Civil Engineering-I (Subjective)

DETAILS EXPLANATIONS

1. (A) Assumptions :

1. All the load must be applied only at the joints.
2. All the members must be straight and connected by smooth pins.
3. In a truss the total no. of joints 'j' are related by,

$$m = 2j - 3$$

Where

$$m = \text{No. of members}$$
$$j = \text{Number of joints}$$

- For first 3 joints we require 3 members.
- For an additional joint we require 2 members.

(B) Static Indeterminacy for plane truss :

The truss is said to be statically indeterminate when the total number of reactions and member axial forces exceed the total number of static equilibrium equations.

Static Indeterminacy can be calculated by,

$$D_S = (m + r) - 2j$$

Where,

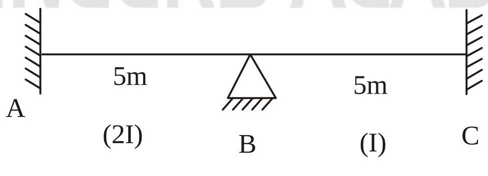
$$m = \text{No. of members in truss}$$
$$r = \text{Unknown reaction}$$
$$j = \text{No. of joints}$$

The indeterminacy in the truss may be external, internal or both. A plane truss is said to be externally indeterminate if the number of reactions exceeds the number of static equilibrium equations available and has exactly (2j – 3) members. A plane truss is said to be internally inderminate if it has exactly 3 reaction components and more than (2j – 3) members. Finally a truss is both internally and externally indeterminate if it has more than three reaction component and also has more than (2j – 3) members.

(C)

$$K_{BA} = \frac{4EI}{L} = \frac{4E(2I)}{5} = \frac{8EI}{5}$$

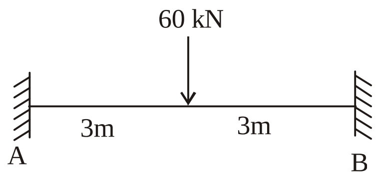
$$K_{BC} = \frac{4EI}{L} = \frac{4EI}{5}$$



$$(DF)_{BA} = \frac{8EI/5}{\frac{8EI}{5}+\frac{4EI}{5}} = \frac{2}{3}$$

$$(DF)_{BC} = \frac{4EI/5}{\frac{8EI}{5}+\frac{4EI}{5}} = \frac{1}{3}$$

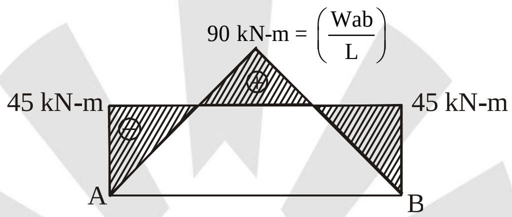
(D)



Fixed-end-moment

$$(FEM)_{AB} = \frac{-wl}{8} = \frac{-(60)(6)}{8} = -45 \text{ kN-m}$$

$$(FEM)_{BA} = \frac{wl}{8} = \frac{60 \times 6}{8} = 45 \text{ kN-m}$$



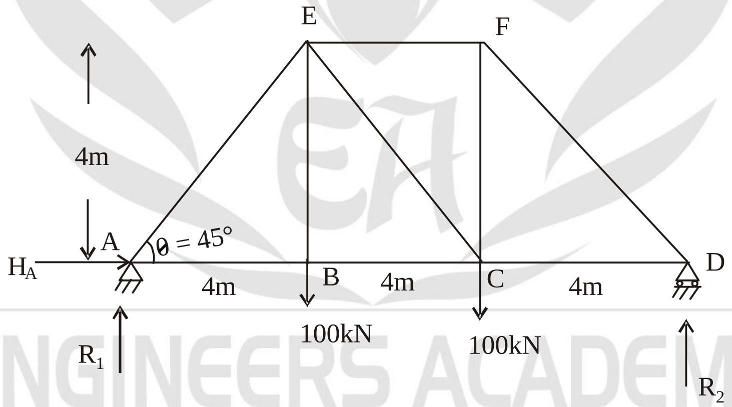
BMD

2. $R_1 + R_2 = 200$ .....(i)

$$\Sigma M_A = 0$$

$$100 \times 4 + 100 \times 8 = R_2 \times 12$$

$$R_2 = 100 \text{ kN}$$



From euqation (i) $R_1 = 100 \text{ kN}$

$$\Sigma M_E = 0$$

$\Rightarrow$ $100 \times 4 - 100 \times 8 + 100 \times 4 + H_A \times 4 = 0$

$\Rightarrow$ $H_A = 0$

At, Joint A $F_{AE}\sin 45° + R_1 = 0$ $(\Sigma f_y = 0)$

$$F_{AE} = -100\sqrt{2} \text{ kN}$$

$$\Sigma f_x = 0$$

$\Rightarrow$ $F_{AE}\cos 45° + F_{AB} + H_A = 0$

$\Rightarrow$ $F_{AB} = -F_{AE}\cos 45°$

$\Rightarrow$ $F_{AB} = 100 \text{ kN}$

At, Joint B $\Sigma f_x = 0 \Rightarrow F_{BA} = F_{BC}$

$\Rightarrow$ $F_{BC} = 100$ kN Ans.

$\Sigma f_y = 0$

$\Rightarrow$ $F_{BE} = 100$ kN

At, Joint E

$\Sigma F_y = 0$

$\Rightarrow$ $F_{AE}\cos 45° + F_{EB} + F_{EC}\cos 45° = 0$

$\Rightarrow$ $-100 + 100 + F_{EC}\cos 45° = 0$

$F_{EC} = 0$ Ans.

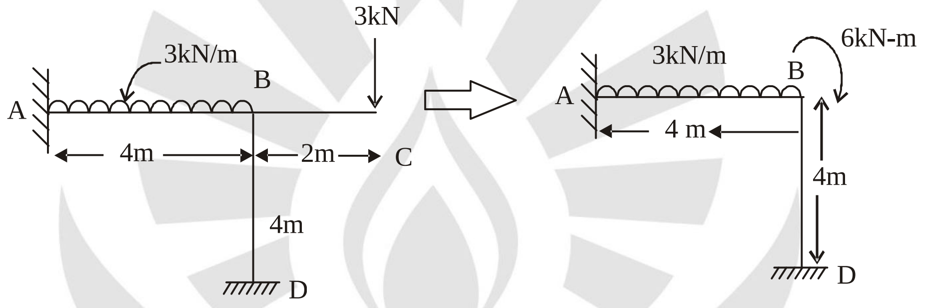
At, Joint C

$\Sigma f_y = 0$

$\Rightarrow$ $F_{CF} + F_{CE}\cos 45° = 100$

$F_{CF} = 100$ kN Ans

3.



Fixed end moment $(FEM)_{AB} = \dfrac{-wl^2}{12} = \dfrac{-(3)(4)^2}{12} = -4$ kN-m

$$(FEM)_{BA} = \frac{-wl^2}{12} = \frac{-(3)(4)^2}{12} = 4 \text{ kN-m}$$

$$(FEM)_{BD} = (FEM)_{DE} = 0$$

Using slope-deflection method,

$$M_{AB} = (FEM)_{AB} + \frac{2EI}{L}\left(2\theta_A + \theta_B - \frac{3\Delta}{L}\right)$$

$\Rightarrow$ [$\therefore$ A is a fixed support $\therefore$ $\theta_A = 0$ and no settlement is given, So, $\Delta = 0$]

$$M_{AB} = -4 + \frac{2EI}{4}[\theta_B] \quad \ldots(1)$$

Similarly,

$$M_{BA} = 4 + \frac{2EI}{4}[2\theta_B] \quad \ldots(2)$$

$$M_{BD} = \frac{2EI}{4}[2\theta_B] \quad \ldots(3)$$

$$M_{DB} = \frac{2EI}{4}[\theta_B] \quad \ldots(4)$$

$\therefore \quad M_{BA} + M_{BD} = 6$

$\Rightarrow \quad 4 + EI\theta_B + EI\theta_B = 6$

$EI\theta_B = 1$

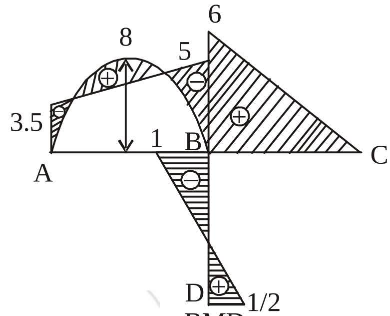
Putting value of $EI\theta_B$ in equation (1), (2), (3) and (4)

$\Rightarrow \quad M_{AB} = \frac{-7}{2}$ kN-m

$\Rightarrow \quad M_{BA} = 5$ kN-m

$\Rightarrow \quad M_{BD} = 1$ kN-m

$\Rightarrow \quad M_{DB} = \frac{1}{2}$ kN-m



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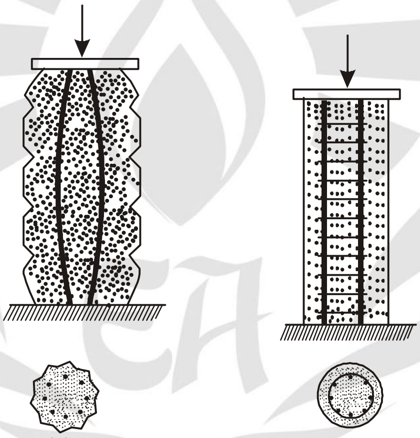
4. **(A)** Figure (a) shows a concrete column reinforced with longitudinal reinforcement without any lateral ties. When load is applied on such a column, the concrete bulges out laterally as shown. The individual bars themselves acts as long slender column and therefore tend to buckle away from the column's axis. Due to this, tension is caused in the outside shell of the concrete which opens out. The failure usually takes place suddenly. In order to check this tendency the longitudinal reinforcement is tied transversely at suitable intervals with the help of ties as shown in Figure (b).


(a) (b)

Hence, these ties may have following functions :

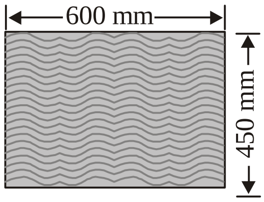
(i) Prevent the premature buckling in individual bars.

(ii) Confine the concrete in 'core', thus improving ductility and strength.

(iii) Hold the longitudinal bars in position during construction and compaction.

(iv) Provide resistance against shear and torsion, if required.

Area of Steel for given column :



Factored axial load, $P_u = 3000$ kN

Least lateral dimensions $D = 450$ mm

Assuming minimum eccentricity equals to 20 mm as the effective length of column is not given

But $0.05\ D = 0.05 \times 450 = 22.5$ mm

Hence $e_{min} < 0.05$ D, therefore given column can be considered as axially loaded column

$$\therefore \quad P_u = 0.4\ f_{ck}A_c + 0.67\ f_yA_{sc}$$

$$P_u = 0.4\ f_{ck}(A_g - A_{sc}) + 0.67\ f_yA_{sc}$$

$$3000 \times 10^3 = 0.4 + 20 \times (600 \times 450 - A_{sc}) + 0.67 \times 415\ A_{sc}$$

$$3000 \times 10^3 = 2160 \times 10^3 - 8\ A_{sc} + 278.05\ A_{sc}$$

$$270.05\ A_{sc} = 840 \times 10^3$$

$$A_{sc} = 3110.53\ mm^2$$

Minimum area of longitudinal reinforcement $= 0.8\%$ of $A_g = \dfrac{0.8}{100} \times 600 \times 450 = 2160 < A_{sc}$

Hence the minimum area of longitudinal reinforcement to be provided is 3110.53 mm^2

(B) Consider the beam to be simply supported with span 6.3 m

$$\text{Maximum shear} = V_m = \frac{wL}{2} = \frac{30 \times 6.3}{2} = 94.5\ kN$$

$$\text{Maximum BM} = \frac{wl^2}{8} = \frac{30 \times 6.3^2}{8} = 148.83\ kN\text{-}m$$

$$\text{Shear stress in beam} = \frac{V_m}{t_w \times h} = \frac{94.5 \times 1000}{7.4 \times 350}$$

$$\tau_{v_{claculated}} = 36.48\ N/mm^2 < 100\ N/mm^2$$

$\therefore$ Beam is safe in shear

$$\text{Bending stress in beam} = \frac{M}{I} \times y = \frac{148.33 \times 10^6}{131.583 \times 10^6} \times \frac{350}{2}$$

$$\sigma_{bc} = 197.93\ N/mm^2 > 165\ N/mm^2$$

$\therefore$ Beam is not safe in bending

5. Super imposed load = 80 kN/m

$$\text{Selfweight} = B \times D \times \gamma_C$$

$$= 0.30 \times 0.70 \times 25 = 5.25\ kN/m\ [\because \gamma_C = 25\ kN/m^3]$$

Total load, $W = 80 + 5.25 = 85.25$ kN/m

Maximum bending moment, $M_{max} = \dfrac{Wl^2}{8} = \dfrac{85.25 \times 6^2}{8} = 383.625$ kN-m

Assuming effective cover = 70 mm

Working Stress Method:

Moment of resistance of balanced section,

$$MR_{bal} = Q.Bd^2$$

where, $Q = \dfrac{1}{2}\sigma_{cbc} \cdot j \cdot k$

σ_{cbc} = permissible stress in concrete in bending compression

$= 7\ N/mm^2$ [for M20]

$\therefore$ Modular ratio (m) $= \dfrac{280}{3\sigma_{cbc}} = \dfrac{280}{3\times 7} = 13$

k_{bal} = coefficient of critical depth

$$= \frac{m\cdot\sigma_{cbc}}{t + m\cdot\sigma_{cbc}} = \frac{13\times 7}{230+13\times 7} = 0.2835$$

j = coefficient of lever arm

$$= 1 - \frac{k_{bal}}{3} = 1 - \frac{0.2835}{3} = 0.9055$$

$$Q = \frac{1}{2}\sigma_{cbc}\cdot j\cdot k_{bal} = \frac{1}{2}\times 7\times 0.9055\times 0.2835 \simeq 0.90$$

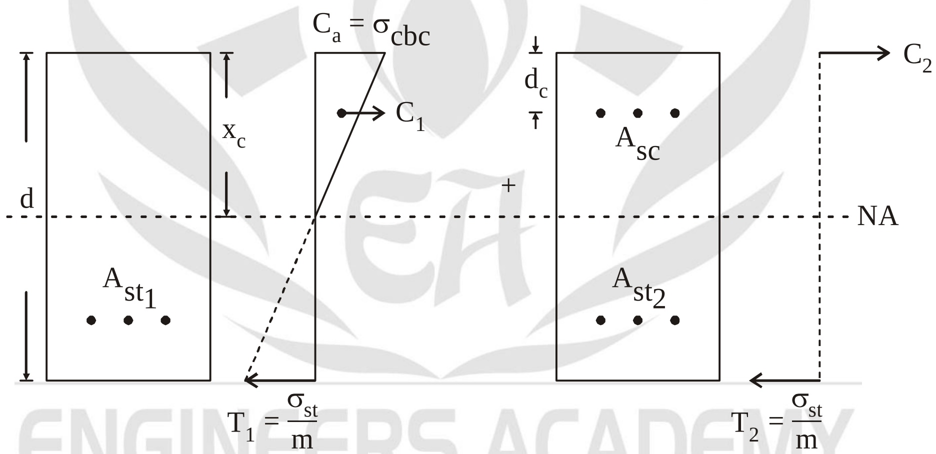
$$MR_{bal} = QBd^2 = 0.90 \times 300 \times 630^2 \times \frac{1}{10^6} = 107.163 \text{ kNm}$$

Here $M_{max} > MR_{bal}$

So a doubly reinforced section will be required

Say, $MR_{bal} = MR_1 = 107.163$ kN-m

$MR_2 = 383.625 - MR_1 = 276.462$ kN-m



$$A_{st_1} = \frac{MR_1}{\sigma_{st}\left(d - \frac{x_c}{3}\right)} = \frac{MR_1}{\sigma_{st}\cdot jd}$$

$$= \frac{107.163\times 10^6}{230\times 0.9055\times 630} = 816.74 \text{ mm}^2$$

and $A_{st_2} = \dfrac{MR_2}{\sigma_{st}(d-d_e)}$

Assuming effective cover to compression steel (d_c) = 70 mm

$\therefore$ $A_{st_2} = \dfrac{276.462\times 10^6}{230(630-70)} = 2146.44 \text{ mm}^2$

Total area of tensile steel, $A_{st} = A_{st_1} + A_{st_2} = 816.74 + 2146.44 = 2963.18\ mm^2$

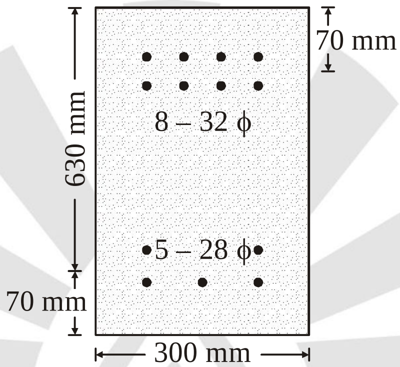
Using 28 mm ϕ bars

$$\text{Number of tensile bars} = \frac{A_{st}}{\frac{\pi}{4}\times 28^2} = \frac{2963.18}{\frac{\pi}{4}\times 28^2} \simeq 5$$

Area of compression steel A_{sc},

$$A_{sc} = \frac{m(d-x_c)}{(1.5m-1)(x_c-d_c)}\times A_{st_2} = \frac{m(d-k_{bal}d)}{(1.5m-1)(k_{bal}d-d_c)}\times A_{st_2}$$

$$= \frac{13\times(630-0.2835\times 630)}{(1.5\times 13-1)(0.2835\times 630-70)}\times 2146.44 = 6270\ mm^2$$



Using 32 mm ϕ bars

$$\text{Number of bars} = \frac{A_{sc}}{\frac{\pi}{4}\times 32^2} = \frac{6270}{\frac{\pi}{4}\times 32^2} \simeq 8$$

Limit State Method :

Dead load, $W_d = 0.3 \times 0.7 \times 25 = 5.25$ kN/m

Live load, $W_L = 80$ kN/m

Total load, $W = 85.25$ kN/m

Factored Load, $W_u = 1.5 \times 85.25 \simeq 128$ kN/m

Maximum factored, $BM = \frac{W_u L^2}{8} = \frac{128\times 6^2}{8} = 576$ kN-m

Let effective cover = 70 mm

$\therefore$ $d = 700 - 70 = 630$ mm

Limiting depth of NA, $x_{ulim} = 0.48\ d$ [For Fe-415]

$= 0.48 \times 630 = 300$ mm (Approx.)

MOR corresponding to balanced section

$M_1 = M_{lim} = Q_B d^2$

where, $Q = 0.36\ f_{ck}.k.(1 - 0.42\ k)$

For M_{20} and Fe_{415} $Q = 0.36 \times 20 \times 0.48(1 - 0.42 \times 0.48) = 2.76$

$\therefore$ $M_1 = 2.76 \times 300 \times 630^2 = 328.6$ kN-m

$$A_{st_1} = \frac{0.36 f_{ck} B x_{u\,lim}}{0.87 f_y} = \frac{0.36 \times 20 \times 300 \times 300}{0.87 \times 415} = 1800 \text{ mm}^2$$

The bending moment = M_2 has to be resisted by a couple consisting of compression steel (A_{sc}) and the corresponding tension steel (A_{st_2})

If $\quad \frac{d_c}{d} = 0.1$

Therefore, Fe415 $\quad f_{sc} = 353 \text{ N/mm}^2$

$$M_2 = (f_{sc}A_{sc} - f_{cc}A_{sc})(d - d_c)$$

$$A_{sc} = \frac{M_2}{(f_{sc} - f_{cc})(d - d_c)}$$

where $\quad f_{cc} = 0.45 f_{ck}$

$$\therefore \quad A_{sc} = \frac{247.4 \times 10^6}{(353 - 0.45 \times 20)(630 - 0.1 \times 630)} = 1268 \text{ mm}^2$$

Corresponding tensile steel A_{st_2},

$$0.87 f_y A_{st_2} = f_{sc} A_{sc}$$

$$A_{st_2} = \frac{353 \times 1268}{0.87 \times 415} = 1240 \text{ mm}^2$$

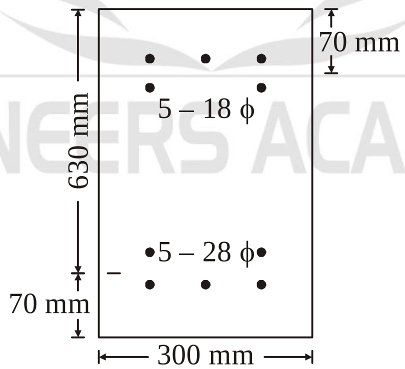
Total tension steel, $\quad A_{st} = A_{st_1} + A_{st_2} = 1800 + 1240 = 3040 \text{ mm}^2$

Using 28ϕ mm bars in tension

$$\text{Number of bars} = \frac{A_{st}}{\frac{\pi}{4} \times 28^2} = \frac{3040}{\frac{\pi}{4} \times 28^2} \simeq 5$$

Using 18ϕ mm bars in compression

$$\text{Number of bars} = \frac{A_{st}}{\frac{\pi}{4} \times 18^2} = \frac{1260}{\frac{\pi}{4} \times 18^2} \simeq 5$$



6. (A) (i) In steel building construction when the web of the plate girder acting alone (i.e. without stiffeners) proves inadequate, stiffeners may be provided for the improvement in buckling strength of slender web.

(ii) Types of tension members

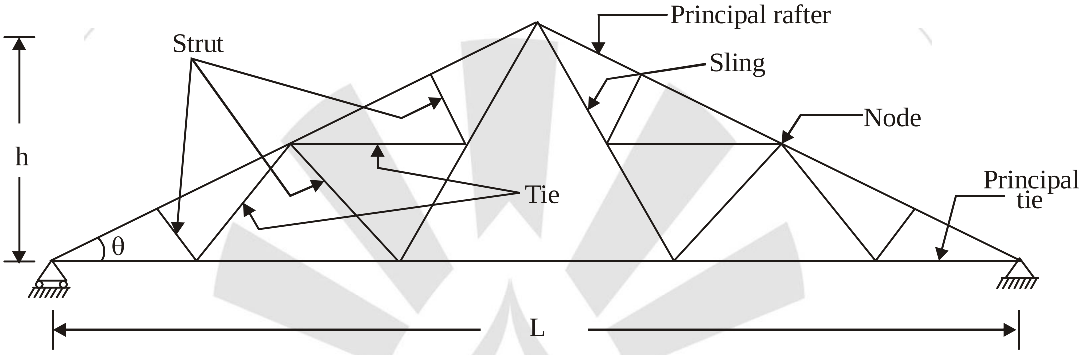
The types of structure and method of end connections determine the type of a tension member in structural steel construction. Tension members used may be broadly grouped into four groups.

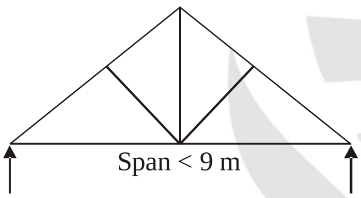
1. Wires and cables
2. Rods and bars
3. Single structural shapes and plates
4. Built-up members

(iii) A steel column can fail in following ways.

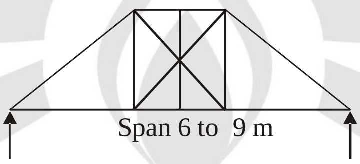
1. Squashing
2. Local buckling
3. Flexural buckling
4. Torsional buckling
5. Flexural torsional buckling

(B)

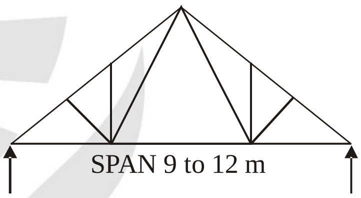




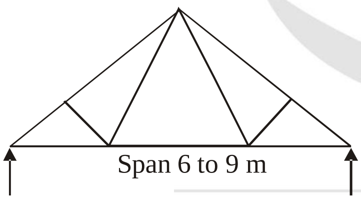
(1) KING POST



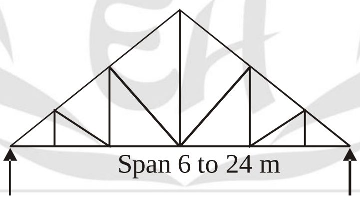
(2) QUEEN POST



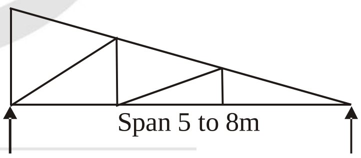
(3) FAN TYPE



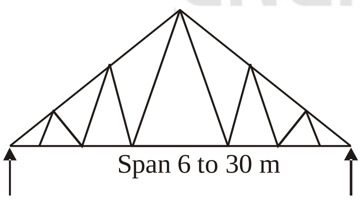
(4) FINK TYPE



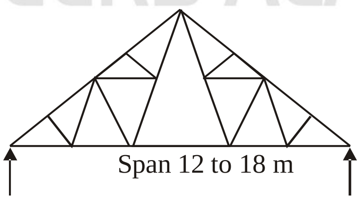
(5) HOWE POST



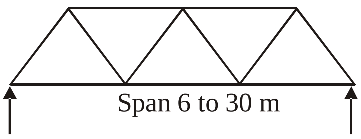
(6) NORTH LIGHT TYPE



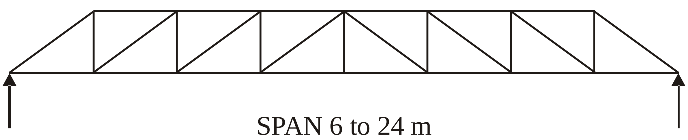
(7) TRIANGULAR



(8) COMPOUND FINK



(9) WARREN TYPE


SPAN 6 to 24 m
(10) HOWE TYPE (FLAT)

7. **(A) (i)** Compressibility is the process of decrease in volume of soil mass due to increase in loading. The volume reduction in the soil may be due to

(a) Compression and expulsion of pore air

(b) Expulsion of pore water

(c) Change in the orientation of molecules

(d) Compression of water and solid molecules

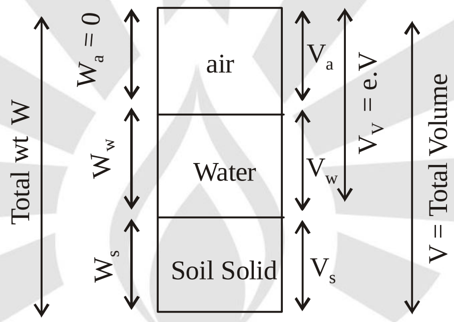
$\frac{\Delta e}{\Delta P}$ is defined as the coefficient of compressibility of the soil mass and is represented by a_v. This is also called coefficient of compression.

$$a_v = \frac{\Delta e}{\Delta P}$$

Where, Δe = Reduction in the void ratio.

ΔP = Additional Stress

(ii) Three-phase diagram :



The diagramatic representation of the different phases in a soil mass is called the phase diagram or block diagram.

A three phase diagram is applicable for a partially saturated soil (0 < S < 1)

Void ratio : $$e = \frac{\text{Vol. of voids}}{\text{Vol. of solid(V)}} \times 100\% = \frac{V_v}{V_s}$$

(iii)
$$D_{60} = 0.71$$
$$D_{10} = 0.18$$
$$D_{30} = 0.34$$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.71}{0.18} = 3.9444$$

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(0.34)^2}{0.71 \times 0.18} = \frac{0.1156}{0.1278} = 0.904538$$

(B) Foundation which is placed at a greater depth or transfer the loads to deep strata is called deep foundation.

- The depth of shallow foundation is generally about '3' meters but the depth of deep foundation greater than shallow foundation.
- Shallow foundation is cheaper but, deep foundation are generally more expensive.

- Shallow foundation are easier to construct but, construction process of deep foundation is more complex.

(i) Classification of Piles based on load transfer mechanism :

(a) End bearing piles or point bearing piles.

(b) Friction piles or Cohesion piles.

(c) Combination of friction and cohesion piles.

(ii) Classification of pile based on method of installation can be classified in several items which is

* **Driven piles :**

(a) Timber (Round or Square section)

(b) Precast concrete (Solid or Hollow section)

(c) Pre-stressed concrete (Solid or Hollow secton)

(d) Steel H-section, box and tube

* **Bored Piles :**

(a) Continuous bored

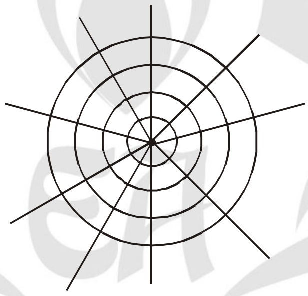
(b) Cable precussion drilling

(c) Augered

(d) Large diameter under-reamed

(e) Drilled in tubes.

8. (A) (i) Newmark's chart :



Newmark's chart

Newmark's influence chart is an illustration used to determine the vertical pressure at any point below a uniformly loaded flexible area of soil of any shape.

(ii) Negative skin friction :

Negative skin friction is a downward drag action on the piles due to relative movement between piles and their surrounding soil. This condition occurs when soil in upper portion is loose/soft where as in lower portion is dense or stiff.

Negative skin friction reduces load carring capacity of pile.

Negative skin friction may develop under following condition :

(a) A cohessive fill is placed over non-cohessive soil layer and a piles is drive into such medium.

(b) Increase in surcharge over surrounding soil.

(c) A non-cohesive fill is placed over a soft cohesive layer and a pile installed in such medium

(d) Lowering of the ground water table.

(e) Disturbance due to dynamic or seismic effect.

(iii) Sand drains :

Sand drains is a process of radial consolidation which increase rate of drainage in the embankment by driving a casing into the embankment and making vertical bore holes.

(B) These three basic types of earth pressure :

(a) Active earth pressure

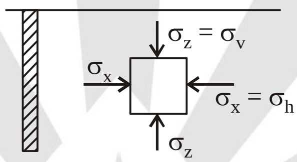
(b) Passive earth pressure

(c) Earth pressure at rest

(a) Active earth pressure :

Consider a soil mass element of a depth 'z' below the ground surface, which is subjected to vertical stress (σ_z) and horizontal stress (σ_x).

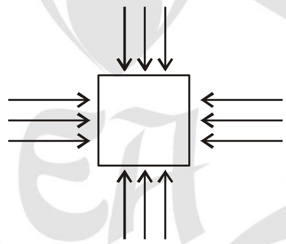
• As there is no shear stress acting on horizontal and vertical plane, σ_x and σ_z are principle stress. Stress



We, know that, σ_2 is major principle stress (σ_1) and σ_x is minor principle stress (σ_3) (from the theory of earth pressure at rest)

• As, wall starts moving away from soil, (σ_x) remains constant but, σ_x will be go on reducing and on verge of shear failure its value will become

$$(\sigma_x)_{active} = \text{Active earth pressure.}$$



$$\sigma_z = \sigma_1 \;;\; \sigma_x = \sigma_3 = \sigma_{active} = \text{active earth pressure}$$

The value of active earth pressure P_a or σ_{active} or σ_3 can be determine in terms of σ_z or σ_1.

We know that, $$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right)+2.C\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

Here, $$\sigma_1 = \sigma_z \times \gamma \times z$$
$$\sigma_3 = P_a$$

$\therefore$ $$\sigma_z = P_a \times\left(\frac{1+\sin\phi}{1-\sin\phi}\right)+2C\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

or $$P_a = \sigma_z \times\left(\frac{1-\sin\phi}{1+\sin\phi}\right)-2.C.\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

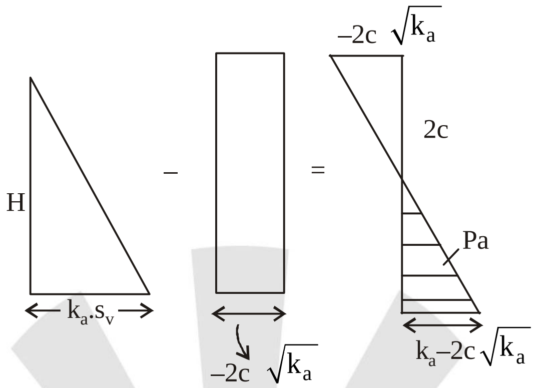
Where, $$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \text{coefficient of active earth pressure.}$$

$$P_a = k_a \times \sigma_z - 2.C.\sqrt{k_a}$$

For, cohesionless soil, $C = 0$

$$P_a = k_a \times \sigma_z$$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45 - \phi/2)$$



Depth of tension crack, $Z_c = \dfrac{2.c'}{\gamma \times \sqrt{k_a}}$

(b) Passive earth pressure :-

In case of movement of wall towards the soil, mass experienes a uniform compression in the horizontal direction. Hence the value of size increases from its origional value.

As the deformation of soil mass goes on increasing, a state comes at which, $\sigma_x = \sigma_z$. But, at the state of failure, i.e. when the soil mass attains plastic equilibrium 'σ_x' becomes greater than σ_z ($\sigma_x > \sigma_z$).

For this condition σ_z, becomes minor principle stress (σ_3) and σ_x becomes major principle stress (σ_1). At time of plastic failure when σ_x is maximum the soil, is said to be in passive Rankine state.

We know that, $$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2.C\sqrt{\left(\frac{1+\sin\phi}{1-\sin\phi}\right)}$$

$$\sigma_1 = P_P$$

$$\sigma_3 = \gamma \times z = \sigma_z$$

$$P_P = \sigma_2\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2.C\sqrt{\left(\frac{1+\sin\phi}{1-\sin\phi}\right)}$$

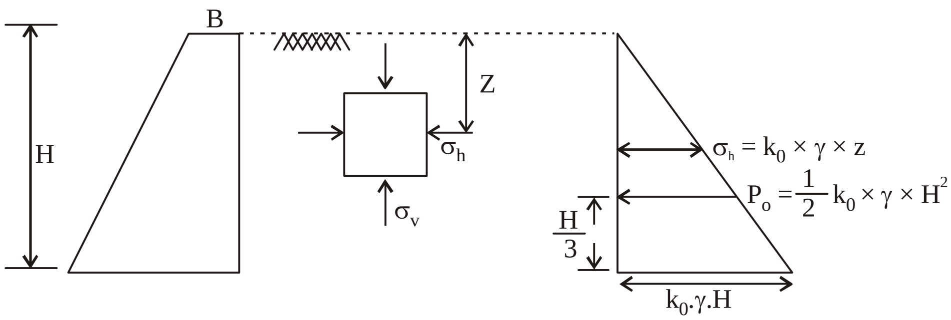
Where $$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

k_p = Coefficient of passive earth pressure.

$\therefore$ For cohesion less soil $C = 0$, $P_P = \sigma_z k_p$

(c) Earth pressure at rest Condition :-

If the wall is rigid and does not move the pressure exerted on the wall, the soil behind the wall will be in a state of elastic equilibrium.

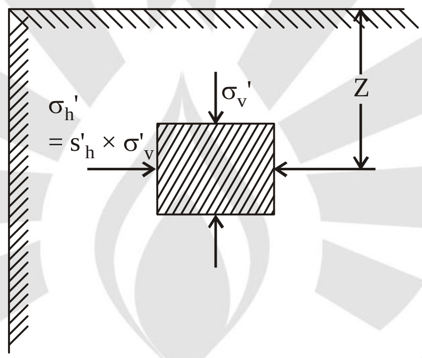


Coefficient of earth pressure at rest

Condition : $k_0 = \frac{\sigma_h'}{\sigma_v'}$

Where $\sigma_v' = \gamma \times z$

$\sigma_h' = k_0 \times (\gamma \times z)$



K_0, for most soils ranges between 0.5 and 1.0

❍❍❍