M	7	24	31	26	33	2
N	9	31	40	33	42	2

(iv) If activity 'G' takes 10 weeks instead of 5 weeks

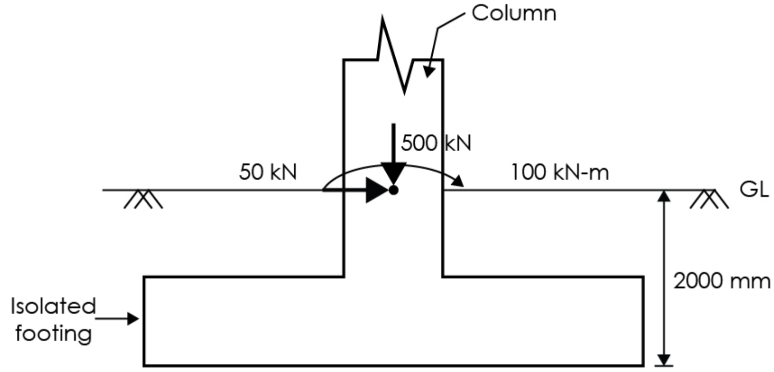
Path	Duration
A-B-D-G-Dummy-M-N	5+2+12+10+0+7+9= 45
A-B-D-G-J-K-L	5+2+12+10+2+3+9 = 43

If activity 'G' duration changes from 5 to 10 weeks then the critical path changes and project duration becomes 45 weeks. (Instead of 42 weeks).

Network with new critical path:-



05. (c)



An isolated footing is transferring load from a column (300 mm × 300 mm) as shown in figure. Arrange the plan dimensions of footing so that there will be uniform soil pressure intensity. Following parameters may be used.

1. **Column size: 300 mm × 300 mm**
2. **Safe Bearing capacity: 100 kN/m^2**
3. **Dead weight of footing and soil weight over it may be taken as 10% of vertical load of column.** **(12 M)**

Sol:

Given size of column = 300 mm × 300 mm

Axial load (W) = 500 kN

Horizontal force at the ground level = 50 kN

Bending moment at the level of footing = 50 × 2 = 100 kN.m

Additional Bending moment shown in figure = 100 kNm

Total bending moment = 100 + 100 = 200 kN.m

SBC of soil = 100 kN/m^2

Dead weight of footing and soil over it = 10% of 500 = 50 kN

Step1: Dimensions of footing:

Total axial load including self weight

W + W′ = 500 + 50 = 550 kN

Base Area of footing required = $\dfrac{W}{SBC}$

$$= \frac{550}{100}$$

$$= 5.5 \text{ m}^2$$

Designing a square footing of side 'B'

$\Rightarrow B^2 = 5.5 \text{ m}^2$

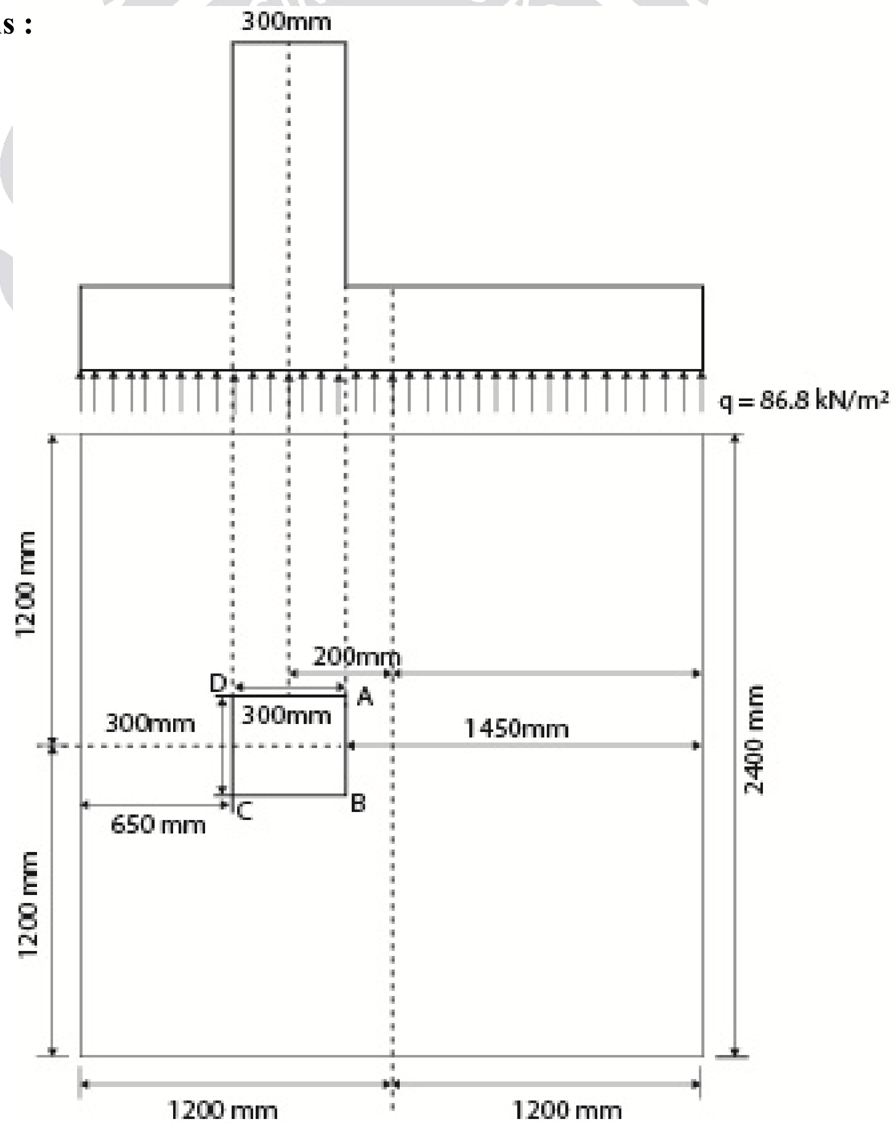
$\Rightarrow B = 2.345$ m

$\therefore$ Provide B = 2.4 m

For the pressure to be uniform, the eccentricity required between centroid of column and footing

$$= \frac{M}{W} = \frac{200}{500} = 0.4 \text{ m} = 400 \text{ mm}$$

Plan Dimensions :



Net upward soil pressure $P_o = \dfrac{W}{A}$

$$= \frac{500}{2.4 \times 2.4} = 86.8 \text{ kN/m}^2$$

Cantilever length to the right of AB

= 1200 + 400 –150 = 1450 mm

Cantilever length to the right of CD = 1200–400–150 = 650 mm

05. (d) An ISMB 400 beam is spliced at a section carrying factored bending moment of 120 kNm and factored shear force of 80 kN. The splice is to be designed so that the flange splice will carry the bending moment and the web splice will carry the shear force. Field welding with 8 mm fillet will be used. Determine the size of 100 mm wide flange plate using the following data: t_f =16 mm; t_w = 8.9 mm and b_f = 140 mm

(12 M)

Sol: Factored bending moment M = 120 kN-m

Factored shear force V = 80 kN

Width of flange plate b = 100mm

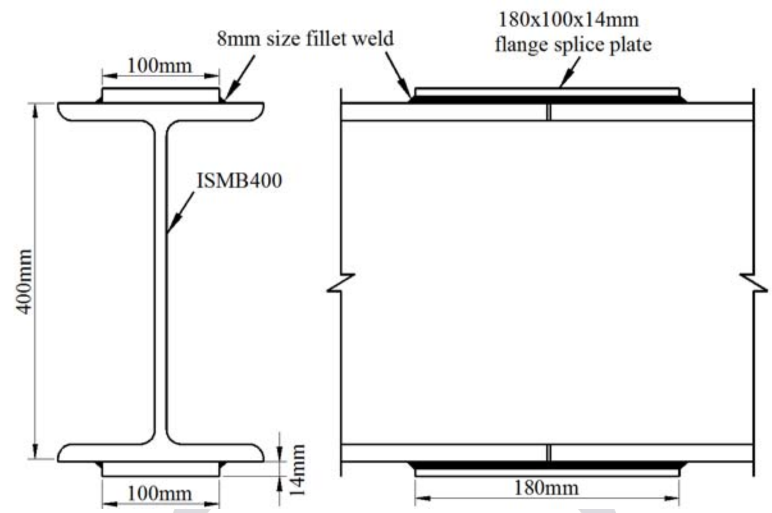
Depth of beam section h=400mm

Assume yield stress of steel f_y = 250 Mpa

Assume Ultimate tensile stress of weld and steel f_u=410Mpa

Partial safety factor for filed weld γ_{mw} = 1.50

Partial safety factor against yield stress γ_{mo} = 1.10



$$\text{Flange force } F = \frac{M}{(h - t_f)} = \frac{120\times10^3}{(400-16)} = 312.5\text{kN}$$

Gross area of flange A_g = 140×16= 2240 mm^2

$$\text{Flange capcity} = \frac{A_g \times f_y}{\gamma_{mo}} = \frac{2240\times250}{1.10} = 509.09\times10^3\text{N} = 509.9\text{kN} \geq F = 312.5\text{kN}$$

which is safe

Size of fillet weld S = 8mm

Effective throat thickness of fillet weld t_t = K×S = 0.7 × 8 = 5.6mm

The effective length of fillet weld required

$$L_w = \frac{F}{t_t \times f_u/\sqrt{3}\gamma_{mw}} = \frac{312.5\times10^3}{5.6\times410/\sqrt{3}\times1.5} = 353.6\text{mm} = 360\text{mm}$$

Minimum length of splice plate (by providing weld on two longitudinal edges of flange splice plate)

l_s= 360/2 = 180mm

Width of flange splice plate b_s = 100mm

Thickness of splice plate required t_s = 312.5×10^3/(250×100)/1.10 =13.75mm≈14mm

$$\text{Flange splice plate capacity} = \frac{A_g \times f_y}{\gamma_{mo}} = \frac{100\times14\times250}{1.10} = 318.18.09\times10^3\text{N} = 318.18\text{kN} \geq F = 312.5\text{kN}$$

which is safe

Use flange splice plate of size 180mm × 100mm × 14mm

05. (e) A cylindrical water tank of capacity 500 m^3 is resting on ground and have a free flexible joint at base (vertical wall-base slab connection). Overall height of tank is restricted to 4.3 m (it includes a free board of 0.3 m). Design the vertical cylindrical wall of tank only. Following parameters may be used for design, if required :

1. $\sigma_{cbc} = 10\ N/mm^2$

2. $\sigma_{cbt} = 2.0\ N/mm^2$

3. $\sigma_{ct} = 1.5\ N/mm^2$

4. $\sigma_{st} = 130\ N/mm^2$

5. $\gamma_w = 10\ kN/m^3$

6. Main reinforcing bar diameter : 16 mm

7. m = 10.

(12 M)

Sol: Given:

Capacity of water tank = 500 m^3

Freeboard = 300 mm

Overall height = 4.3 m

Depth of water in tank = 4.3 – 0.3 = 4 m

If 'D' is inside diameter of tank

Volume of the tank = 500 m^3

$$\frac{\pi}{4}D^2 \times 4 = 500$$

$\Rightarrow$ D = 12.61 m say 13 m

$\therefore$ Provide inside diameter of 13 m

Unit weight of water = 10 kN/m^3

Design:

Consider bottom 1 m height of the wall. Pressure intensity corresponding to the *centre of bottom 1m* height of wall, $p = \gamma_w.h = 10 \times 3.5 = 35\ kN/m^2$

Maximum hoop tension, $T = \dfrac{pD}{2}$

$$= \frac{35 \times 13}{2}$$

= 227.5 kN per 'm' ht at base

σ_{st} = 130 N/mm^2

∴ Area of hoop steel required at base

$$A_{st} = \frac{227.5 \times 10^3}{130} = 1750 \text{ mm}^2\text{/m height}$$

Using 16 mm dia bars, Spacing = $\dfrac{1000 \times \frac{\pi}{4} \times 16^2}{1750}$

= 114.9 mm

∴ Provide hoop steel of 16 mmϕ @ 110 mm spacing

$$\text{Actual } A_{st} = \frac{1000 \times \frac{\pi}{4} \times 16^2}{110}$$

= 1828 mm^2/m

Calculation of thickness of wall:

- From allowable tensile stress criteria of concrete

If 't' is thickness of wall,

$$\sigma_{ct} = \frac{T}{A + (m-1)A_{st}}$$

$$1.5 = \frac{227.500 \times 10^3}{1000t + (10-1)1828}$$

1500 t + 24678 = 227500

⇒ t = 135.21mm say 150 mm

- Empirical relation for thickness

= 30 mm/m depth of water + 50 mm

= 30 × 4 + 50 = 170 mm

Hence, provide 170 mm thickness for side wall

(as per code minimum thickness is 150 mm)

At top spacing of hoops can be increased

Minimum reinforcement = 0.3%

$$= \frac{0.3}{100} \times 1000 \times 170 = 510 \text{ mm}^2$$

$\therefore$ Spacing of hoops at top $= \frac{\pi}{4} \times 16^2 \times \frac{1000}{510}$

$= 394.23$ mm

Max. permissible spacing = 3t = 3 × 170 = 510 mm

$\therefore$ Provide spacing @ 300 mm c/c at top part of wall

Since thickness < 225 mm, hoops are placed at centre of wall

Vertical Distribution Steel ($A_{st\,2}$):

Distribution reinforcement = 0.3% for 100 mm thick wall

= 0.2% for 450 mm thick wall

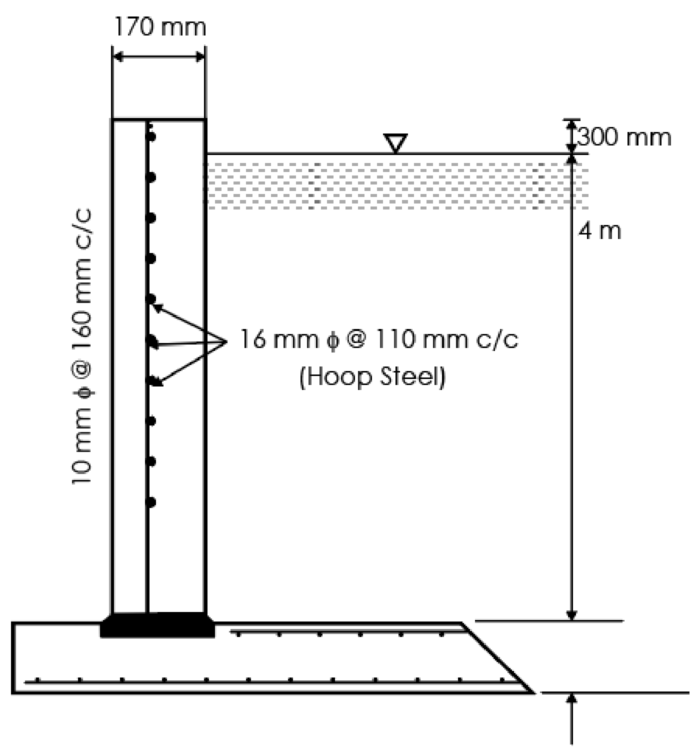
for 170 mm thick wall $A_{st\,1}$

$$= 0.3 - 0.1\left(\frac{170-100}{450-100}\right) = 0.28\%$$

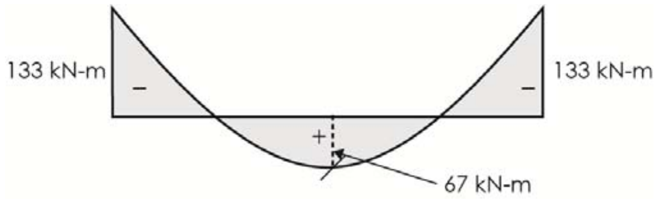
$$A_{st\,2} = \frac{0.28}{100}\times 1000\times 170 = 476\text{ mm}^2$$

Using 10 mm ϕ bars spacing $= \frac{\pi}{4}\times 10^2 \times \frac{1000}{476} = 164.99$ mm

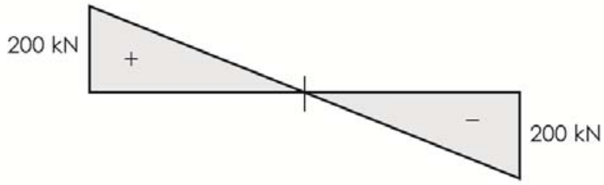
∴ Provide 10 mm ϕ @ 160 mm c/c in vertical direction. It will also be useful for tying hoop reinforcement



06. (a)



Service State Bending Moment Diagram



Service State Shear Force Diagram

Design the reinforcement at critical section only for a beam for which Bending moment and shear force diagram has been shown in figure. Following parameters may be used for design:

1. M-20 grade concrete **2. Fe-415 grade steel**

3. Nominal concrete cover : 30 mm **4. (Depth/width) ratio of beam : (02)**

5. Diameter of reinforcing bar: 20 mm for flexure reinforcement and 8 mm for shear reinforcement

6. Shear strength of concrete = 0.6 N/mm² 7. $(M_{u,\,lim}/f_{ck}\,bd^2) = 0.138$

8. $(p_{t,\,lim}\,f_y/f_{ck}) = 19.82$ **(20 M)**

Sol: Given:

At critical section (i.e. @ supports)

Service BM = 133 kN (Hogging)

Service SF = 200 kN

Design Bending Moment (M_u) = 1.5 × 133 = 200 kN.m

Design SF (V_u) = 1.5×200 = 300 kN

b = width of beam = D/2 (Given)

d = effective depth of beam

Nominal cover to concrete = 30 mm

Note: Nominal cover is defined as the cover to all steel reinforcement, including stirrups or links

D = overall depth of beam = (d + 30 + 20/2 + 8) mm

$M_u = 0.138 f_{ck}.\, bd^2$

$d = \sqrt{\dfrac{M_u}{0.138 f_{ck}.b}}$ $\quad b = \dfrac{D}{2}$ [Taking $b \approx \dfrac{d}{2}$] to solve

$$\Rightarrow d = \sqrt{\frac{200\times10^6}{0.138\times20(d/2)}}$$

$1.38d^3 = 200 \times 10^6$

$\Rightarrow \quad d = 525.3$ mm

Provide overall depth, D = 525.3 + 30 + 20/2 + 8 = 573.3 mm

Say, D = 600 mm

Width of the beam $b = \dfrac{600}{2} = 300\,\text{mm}$

Effective depth, $d_{provided}$ = 600 – 30 – 20/2 – 8 = 552 mm

Provided size of the beam = 300 mm × 600 mm

Reinforcement required at support for flexure:

Design BM (M_u) = 200 kN.m

Limiting moment of resistance

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

$= 0.138 \times 20 \times 300 \times 552^2$

$= 252.2 \times 10^6$ N.mm

$= 252.2$ kN.m

Since, $M_u < M_{u\,lim}$

∴ The given beam is under reinforced singly reinforced

Designed for top steel near support:

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

$$= \frac{0.5\times20}{415}\left[1-\sqrt{1-\frac{4.6\times200\times10^6}{20\times300\times552^2}}\right]300\times552$$

$= 1197.10\ \text{mm}^2$

No. of bars required

$$n = \frac{A_{st}}{a_{st}} = \frac{1197.10}{\frac{\pi}{4}\times20^2} = 3.81 \simeq 4$$

Provided 4 # 20 mm diameter Fe 415 bars at top at support

Reinforcement required at mid span

Design moment = 1.5×67

= 100.5 kN.m

$M_u < M_{u\,lim}$

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 100.5\times 10^6}{20\times 300\times 552^2}}\right] 300\times 552$$

$= 558.65 \text{ mm}^2$

No of bars required

$$n = \frac{A_{st}}{a_{st}} = \frac{558}{\frac{\pi}{4}\times 20^2} = 1.77$$

Providing 2# 20 ϕ bars at bottom face at mid span

Shear reinforcement:

Design SF, $V_u = 300$ kN

Nominal Shear stress $\tau_v = \frac{V_u}{b.d} = \frac{300\times 10^3}{300\times 552} = 1.81 \text{MPa}$

Maximum shear strength of concrete from IS:456-2000, is 2.8 MPa for M20

$\tau_v < \tau_{c\,max}$ $\therefore$ OK

Shear strength of concrete = 0.6 MPa (Given)

$\tau_v > \tau_c$ $\therefore$ Not safe in shear

Hence design for shear

Design shear force for stirrups:

$V_{us} = V_u - \tau_c \times b\times d$

$= 300 \times 10^3 - 0.6 \times 300 \times 552 = 200640$ N

Design vertical stirrups

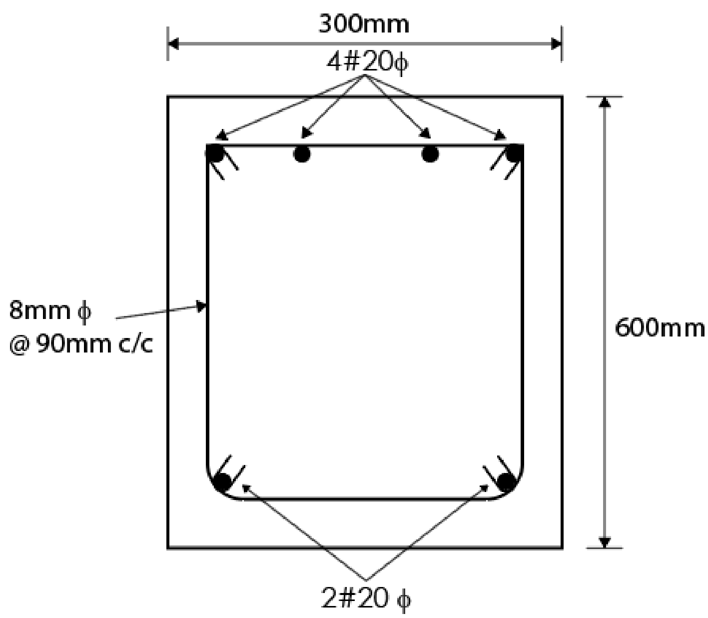
$$V_{us} = 0.87 f_y A_{sv} \frac{d}{s_v}$$

$$200640 = 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times \frac{552}{s_v}$$

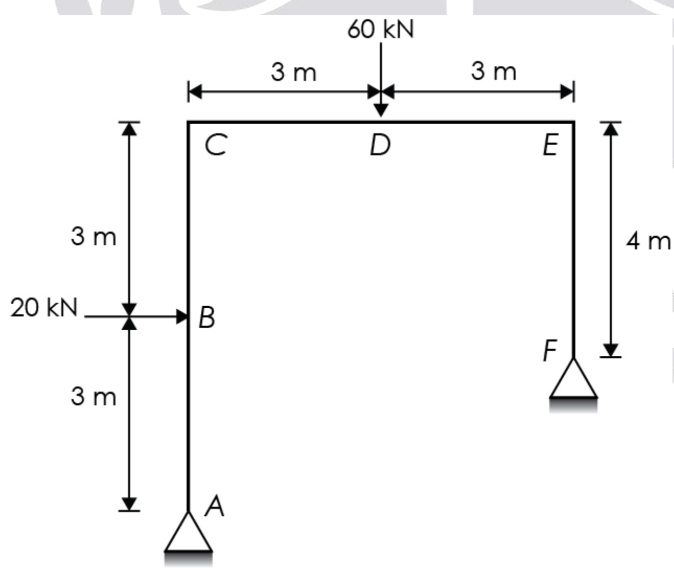
$s_v = 99.85$ mm < 300 mm ∴ OK

Providing 2 legged 8 mm ϕ @ 100 mm c/c

Detailing:



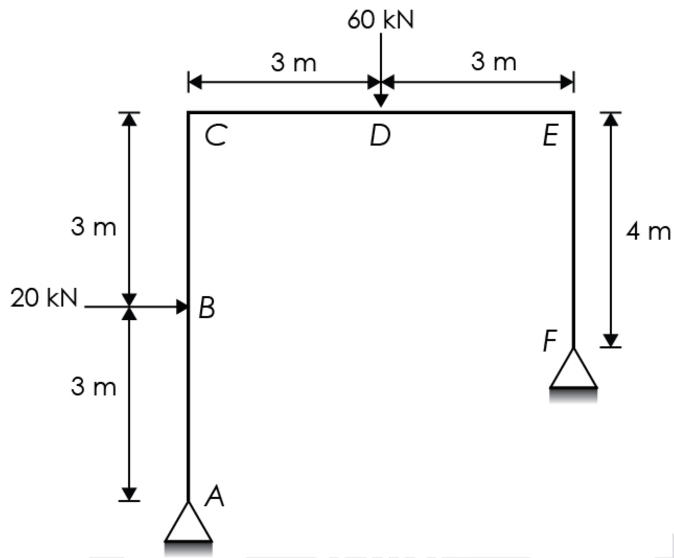
06. (b)



Find the collapse load for a frame of uniform cross-section shown in figure under the applied forces. Also determine the minimum section in steel required to resist the applied forces.

(20 M)

Sol:



Comment:

In this problem collapse load is asked along with design section. The paper setter idea may be collapse moment and design as loads are given.

Static indeterminacy $D_s = D_{se} + D_{si}$

External indeterminacy D_{se} = Number of reactions – Number of equilibrium equations.

$D_{se} = 4–3 = 1$

Internal indeterminacy $D_{si} = 3C$

Where,

C = number of cuts required to open a closed configuration (or) number of closed boxes

$D_{si} = 3\times0 = 0$

$\therefore\ D_s = 1$

Number of possible plastic hinges N = 4 [At B, C, D & E]

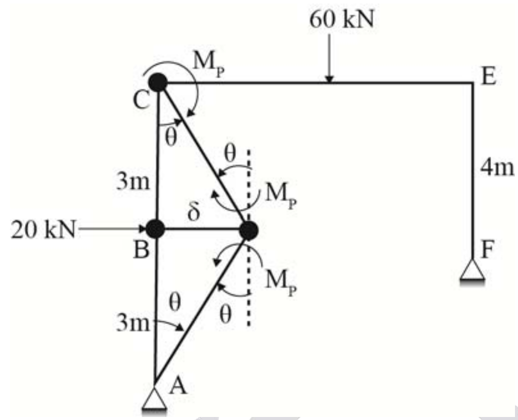
Number of plastic hinges required to form a mechanism

$n = D_s + 1 = 2$

Number of independent mechanism $I = N – D_s$

$I = 4–1 = 3$

Collapse of the frame due to plastic hinges developed at B & C



External work done (W_e) = load × Displacement under the load

$W_e = 20\times\delta \times 60\times 0 = 20\delta$

$\delta = 3\theta$

$W_e = 20 \times 3\theta = 60\times\theta$

Internal work done (W_i) = Moment × Rotation

$= M_p\theta + M_p\theta + M_p\theta$

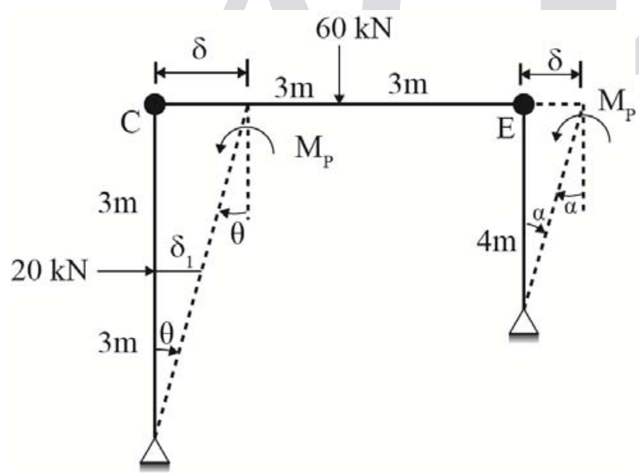
$= 3M_p\theta$

Equating external work done and internal work done

$60\times\theta = 3M_p\theta$

$M_p = 20$ kN-m

Collapse of the frame due to plastic hinges developed at C and E.



$\delta = 6\theta = 4\alpha$

$\alpha = 1.5\theta$

$\delta_1 = 3\theta$

$W_e = 20\delta_1 + 60\times0 = 20\times3\theta = 60\times\theta$

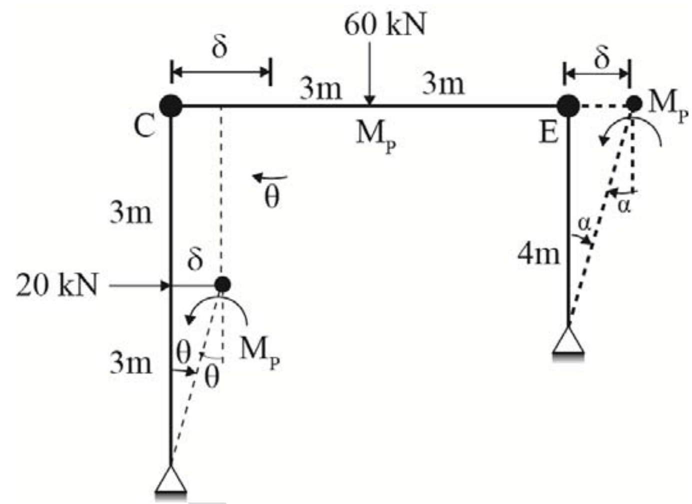
$W_i = M_p\theta + M_p\alpha = M_p\theta + M_p\,(1.5\theta)$

$= 2.5\,M_p\theta$

$W_e = W_i$

$60\times\theta = 2.5\,M_p\theta$

$M_p = 24$ kN-m



Collapse of the frame due to plastic hinges developed at B and E

$\delta = 3\theta$, $\delta = 3\theta = 4\alpha$, $\alpha = \frac{3}{4}\theta$

$W_e = 20\times\delta = 20\times3\theta = 60\theta$, $W_i = M_p\theta + M_p\alpha$

$W_i = M_p\theta + M_p\left(\frac{3}{4}\theta\right)$

$= 1.75\,M_p\theta$

$W_e = W_i$

$60\theta = 1.75\,M_p\theta$

$M_p = 34.28$ kN-m

Combined mechanism: Collapse of frame due to plastic hinges developed at D and E

$\delta = 6\theta = 4\alpha$, $\delta_1 = 3\theta$

$\alpha = \frac{3}{2}\theta$ $\delta_2 = 3\theta$

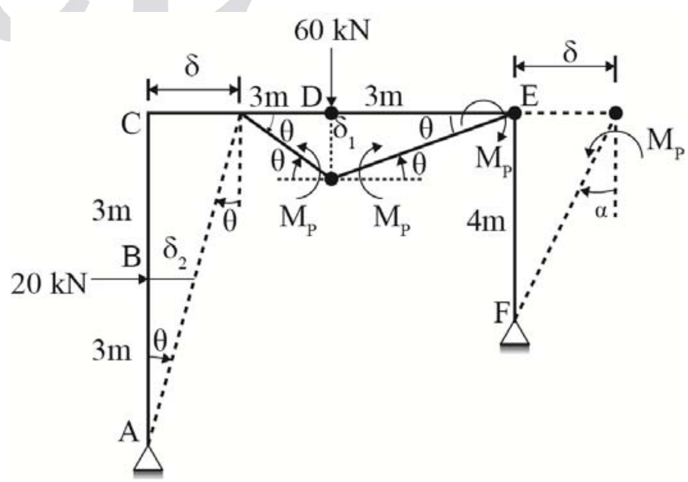
$W_e = 20\delta_2 + 60\delta_1 = 20\times3\theta + 60\times3\theta$

$W_e = 240\,\theta$

$W_i = 3M_p\theta + M_p\alpha = 3M_p\theta + M_p\left(\frac{3}{2}\theta\right)$

$W_i = 4.5\,M_p\theta$

$W_e = W_i$



$240\theta = 4.5\, M_p\theta$, $M_p = 53.33$ kN-m

This is greater than the value of M_p obtained for the previous cases. Hence the greatest moment reached is 53.33 kN-m.

The frame section should therefore be designed for a collapse moment of 53.33 kN-m

The frame will collapse developing plastic hinges at D and E.

Design by uniform section:

Plastic modulus required $(\tau_p) = \dfrac{M_p}{f_y}$

Where,

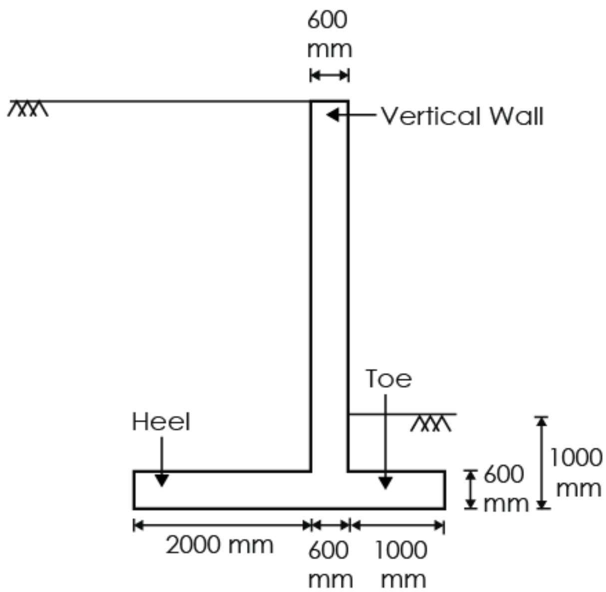
f_y = yield stress = 250 N/mm^2

$$Z_p = \frac{53.33\times10^6}{250} = 213320\,\text{mm}^3 = 213.320\times10^3\ \text{mm}^3$$

Plastic modulus of ISMB 200 = 223.5×10^3 mm^3

I – section (rolled and built up sections) are must efficient and economical shapes.

06. (c)



Design and sketch the reinforcement in vertical wall, Toe slab and Heel slab (for maximum Bending moment and maximum shear force only) for the cantilever retaining structure shown in figure.

Following parameters may be used for design and sketch:

1. **Maximum Bending moment in vertical wall : 400 kN-m, Toe : 160 kN-m and in Heel : 200 kN-m**
2. **Maximum shear force in vertical wall : 200 kN, Toe : 120 kN and in Heel : 160 kN**
3. **Grade of concrete M20.**
4. **Grade of reinforcing bar Fe415.**
5. **Nominal concrete cover 20 mm.**
6. **Development length: 47 × diameter of bar.**
7. **Diameter of main bar in vertical wall : 25 mm, in Heel : 25 mm and in Toe : 25 mm.**
8. **$(M_{u,\,lim}/f_{ck}\,bd^2) = 0.138$.**
9. **$(p_{t,\,lim}\,f_y/f_{ck}) = 19.82$.**
10. **Shear strength of concrete = 0.60 N/mm^2.**

(20 M)

Sol:

(i) Design of vertical wall:-

Effective thickness = 600 –20– $\frac{25}{2}$ = 567.5 mm

Maximum bending moment = 400 kNm.

Occurs at junction of wall and slab

Factored bending moment M_u = 1.5×400 = 600 kNm

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

$= 0.138 \times 20 \times 1000 \times 567.5^2$

$= 888875$ N-mm = 888.9 kN-m

$M_u < M_{u\,limit}$

$\Rightarrow$ under reinforced singly reinforced

Area of tension steel required

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 600\times 10^6}{20\times 1000\times 567.5^2}}\right] 1000\times 567.5 \;\; = 3418.67 \text{ mm}^2$$

$$P_{t\,lim}\frac{f_y}{f_{ck}} = 19.82$$

$$\Rightarrow \frac{A_{st,\,lim}}{bd}\times 100\times\frac{415}{20} = 19.82$$

$$\Rightarrow A_{st,lim} = \frac{19.82\times 20\times 1000\times 567.5}{415\times 100}$$

= 5420 mm^2 > 3418 mm^2

Hence OK

Using 25 mm bars spacing required

$$= \frac{\pi}{4}\times 25^2\times\frac{1000}{3418}$$

= 143.54 mm

$\therefore$ Provide 25 mm bars @ 140 mm c/c

Curtailed at a distance of 47ϕ = 47× 25 = 1175 mm beyond 'A' in toe slab

Distribution reinforcement = 0.12%

$$= \frac{0.12}{100} \times 1000 \times 600 = 720\,\text{mm}^2$$

Using 10 mmϕ bars, spacing = $\frac{\pi}{4} \times 10^2 \times \frac{1000}{720}$

= 109.02 mm

∴ Provide 10 mmϕ bars @ 100mm c/c on inner face of wall.

Provide 10 mm ϕ bars @ 200 mm c/c outer wall as temperature reinforcement on both ways

Check for shear:

Factored shear force $\frac{1}{4} = 1.5 \times 200 = 300\,\text{kN}$

Nominal shear stress = $\frac{V_u}{bd}$

$$= \frac{300 \times 10^3}{1000 \times 567.5} = 0.53 \text{ MPa}$$

Shear strength of concrete = 0.6 MPa (Given)

Hence safe in shear

Reinforcement in the heel slab:

Factored B.M, M_u = 1.5 M = 1.5 × 200 = 300 kN-m

Effective depth, d = D – effective cover

$$= 600 - 20 - \frac{25}{2} = 567.5 \text{ mm}$$

$m_u < m_{u\,lim}$ ∴ U.R.S

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 300 \times 10^6}{20 \times 1000 \times 567.5^2}}\right] 1000 \times 567.5 = 1397.8 \text{ mm}^2$$

Spacing:

$$S = \frac{1000 a_{st}}{A_{st}} = \frac{1000 \times \frac{\pi}{4} \times 25^2}{1397.8} = 351 \text{ mm} > 300 \text{ mm}$$

∴ Not safe

Provide 25 mm ϕ @ 300 mm c/c

Reinforcement in toe slab:

Factored B.M, $m_u = 1.5 \text{ m} \times 160 = 240$ kN-m

Effective depth, $d = 600 - 20 - \frac{25}{2} = 567.5$ mm

Provided 567.5 mm

$m_u < M_{u\,lim}$ ∴ U.R.S

Area of tension required for an U.R.S

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 240 \times 10^6}{20 \times 1000 \times 567.5^2}}\right] 1000 \times 567.5$$

$= 1226 \text{ mm}^2$

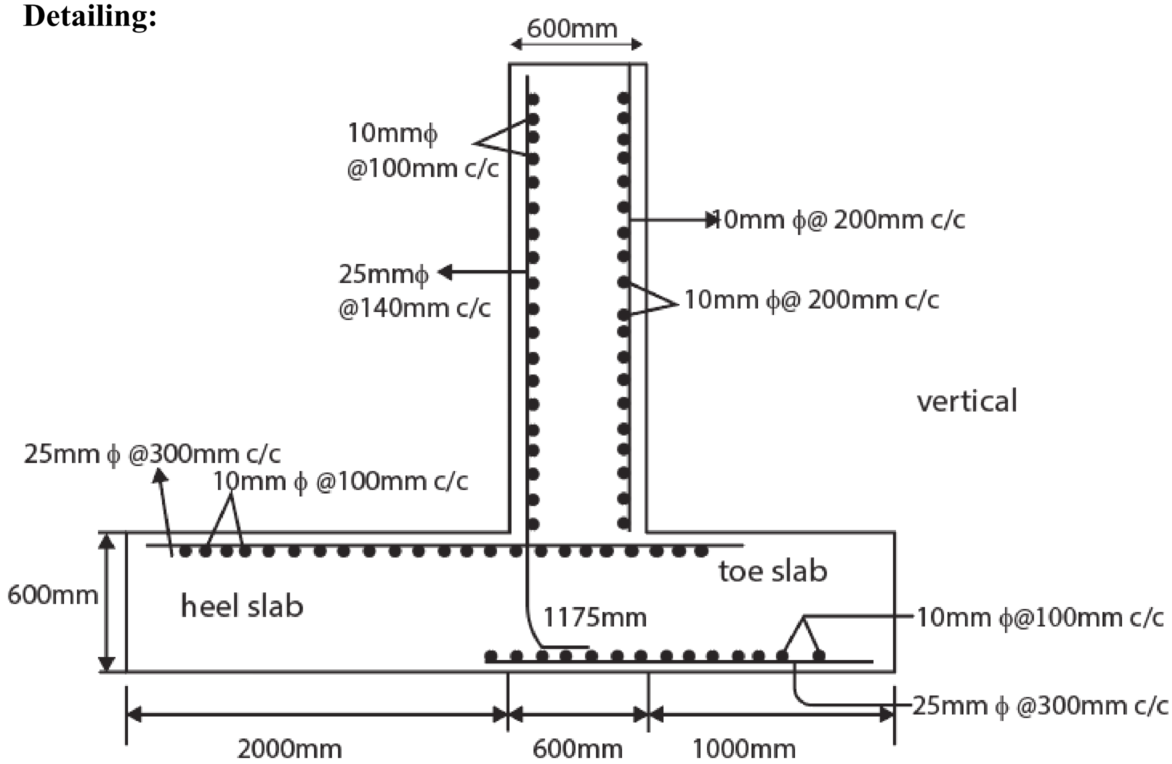
Spacing:

$$S = 1000 \frac{a_{st}}{A_{st}} = 1000 \times \frac{\frac{\pi}{4} \times 25^2}{1226} = 400.2 \text{ mm} > 300 \text{ mm}$$

∴ Not safe

Provided 25 mm ϕ @ 300 mm c/c

Detailing:



07. (a) Write in brief the principles of Dragline and Clamshell used as excavation equipments, the detail of their components and neat sketches showing their parts. How both the equipments can be compared? (20 M)

Sol:

Principle, details of components & sketches showing parts of Dragline used as excavation Equipment:

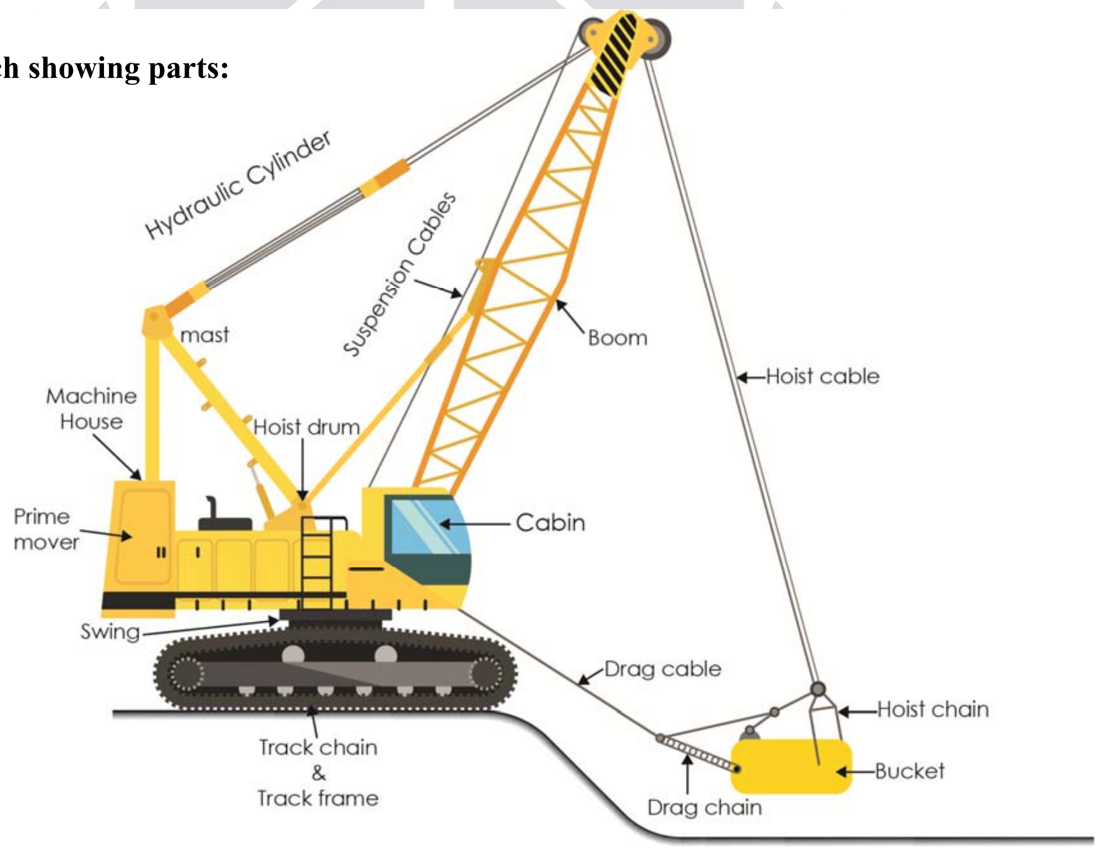
Drag line falls in heavy earth moving equipment's category, used for civil engineering. Projects and surface mining. It has the ability to excavate very deep down the earth. It has the ability to drag material at far distance from the machine.

Drag line is designed to excavate below the level of its base. It does not have to go into a pit or hole in order to excavate. This is advantageous when earth is removed from a ditch,canal or pit containing water. It is found in the excavation for canals and depositing on the embankment without hauling units.

Drag line is a pull type excavator, which excavation direction is back towards machine and below its base level at ground. During the excavation, it works, under the help of inertia force to throw out the bucket with the far distance, so its excavate width and depth all very big. i.e., operation of dragging the bucket against the material to be dug. It has a long light crane boom and the bucket is loosely attached to the boom through the cables. Dragline has long reaches; suited to drag soft material, below its track level particular by under water.

The basic parts of a drag line includes
(i) Boom
(ii) Hoist cable
(iii) Drag cable
(iv) Hoist chain
(v) Drag chain and
(vi) Bucket
(vii) Track frame, driving prime mover

Drag line sketch showing parts:



Details of components/ parts:

Bucket: A suitable size of bucket with tooth is attached with drag line boom (it is truss like structure) Bucket is controlled with the help of number of ropes and chains.

Ropes: There are two separate function rope are attached with bucket. These are:

(i) Hoist rope (ii) Drag rope

(i) Hoist rope is controlled with the help of hoist motor (prime mover) and it supports the hoist-coupler assembly (i.e. part attaching to chain & ropes) and the bucket from the boom.

(ii) Drag rope is used to drawing of bucket horizontally towards the machine.

Drag line drops the bucket on the source material and then drag it horizontally. Bucket start tilting as it came closer to the machine.

Hoist rope is powered by large diesel (or) Electric motors supports the bucket and couplers from the boom.

Boom: Boom is made as truss frame, bottom end attached to machine body boom heavy pin and other long end hoist lifting pulley, acts as heavy structure. It is a long in length to dispose of the soils in one operation along the embankment (or) the pit. This eliminates the need for hauling equipment, thus reduces cost of handling the material.

Track (or) Propeller mechanism:

Three types mostly

(i) Crawler - mounted

(ii) Wheel – mounted

(iii) Truck – mounted

Crawler-mounted drag lines can operate over a soft ground conditions that would support wheel (or) truck mounted equipment. Travel speed of crawler type drag line is very slow (less than 2 KMPH) and necessary to use auxiliary hauling Equipment to transport the equipment from one job site to another job site.

Wheel mounted unit may have travel speed of 45 KMPH. The output of drag line is measured in cubic meters per hour.

Most is a vertical heavy frame structure which supports all the parts of the machine to handle material.

Clamshell: It is used to handle loose material such as sand, gravel and crushed stone, specially suited for lifting material vertically. It performs excavation & crane operations. A hinged bucket on a crane boom used for vertical excavating at, above and below the ground level.

Clam dam shall bucket consists of two scoops hinged together to work like the shall of a clam.

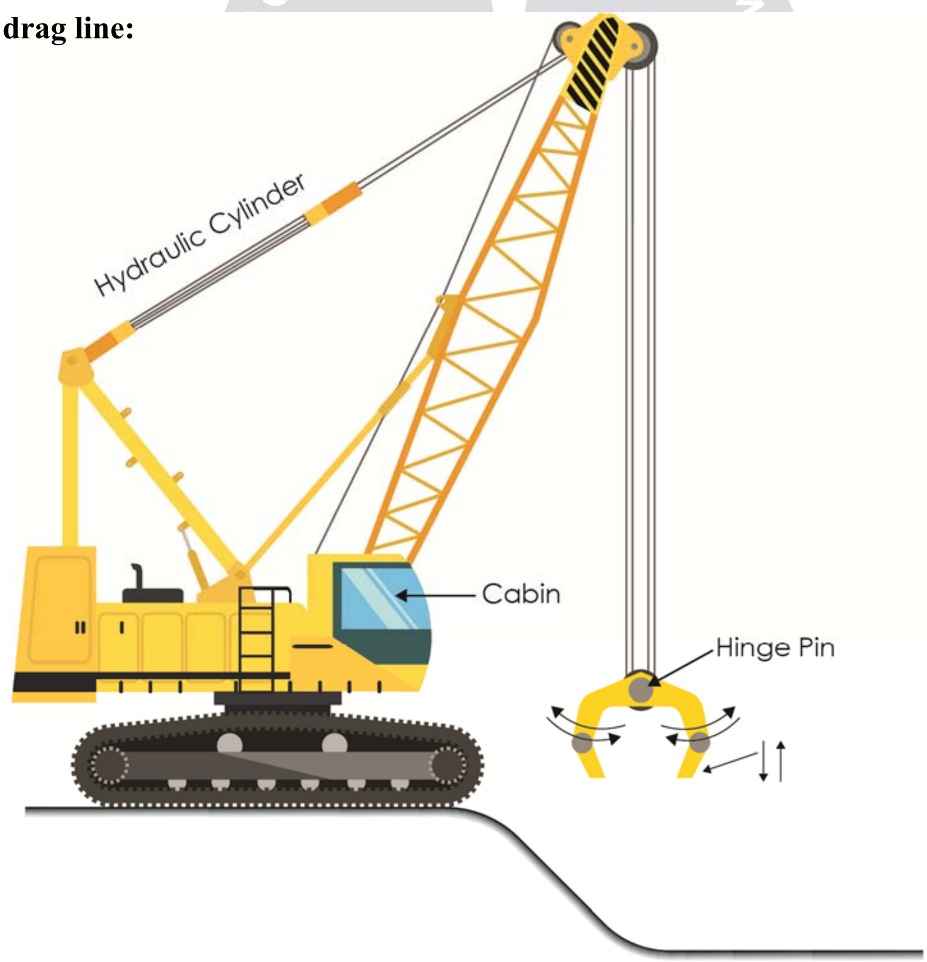
Clam shell is resemblance of its bucket to a clam which is like a shell-fish with hinged double shell. The front end is essentially a crane boom with a specially designed bucket loosely attached at the end through cables as in a DRAGLINE. The capacity of a clam shell bucket is in cubic meters.

Clam shell used for removing material from coffer dam, sewer main holes, well foundation etc.

Details of **components /parts of a clam shell:**

The main parts of clam shell bucket are the closing line, hoist line, sheaves, brackets, tag line, shell and hinges.

Sketch of drag line:



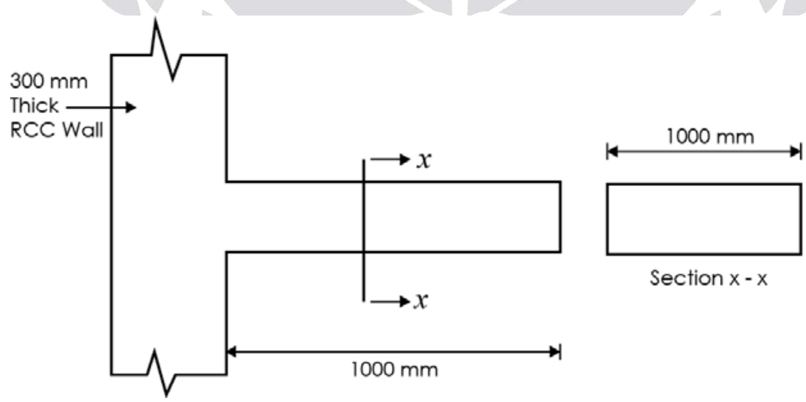
It is used with a crane for vertical digging below the ground level and placing material at considerable height, depth and distance. Also used for moving buck materials from stock piles to plant bins, loading hoppers and conveyors.

Bucket open and closed with hydraulic cylinders. It is a rigid framed structure with almost all components similar to a drag line components except boom structure and bucket & its operation.

Comparison between Dragline and Clam shell:

S.No	Item of comparison	Drag line	Clam Shell
1	Excavation in hard soil (or) rock	Not good	Poor
2	Loading efficiency	Moderately good	Precisely good
3.	Cycle time	More	Relatively less
4.	Digging level	Digs below footing level	Digs at (or) below footing level
5.	Lifting capacity	3 to 30 T	20T to 40T
6.	Excavation depth	More	Less
7.	Work levels	Below base of track	Any level
8.	Operation	Pull shovel	Lift shovel
9	Bucket	Single unit	Two halves

07. (b)



Design a cantilever slab shown in figure for flexure only. Sketch the reinforcement also. Following parameters may be used for design, applying different checks and detailing the reinforcement :

1. Span to effective depth : 10 (maximum)

2. Mild exposure condition : Nominal concrete cover 20.0 mm

3. 2.0 Hours of fire resistance : Nominal concrete cover 25.00 mm

4. Maximum live load : 30 kN/m^2

5. Load combination: 1.5 × Dead load + 1.5 × live load

6. Effective length: length to the face of support plus half the effective depth

7. Grade of concrete M-20

8. Grade of reinforcing steel Fe-415

9. Unit weight of RCC: 25 kN/m^3

10. Development length in Tension : 48 × diameter of reinforcing bar

11. Development length in compression : 37 × diameter of bar

12. Minimum reinforcement : 0.12% of total cross-sectional area

13. Maximum spacing of main reinforcement: 3 × effective depth

14. Maximum spacing of distribution reinforcement: 5 × effective depth

15. Diameter of main reinforcing bar : 10 mm.

16. $(M_{u,\ lim}/f_{ck}\ bd^2) = 0.138$.

17. $(p_{t,\ lim}\ f_y/f_{ck}) = 19.82$.

(20 M)

Sol: Given:

Span = 1 m

Maximum live load = 30 kN/m^2

Depth calculation:

For serviceability criteria :

$$\frac{\ell}{d} \geq 10$$

$$d \geq \frac{1000}{10} = 100 \text{ mm}$$

Say effective depth = 150 mm

Nominal concrete cover = 25 mm (Maximum 2 & 3)

Given, 10 mm dia bars

Total depth, D = 150 + 25 + 10/2 = 180 mm

Assume, total depth of D = 200 mm

Assume, effective depth = 170 mm

l_e = effective span of slab $l_e = \ell_c + \frac{d}{2}$

$$= 1000 + 170/2 = 1085 \text{ mm} = 1.085 \text{ m}$$

Load Calculations:

Self weight of slab (DL) = 25 × 1 × 0.20

= 5 kN/m

Live load (LL) = 30 × 1 (Assume width of slab = 1m)

= 30 kN/m

Total load (w) = DL + LL

= 35 kN/m

Ultimate load (W_u) = 1.5 W = 52.5 kN/m

Ultimate maximum B.M (M_u) = $W_u \frac{\ell_e^2}{2}$

$$= 52.5 \times \frac{1.085^2}{2} = 30.90 \text{ kN.m/m}$$

Check for depth in flexure:

$$d = \sqrt{\frac{30.90 \times 10^6}{0.138 \times 20 \times 1000}} = 105.80 \text{ mm} < 170 \text{ mm provided}$$

Hence safe

Reinforcements:

Main Reinforcement:

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 30.9 \times 10^6}{20 \times 1000 \times 170^2}}\right] \times 1000 \times 170$$

= 573.4 mm^2/m

$$A_{st\ min} = \frac{0.12}{100} \times 1000 \times 200 = 240\,mm^2 < 573.4\,mm^2/m$$

Spacing of 10 mm bars = $\frac{1000}{573.4} \times 78.5 = 136.90$

= 137 mm < 3d [OK]

Hence providing main bars of 10 mm ϕ @ 130 c/c

Spacing of distribution steel (8 mm dia) $= \frac{1000}{240} \times 50.26 = 209.14$ mm

Distribution reinforcement of 8 mm dia @ 200 c/c (< 5d)

Anchorage length @ support (l_o)

Development length required in tension

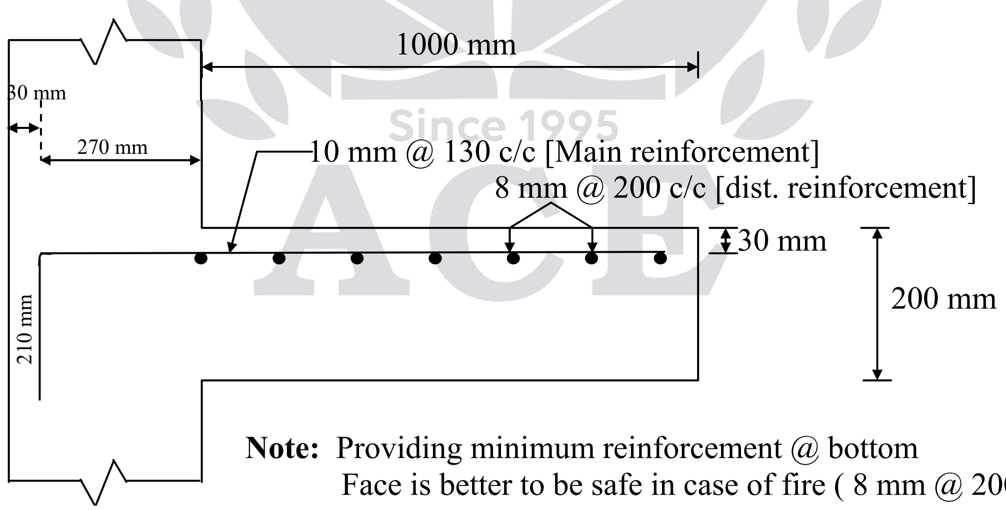
$L_{d\ tension} = 48\ \phi_{main} = 48 \times 10$

= 480 mm

Straight length available with in wall leaving cover = 300 – 30 = 270 mm

Hence, additional anchorage of 210 mm in vertical direction

Detailing:



Note: Providing minimum reinforcement @ bottom Face is better to be safe in case of fire (8 mm @ 200 c/c)

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07. (c) A laced column consisting of two ISMC 300 channels placed back-to-back is subjected to factored axial load of 1100 kN. The 10 m long column is restrained in translation but not in rotation at ends. Single lacing at 45^o is provided, and connected to flanges by bolts. Verify the capacity of the selected section and determine their spacing. Also determine the size of 50 mm wide lacing rods considering only compressive force in them. Take f_y = 250 MPa and gauge length of lacing rods = 50 mm. The properties of ISMC 300 are as follows: A = 4630 mm^2, B = 90 mm, t = 7.8 mm, T = 13.6 mm, α = 96^o, C_Y = 23.5 mm, I_{xx} = 6.42 × 10^7 mm^4, I_{yy} = 3.13 × 10^6 mm^4, r_x = 118.0 mm, r_y = 26.0 mm. Do not design the connection of lacing rod to the channel member. Table 9(c) of IS: 800 is enclosed for reference.

(20 M)

Sol: Factored axial compressive load P = 1100 kN = 1100×10^3 N

Gross sectional area of built up column A = 2×4630 = 9260 mm^2

Unsupported length of built -up column L=10m

Effective length of built -up column KL=10m

Let 'S' be the back to back spacing between channels.

The sections are so placed that the least radius of gyration of built up column becomes as large value as possible. Therefore the radius of gyration about yy axis increased so that it becomes greater than equal to radius of gyration about xx axis and to have most efficient built up column section, the radius of gyration about YY axis should be same as radius of gyration about XX axis

$$r_{YY} = r_{XX} \Rightarrow I_{YY} = I_{XX}$$

Moment of inertia of built up column about XX axis

$I_{XX} = 2\times I_{xx} = 2 \times 6.42 \times 10^7 = 12.84 \times 10^7$ mm^4

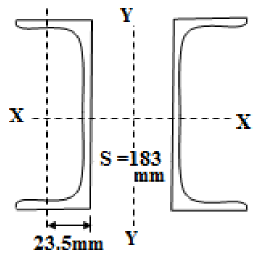
Moment of inertia of built up column about YY axis

$$I_{YY} = 2\times\left[3.13\times10^6 + 4630\times\left(\frac{S}{2}+23.5\right)^2\right]$$

Equating $I_{YY} = I_{ZZ}$

$$2\times\left[3.13\times10^6 + 4630\times\left(\frac{S}{2}+23.5\right)^2\right] = 12.84\times10^7$$

S = 182.69mm ≈ 183mm



Moment of inertia of built up column about YY axis

$$I_{YY} = 2\times\left[3.13\times10^7 + 4630\times\left(\frac{183}{2}+23.5\right)^2\right] = 12.87\times10^7\ \text{mm}^4$$

Minimum moment of inertia of built-up column

$$I_{min} = 12.84 \times 10^7\ \text{mm}^4$$

Minimum radius of gyration of built-up column $r_{min} = \sqrt{\frac{I_{min}}{A_e}} = \sqrt{\frac{12.84\times10^7}{2\times4630}} = 118\,mm$

Effective slenderness ratio of laced built-up column

$$\frac{KL}{r_{min}} = \frac{10000}{118} = 84.75$$

Increase effective slenderness ratio of laced built-up column by 5% as per IS800:2007

$$1.05\times84.75 = 88.98$$

Design compressive strength of built up column $P_d = f_{cd} \times A_e$

Design stress in axial compression of built-up column as per table 9(c) of IS800

$$f_{cd} = 136 - \frac{(136-121)}{(90-80)}\times(88.98-80)$$

$$= 122.53\ \text{N/mm}^2$$

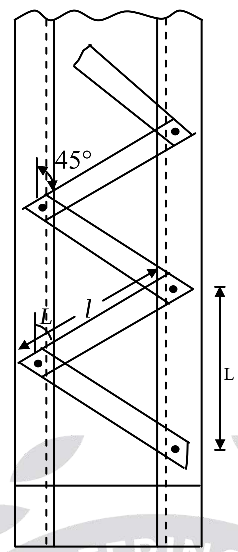
Design compressive strength of built-up column $P_d = f_{cd} \times A_e$

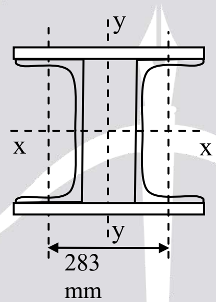
$$P_d = 122.53\times(2\times4630)$$

$$= 1134.62\times10^3\ \text{N} = 1134.62\ \text{kN} \geq 1100\ \text{kN}$$

Hence laced built-up column is safe against buckling

Angle of inclination of single lacing with longitudinal axis $\theta = 45^o$





Transverse shear force V = 2.5 % Factored axial compressive load

$$V = \frac{2.5 \times 1100}{100} = 27.5kN$$

Design axial compressive load in single lacing bar F

$$F = \frac{V}{2\sin\theta} = \frac{27.5}{2\sin 45^0} = 19.45kN$$

Center to center distance of connection = 183+50+50 = 283 mm

Let l = length of flat lacing bar

$$\sin 45^o = \frac{283}{l}$$

l = 400.22 mm

Effective length of flat lacing bar $1.0\,l$ =400.22mm

Minimum thickness of single flat lacing bar t $\nless \frac{l}{40} = \frac{400.22}{40}$ = 10.00 mm

Adopt flat lacing bar 50× 10 mm and assume flat lacing bar is connected flange of channel using one bolt.

Effective slenderness ratio of flat lacing bar $= \frac{1.0 \times l}{t/\sqrt{12}} = \frac{400.22}{10/\sqrt{12}} = 138.64 \leq 145$

which is safe

For slenderness ratio of flat lacing bar = 138.64

Design stress in axial compression of lacing bar as per table 9(c) of IS800

$$f_{cd} = 74.3 - \frac{(74.3 - 66.2)}{(140 - 130)} \times (138.64 - 130) = 67.30 \text{ N/mm}^2$$

Design compressive strength of lacing bar $P_d = f_{cd} \times A_e$

$P_d = 67.3 \times (50 \times 10)$

$= 33.65 \times 10^3$ N= 33.65 kN ≥ F=19.45 kN

Hence lacing bar is safe against compressive force.

08. (a)

(i) What is a work breakdown structure in Construction Project Management? Define and explain in brief. Further, how Work Breakdown Structure is classified into different levels for making the job convenient? Explain with an example. (12 M)

Sol:

Work Breakdown Structures (WBS)

Overview:

The WBS assists project leaders, participants, and stakeholders in the development of a clear vision of the end products or outcomes to be produced by the project.

It provides the framework for all deliverables throughout the project life cycle.

Design:

The WBS provides a graphical representation or textual outline of the project scope.

Some of the main roles of WBS are

Decomposes: the overall project scope into clearly defined deliverables.

Defines: the scope of the project in terms that the stakeholders can understand.

Provides: a structure for organizing information regarding the project's progress, status, and performance.

Supports: tracking of risks to assist the project manager in identifying and implementing necessary responses.

Levels:

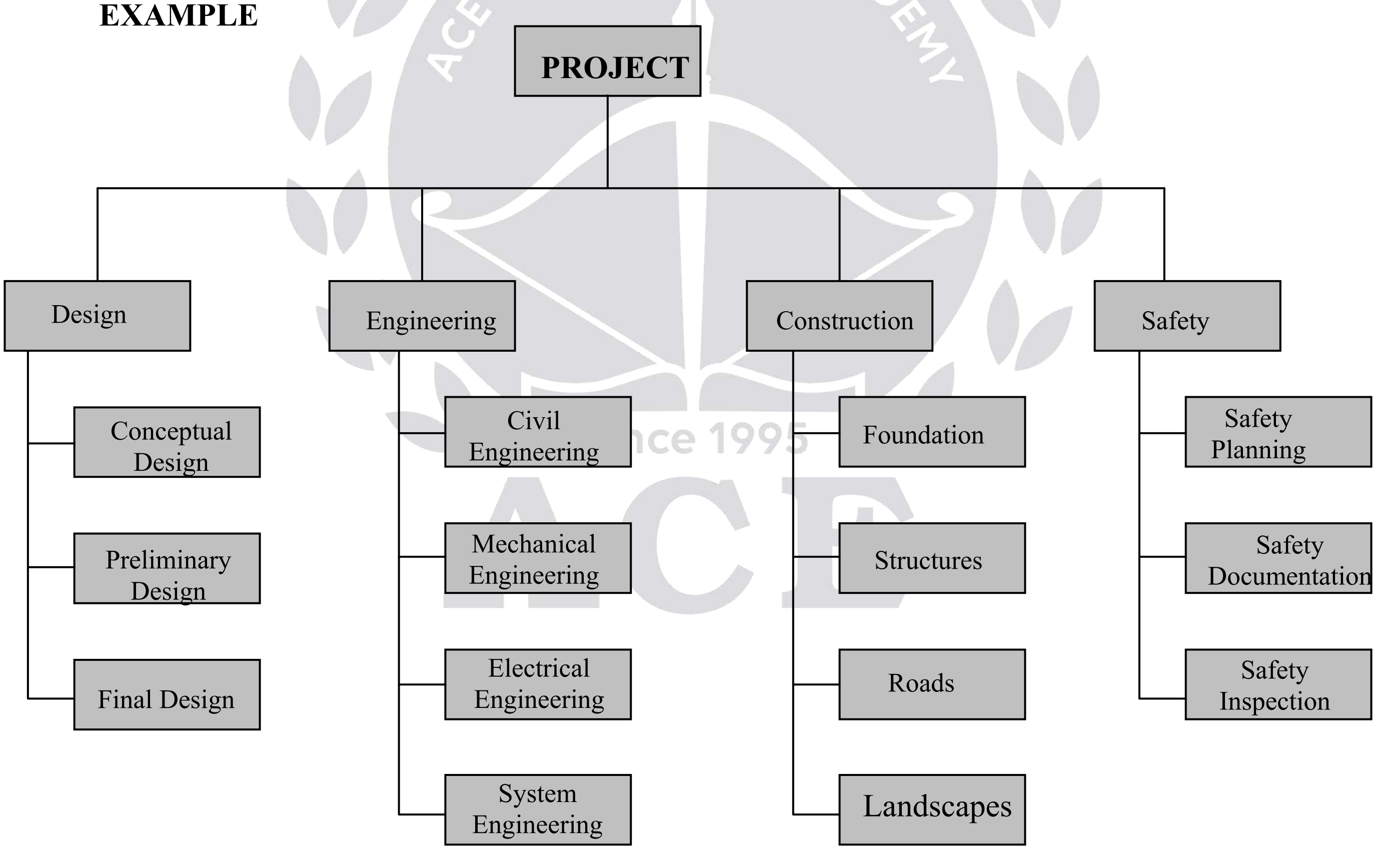
The depth of the WBS is dependent upon the size and complexity of the project and the level of detail needed to plan and manage it.

The 100% Rule:

This rule states that the WBS includes 100% of the work defined by the project scope and captures all work deliverables to be completed, including project management.

The rule applies to all levels within the hierarchy.

EXAMPLE



As you can see, the first level describes the product we want to achieve once the project is complete.

By meeting with the stakeholders, the team can start decomposing the product into smaller, more manageable components.

If we take the component called construction for example, we may want to break it down into several pieces. However, the team has to decide how much detail the work really requires.

While developing a WBS:

1. The WBS is created with the help of the team.
2. The first level is completed before the project is broken down further.
3. Each level of the WBS is a smaller piece of the level above.
4. The WBS includes only deliverables that are really needed.
5. Deliverables not included in the WBS are not part of the project.

08. (a)

(ii) What is Resource Levelling in Construction Project Management and how it is different than Resource Loading? (8 M)

Resource Leveling: Resource leveling generally breaks things down into two categories: time and available resources. Some projects need to be finished within a certain time frame. These projects will use all the available resources (money and manpower) to complete the project by a certain date.

Projects that aren't as pressing can be spread out for an indefinite period of time until resources do become available. These projects are usually ones that are not on the critical path and will not affect the project completion date.

Resource loading: It mainly involves manpower or employees. In resource loading, each employee is assigned a task or a percentage of a project (X percent of the whole). Usually, it's 25 percent of the whole. Then the employee is assigned other tasks until he or she reaches 100 percent booked. This would then mean that the employees cannot take on any additional work.

With resource loading, a project manager can predict an employee's hours for the year and see how tasks can be assigned. This also allows the project manager to decide whether or not additional employees or contractors are needed to complete the scheduled projects.

The downside to resource loading is that employees cannot be 100 percent booked. Other things may arise to take away their time, such as unexpected problems that need to be fixed. An employee should always be under 100 percent booked. Resource loading increases the chance that a project will not be completed on time because employees are overloaded with projects.

Difference between Resource leveling and Resource loading :

- The main difference between resource allocation, also referred to as resource loading, and resource leveling is that resource loading is the process of allocating resources to planned project activities, while resource leveling is mainly used to relate project requirements with available resources. The leveling process ensures that demand for resources does not exceed available resources at a particular time given the interdependent nature of most project activities
- While resource loading mainly deals with manpower, resource leveling deals with both time (project starting and ending date) and resources, including manpower and budget. Resource leveling tries to balance the conflicting interests of projects with the available resources. Like resource loading, resource leveling also has its problems. It is hard to determine in the beginning which tasks will be on the critical path. Also, delaying a task could cause the entire project to fall behind schedule.
- Resource loading is usually based on an educated guess and that the required estimates usually turn out to be either overstated or understated. Once it turns out that the allocated resources are in excess (slack), businesses can reassign the excess resources to areas that need them, which is known as resource leveling with slack. In other scenarios, the business may lack enough resources to complete the project on time despite utilizing slack and reallocating resources, in which case the business must postpone some project activities by extending the project deadline. Once some activities have been completed and their resources are free, the business can then use the freed resources to complete the remaining part of the project. In cases where the project deadline cannot be extended and the business does not have enough resources, it may be forced to borrow money to expand the resources.

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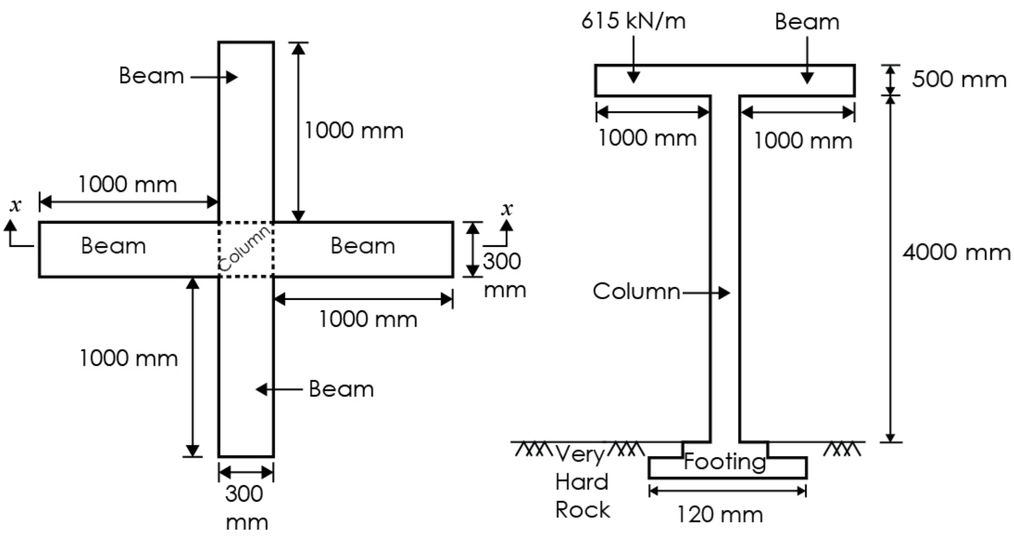
08. (b)



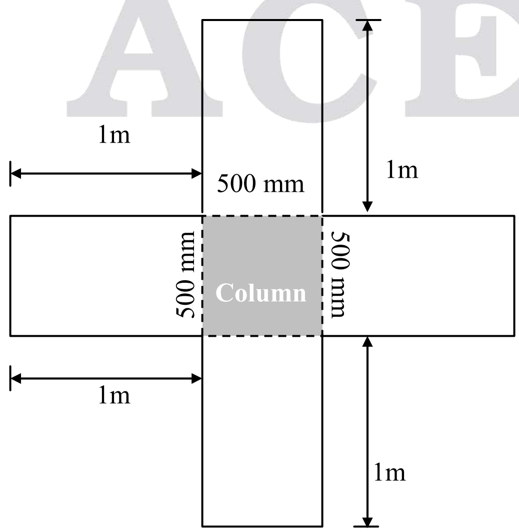
Top Plan   Sectional elevation x - x

A square column (500 mm × 500 mm) carries load from two beams, which are mutually perpendicular as shown in figure. Overhang portion of beams carry a total load of 615.0 kN/m (include self-weight). Design the column at top of footing level. Footing is fully embedded in very hard rock. Beams are restrained against rotation at Beam-column junction. The minimum eccentricity is less than 0.05 times the lateral dimension of column. Sketch all details required at column cross-section.

Use: M-20 grade concrete, Fe-415 grade reinforcing bars, Appropriate co-efficient form 1.0/1.20/1.50/2.0, Main reinforcing bar : 32 mm diameter.

(20 M)

Sol:



Total load on column due to over hanging beams

$= 615\times4 = 2460$ kN

Self weight of column = 25 [(4+0.5) (0.5×0.5)]

$= 28.125$ kN

Total load on column = 2460 + 28.125

$= 2488.125$ kN

Factored load = 1.5×2488.125

$= 3732.18$ kN

$A_g = 500\times500$

$P_u = 0.4\ f_{ck}.\ A_c + 0.67\ f_y.\ A_{sc}$

$3732.18\times10^3 = 0.4\times20\times(500\times500 - A_{sc}) + 0.67\ (415)\times A_{sc}$

$3732.18\times10^3 = 2\times10^6 - 8\ A_{sc} + 278.05\ A_{sc}$

$1732180 = 270.05\ A_{sc}$

$A_{sc} = 6599\ mm^2$

Area of 32 mm bar $= \frac{\pi}{4}\times32^2 = 803.8\ mm^2$

No. of 32 mm bars required $= \frac{6599}{803.8} = 8.2$

How ever provide 10 bars of 32 dia

Design of lateral ties:

Diameter of lateral ties

$\not< \frac{1}{4}$ (main bar dia) = $\frac{1}{4}(32)$ = 8 mm

$\not<$ 6 mm

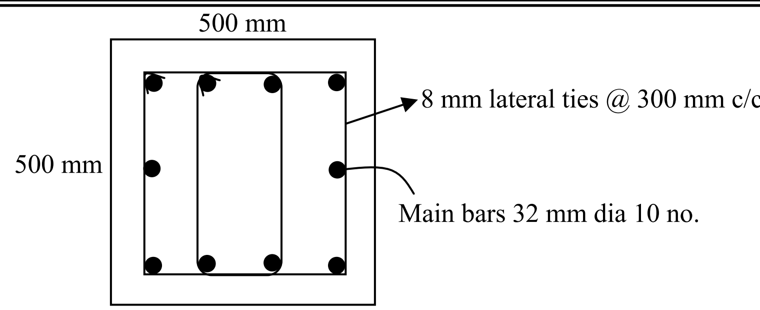
$\Rightarrow$ use 8 mm lateral ties.

Spacing of lateral ties :

$\ngtr$ b = 500 mm

$\ngtr$ 16×32 = 512 mm

$\ngtr$ 300 mm



Details of column cross section

⇒ Use 8 mm dia lateral ties at 300 mm c/c

08. (c) Determine the ultimate bending moments and forces due to vertical and horizontal loads that act on a simply-supported gantry girder. Use the following data :

1. Simply supported span = 6 m

2. Distance between crane wheels = 3.6 m

3. Self-weight of girder = 1.5 kN/m

4. Maximum crane wheel load (static) = 220 kN

5. Weight of crab/trolley = 60 kN

6. Maximum hook load = 200 kN

Take impact factor of 25% and assume double flanged wheels e = 0.15 m while the girder depth, *D* = 0.60 m. (20 M)

Sol: Simply supported span L= 6m

Distance between crane wheels c = 3.6m

Self weight of girder = 1.5 kN/m

Maximum crane wheel load (static) = 220 kN

Weight of crab/trolley = 60 kN

Maximum hook load = 200 kN

1. Moments and forces due to self weight

Factored self weight $W_d = 1.5 \times 1.5 \times 6 = 13.5$ kN

Ultimate mid span bending moment $M_1 = \frac{W_d L}{8} = \frac{13.5 \times 6}{8} = 10.125 kN - m$

Ultimate reaction $R_{A1} = R_{B1} = \frac{W_d}{2} = \frac{13.5}{2} = 6.75 kN$

2. Moments and forces due to vertical wheel load including load factor and 25% impact

   W_c=1.5×1.25×220 = 412.5 kN

   Ultimate maximum bending moment under wheel load (case 1)

$$=\frac{2W_c}{L}\left(\frac{L}{2}-\frac{c}{4}\right)^2=\frac{2\times 412.5}{6}\left(\frac{6}{2}-\frac{3.6}{4}\right)^2=606.375kN$$

   Ultimate maximum bending moment under wheel load (case 2)

$$M_1=\frac{W_cL}{4}=\frac{412.5\times 6}{4}=618.75kN-m$$

   Maximum ultimate bending moment M_1 = 618.75 kN-m

   Ultimate reaction $R_{A2}=W_c\left(2-\frac{c}{L}\right)=412.5\times\left(2-\frac{3.6}{6}\right)=577.5kN$

3. Moments and forces due to horizontal wheel loads horizontal surge load including load factor

   W_{hc}=1.5×0.1×(200+60)= 39 kN

   This is divided among the 4 wheels double flanged wheels

   Horizontal wheel load W_{hc} = 39/4 = 9.75 kN

   Using calculations similar to those for vertical moments and forces,

   Ultimate horizontal bending moment (case 2)

$$=\frac{2W_{hc}L}{4}=\frac{9.75\times 6}{4}=14.625kN-m$$

   Ultimate horizontal bending moment (case 1)

$$=\frac{2W_{hc}}{L}\left(\frac{L}{2}-\frac{c}{4}\right)^2=\frac{2\times 9.75}{6}\left(\frac{6}{2}-\frac{3.6}{4}\right)^2=14.33kN-m$$

4. Bending moments and reaction force due to drag force

   (e=0.15m and depth of girder is 0.6m)

$$R_{A3}=\frac{W_g e}{L}=\frac{1.5\times(0.05\times 220\times 1.25)(0.3+0.15)}{6}=1.55kN$$

   Ultimate bending moment due to drag force

$$M_3=R_{A3}\times\left(\frac{L}{2}-\frac{c}{4}\right)=1.55\times\left(\frac{6}{2}-\frac{3.6}{4}\right)=3.225kN-m$$

   Maximum ultimate design vertical moment

   $M_u = M_1 + M_2 + M_3$ = 10.125+618.75+3.225 = 632.13 kN-m

   Maximum design vertical reaction

   $R_u = R_{A1} + R_{A2} + R_{A3}$ = 6.75+577.5+1.55 = 585.8 kN-m

HEARTY CONGRATULATIONS TO OUR

ESE 2017 RANKERS

TOTAL SELECTIONS 196 | CE 86 | ME 44 | EE 36 | E&T 30