ACE

Engineering Academy

(Leading institute for ESE/GATE/PSUs)

ESE – 2019 MAINS

OFFLINE TEST SERIES

CIVIL ENGINEERING

TEST – 3

SOLUTIONS

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01(a).

Sol:

(i) Necessity of using High strength steel and concrete:

If mild steel is used, the working stress in it (i.e, 140 N/mm^2) is more or less completely lost due to elastic deformation, creep and shrinkage of concrete.

The normal loss of stress in steel is generally about 100 to 240 N/mm^2 and it is apparent that if this loss of stress is to be a small portion of the initial stress, the stress in steel in the initial stages must be very high, about 1200 to 2350 N/mm^2. These high stress ranges are possible only with the use of high strength steel.

High strength concrete is necessary in prestressed concrete since the material offers high resistance in tension shear, bond and bearing. In the zone of anchorages, the bearing stress being higher, high strength concrete is in variably preferred to minimize costs. High strength concrete is less liable to shrinkage cracks, and has a higher modulus of elasticity and smaller ultimate creep strain resulting in smaller loss of prestress in steel. The use of high strength concrete results in a reduction in the cross sectional dimensions of prestressed concrete structural elements. With reduced dead weight of the material, larger spans become technically and economically practicable.

(ii) In case of post-tensioned beams, if there is only one tendor (or) it all the tendons are tensioned simultaneously, there is no loss due to elastic deformation because the applied prestressing recorded after the elastic shortening of the member. For more than one tendon, if the tendons are successively tensioned there will be loss in a tendon during subsequent stretching the other tendons, (i.e loss will occur in tendon '1' when stretched and so on)

Hence, the 1st tensioned tendon will have maximum loss due to elastic shortening while the last tensioned tendon will not have loss.

Incase of pretensioned beams, when the prestress is applied to the concrete, elastic shortening of concrete takes place. This results in an equal and simultaneous shortening of prestressing steel. i.e loss occurs in all the tendons.

Hence the loss due to elastic shortening in post tensioned beams is less than that of pretensioned members.

01(b).

Sol: Given, a T-beam with the following data:

b_f = 2000 mm d_f = 150 mm

b_w = 300 mm D = 1000 mm, d = 950 mm (taking 50 mm cover)

M20 concrete and Fe 415 steel.

Factored design moment, M_{ud} = 2000 kN-m

Calculation of balanced moment of resistance:

$(x_u)_{max} = 0.48 \times 950 = 456$ mm

(N.A in web, & flange in constant stress zone)

$\therefore M_{el} = (0.36f_{ck})\, b_w\, (x_u)_{max}\, [d - 0.42(x_u)_{max}] + 0.45\, f_{ck}\, (b_f - b_w)\, d_f \left(d - \frac{d_f}{2}\right)$

$= 747.07 + 2008.125 = 2755.19$ kN-m

$M_{eI} > M_{ud}$ $\therefore$ UR section

$\because$ $M_{el} > M_{ud}$

Trial 1:

Assuming NA in flange.

$\Rightarrow M_{ud} = (0.36\, f_{ck})\, b_f\, x_u\, (d - 0.42\, x_u)$

$\Rightarrow 2000 \times 10^6 = (0.36 \times 20) \times 2000 \times x_u\, (950 - 0.42x_u)$

$x_u = 157.11 \text{ mm} > d_f$

Hence assumption is wrong

Trial 2:

NA in web and $d_f < \frac{3}{7} x_u$

$\Rightarrow m_{ud} = c_w jd_1 + c_f\, jd_2$

$= (0.36\, f_{ck})\, (b_w x_u)\, (d - 0.42\, x_u) + (0.45\, f_{ck})\, (b_f - b_w) \times d_f \times \left(d - \frac{d_f}{2}\right)$

$\Rightarrow 2000 \times 10^6 = (0.36 \times 20 \times 300 \times x_u\, (950 - 0.42\, x_u)) + 0.45 \times 20 \times 1700 \times 150 \times (950 - 75)$

(not possible)

$\Rightarrow x_u$ = negative value

Hence, assumption is wrong,

Trial 3:

NA in web and $d_f > \frac{3}{7}x_u$ (flange in variable stress zone)

$M_{ud} = C_w j d_1 + C_f\ j\ d_2$

$= (0.36\ f_{ck})\ (b_w x_u)\ (d - 0.42\ x_u) + (0.45\ f_{ck})\ (b_f - b_w)\ y_f \left(d - \frac{y_f}{2}\right)$

Now, $y_f = 0.15\ x_u + 0.65\ d_f = 0.15x_u + 97.5$

$\therefore$ $2000 \times 10^6 = (0.36 \times 20 \times 300 \times x_u)\ (950 - 0.42x_u)$

$+ (0.45 \times 20 \times 1700)\ (0.15\ x_u + 97.5) \left(950 - \frac{(0.15x_u - 97.5)}{2}\right)$

$\Rightarrow$ $x_u = 171.64$ kN-m $> d_f$

$\frac{3}{7}x_u = 73.48 < d_f$

And $y_f = 0.15\ x_u + 97.5 = 123.22$ mm $< d_f$

Calculation of tension steel

$C_w + C_f = T$

$(0.36\ f_{ck})\ (b_w\ x_u) + (0.45\ f_{ck})\ (b_f - b_w)\ y_f = 0.87 \times f_y \times A_{st}$

$\Rightarrow$ $(0.36 \times 20 \times 300 \times 171.46) + (0.45 \times 20 \times 1700 \times 123.22) = 0.87 \times 415 \times A_{st}$

$A_{st} = 6247.39\ mm^2$

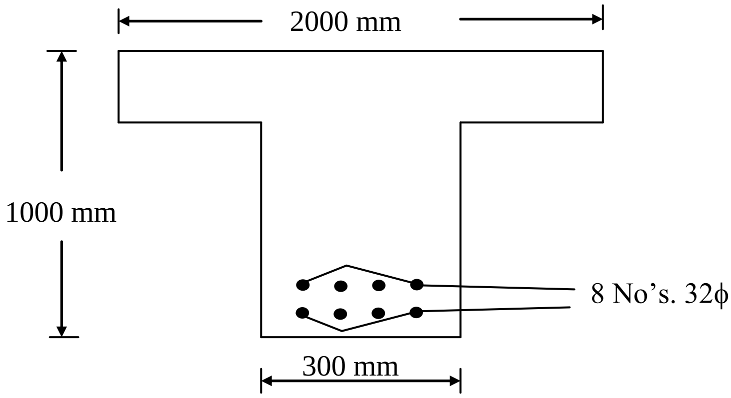
Check for maximum steel

$(A_{st})_{max} = 0.04\ b_{wd}$

$= 0.04 \times 300 \times 950$

$= 11400\ mm^2 > A_{st}$

$\therefore$ Provide 8 No's 32 ϕ in 2 layer as shown below:



01(c).

Sol: Given: M25 grade concrete, Fe 415 steel

Column size = 300 mm × 300 mm

Load = 800 kN

Safe Bearing capacity of soil = 150 kN/m^2

Shear strength of concrete = 0.33 MPa

Step 1: Dimensions:

Assuming: weight of footing + backfill = 10% of load

$\therefore$ Total load P = 800 × 1.1 = 880 kN

Base area required $= \dfrac{880}{SBC} = \dfrac{880}{150} = 5.867\ \text{m}^2$

Since square footing: $B^2 = 5.867$

B = 2.42 m

$\therefore$ Provide footing of size 2.5 m × 2.5 m

Step 2: Thickness of the Footing:

Total factored load = 1.5 × 800 = 1200 kN

Net pressure at ultimate loads, $q_u = \dfrac{1200}{2.5\times 2.5} = 192\ \text{kN/m}^2 < 225\ \text{kN/m}^2\ (1.5 \times 150)\ \therefore$ Safe

(a) From bending Moment Criteria:

Critical section is at the face of the column

$$BM = q_u \times \frac{B(B-b)^2}{8}$$

$$= 192 \times \frac{2.5}{8} \times (2.5-0.3)^2$$

$= 290.4$ kNm

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

$$\Rightarrow d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{290.4 \times 10^6}{0.138 \times 25 \times 2500}} = 183.49 \text{ mm}$$

Say d = 190 mm

(b) From One-way Shear Criteria:

Critical section is at a distance 'd' from the face of the column

Factored shear force $V_u = 192 \times q_u \times B\left(\frac{B}{2} - \frac{b}{2} - d\right)$

$$= \frac{192}{1000} \times 2.5 \times 1000\left(\frac{2500}{2} - \frac{300}{2} - d\right)$$

$= 480\ (1100 - d)$ N

Shear strength of concrete = 0.33 MPa

∴ Shear resistance = $0.33 \times 2500 \times d$

$V_c = 825d$ N

Keep $V_u \le V_c$

$\Rightarrow 480\ (1100 - d) \le 825\ d$

$$\Rightarrow d \ge \frac{480 \times 1100}{(480 + 825)}$$

$\Rightarrow d \ge 404.6$ mm

∴ Provide d = 410 mm

Effective cover = 75 mm

$\therefore$ Total thickness = 485 mm

Check for two way shear:

Critical section is at a distance $\frac{d}{2}$ from fact of column

$V_u = q_u\,[B^2 - (b + d)^2]$

$= 192\,[2.5^2 - (0.3 + 0.41)^2]$

$= 1103.21$ kN

$$\tau_v = \frac{V_u}{4(b+d)d} = \frac{1103.21\times10^3}{4\times(300+410)\times410} = 0.947 \text{ MPa}$$

$\tau_v = \beta\tau_c$

$\beta = 1$ for square column

$$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25} = 1.25 \text{ MPa}$$

$\tau_v = 1.25$ MPa

$\therefore$ Safe in punching.

Hence dimensions of footing = 2.5 × 2.5 × 0.485 m

01(d).

Sol: Given:

b = 300 mm; D = 650 mm

M = 200 kNm, $T_u = 100$ kNm

$V_u = 100$ kN

M20 grade, Fe-415 steel

Effective cover = 50 mm

$\therefore$ d= 650 – 50 = 600 mm

Step 1: Equivalent Moments:

$$M_{Tu} = \frac{T_u}{1.7}\left(1+\frac{D}{B}\right)$$

$$= \frac{100}{1.7}\left(1+\frac{650}{300}\right)$$

$= 186.27$ kNm < M_u

$\therefore$ $M_{Tu} < M_u$ longitudinal reinforcement is not required compression face

Step 2: Design:

$\therefore$ Bending moment for design = $M_u + M_{Tu}$

= 200 + 186.27

= 386.27 kNm

Reinforcement required:

$$\frac{P_t}{100} = \frac{f_{ck}}{2f_y}\left[1-\sqrt{1-4.598\frac{R}{f_{ck}}}\right]$$

$$R = \frac{M_u}{bd^2} = \frac{386.27\times10^6}{300\times600^2} = 3.576$$

$$\therefore \frac{P_t}{100} = \frac{20}{2\times415}\left[1-\sqrt{1-4.598\times\frac{3.576}{20}}\right]$$

$P_t = 1.39\%$

$$\therefore A_{st} = \frac{1.39}{100}\times300\times600 = 2502\ \text{mm}^2$$

No. of 25 mm bars required = $\dfrac{2502}{\frac{\pi}{4}\times25^2} = 5.09$ Say 6 No's

Step 3: Side Face Reinforcement:

Since depth > 450 mm, addition side face reinforcement required = 0.1% bD

$$= \frac{0.1}{100}\times300\times650$$

= 195 mm^2 equally distributed on two side faces

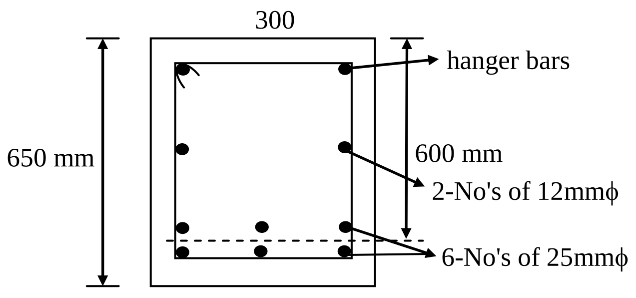
$\therefore$ On one face reinforcement $= \dfrac{195}{2}$ = 87.5 mm^2

$\therefore$ Provide 1-12 mm ϕ bars, on each face

$\therefore$ [Side reinforcement provided $= 2\times\dfrac{\pi}{4}\times12^2$ = 226.2 mm^2

Hence OK

Provide 2-12ϕ bar as hanger bar for holding transverse reinforcement



01(e).

Sol: The following are the different constituents present in brick earth:

(i) Silica:

- A good brick earth contains about 50 to 60 per cent of silica.
- Presence of silica prevents the brick from cracking, shrinking and warping.
- Thus it helps in imparting uniform shape to the brick.
- Excess of silica destroys the cohesion between particles and makes the bricks brittle.

(ii) Alumina:

- A good brick earth contains about 20 to 30 per cent of alumina.
- Alumina imparts plasticity to the brick earth so that it can be easily moulded.
- Excess of alumina makes the raw bricks shrink and warp during drying and burning.

(iii) Lime:

- Lime, in finely powdered state, not exceeding 5% is desirable in a good brick earth.
- Lime prevents shrinkage of raw bricks.
- Excess of lime causes the melting point of brick to decrease hence, it loses its shape.

(iv) Iron Oxide:

- About 5 to 6 percent of iron oxide is desirable in a good brick earth.
- It helps the ingredients to fuse during the burning process.
- It also imparts red colour to the bricks.
- Excess of oxide of iron makes the bricks dark blue or blackish.

(v) Magnesia:

• A small quantity of magnesia in brick earth imparts yellow colour to bricks.

• It decreases shrinkage, but excess of magnesia leads to the decay of bricks.

(vi) Iron pyrites:

If these are present in the brick earth, bricks get crystallized and disintegrated during burning.

(vii) Alkalies:

If the alkalies are present in bricks then bricks will absorb moisture from the atmosphere when they are used in masonry and when it gets evaporated, it leaves behind grey or white deposit on the wall surface.

(viii) Pebbles:

The presence of pebbles are undesirable in brick earth because they will not allow the clay to get mixed uniformly and thoroughly, which will result in weak and porous bricks.

(ix) Organic Matter:

Presence of organic matter in brick earth assists in burning, but if it is not completely burnt, bricks become porous.

02(a).

Sol: Given:

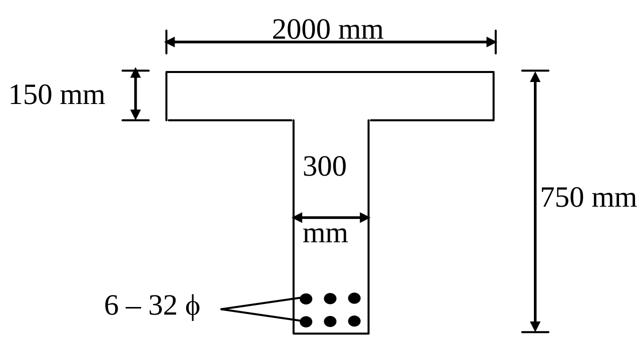
Flange width b = 2000 mm

Span, l_o = 9 m = 9000 mm

Flange thickness, D_f = 150 mm

Overall depth, D = 750 mm

Rib width, b_w = 300 mm



$$A_{st} = 6 \times \frac{\pi}{4} \times 32^2 = 4825.486 \text{ mm}^2$$

Effective width of flange for an isolated T-beam is given as

$$b_f = \frac{\ell_o}{\left(\frac{\ell_o}{b}\right) + 4} + b_w = \frac{9000}{\frac{9000}{2000} + 4} + 300$$

$\Rightarrow b_f = 1358.82$ mm < b

Now calculate the moment of resistance of the beam, we need to calculate the actual depth of neutral axis.

Assuming that the NA lies in the flange

Force of compression $C = 0.36\ f_{ck}\ b_f\ x_u$

Force of tension, $T = 0.87\ f_y\ A_{st}$

But C = T

$\therefore\ 0.36f_{ck}\ b_f\ x_u = 0.87f_yA_{st}$

$$\Rightarrow x_u = \frac{0.87f_yA_{st}}{0.36f_{ck}b_f} = \frac{0.87\times500\times4825.486}{0.36\times25\times1358.82}$$

$\Rightarrow x_u = 171.64$ mm

$\therefore x_u > D_f$

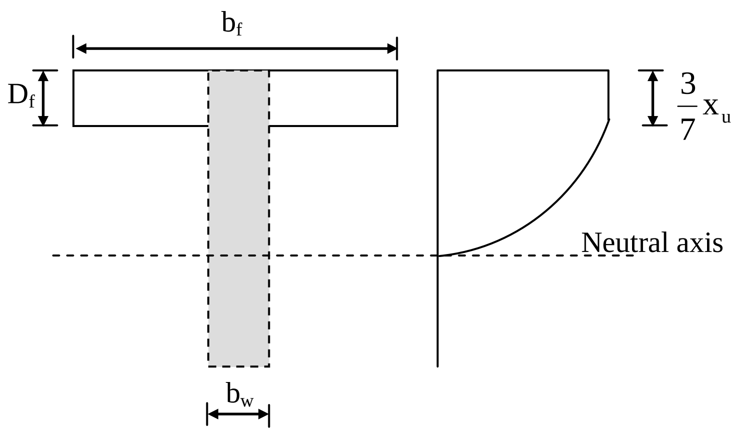
Hence our assumption is wrong

$\therefore$ NA lies in the web

As we can see from the stress diagram, depth of rectangular portion of stress diagram

$$= \frac{3}{7}x_u = \frac{3}{7}\times171.64 = 73.56\,\text{mm}$$

$$\therefore D_f > \frac{3}{7}x_u$$

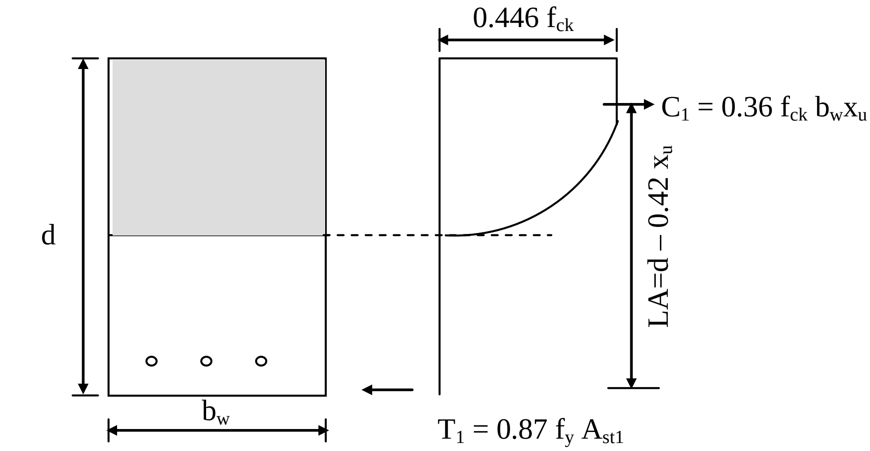


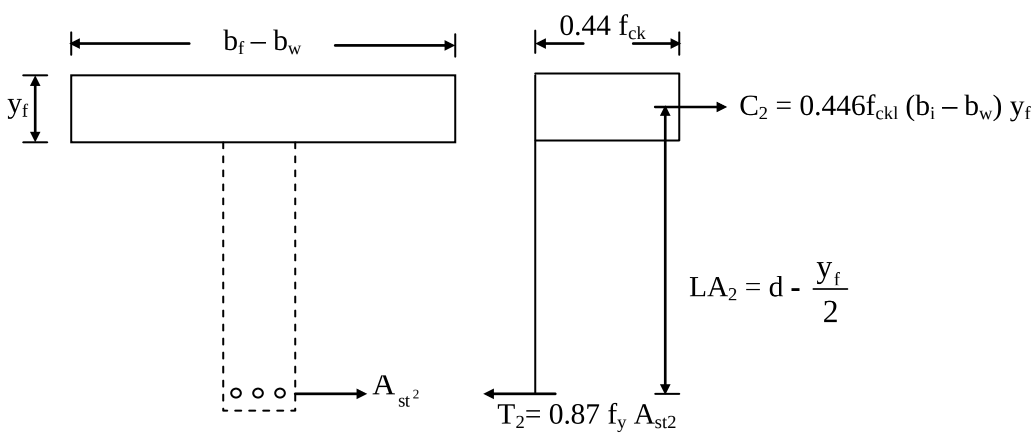
It means that depth of flange is greater than the rectangular portion of stress diagram.

In this case it is assumed that depth of rectangular portion of the stress block is equal to y_f

Where $y_t = 0.15x + 0.65\ D_f < D_f$

$\therefore$ The given T-beam can be shown as





$C = C_1 + C_2$

$\quad = 0.36\ f_{ck}\ b_w x_u + 0.446\ f_{ck}\ (b_f - b_w)\ y_f$

$T = T_1 + T_2$

$\quad = 0.87\ f_y\ A_{st1} + 0.87\ f_y\ A_{st2} = 0.87 f_y\ A_{st}$

But $\quad C = T$

$\therefore\ 0.36\ f_{ck}\ b_w\ x_u + 0.446\ f_{ck}\ (b_f - b_w)\ y_f = 0.87\ f_y A_{st}$

$\Rightarrow 0.36 \times 25 \times 300 \times x_u + 0.446 \times 25 \times (1358.82 - 300)\ y_f = 0.87 \times 500 \times 4825.486$

$\Rightarrow 2700\ x_u + 11805.843 \times (0.15 x_u + 0.65 \times 150) = 2099086.41$

Solving $\qquad x_u = 212.043$ mm

$\qquad x_u > D_f$ (OK)

and $\quad \dfrac{3}{7} x_u = \dfrac{3}{7} \times 212.043 = 90.876$ mm

$\therefore \quad D_f > \frac{3}{7} x_u$

$\therefore$ Moment of the resistance of the beam with respect to concrete is given by

$M_R = C_1 \times L_1 + C_2 \times L_2$

$= 0.36\, f_{ck}\, b_w\, x_u \times (d - 0.42\, x_u) + 0.446\, f_{ck}\, (b_f - b_w)\, y_f\, (d - y_f/2)$

Assuming effective cover = 50 mm

$\therefore$ Effective depth d = D – 50

= 750 – 50 = 700 mm

$y_f = 0.15x_u + 0.65\, D_f$

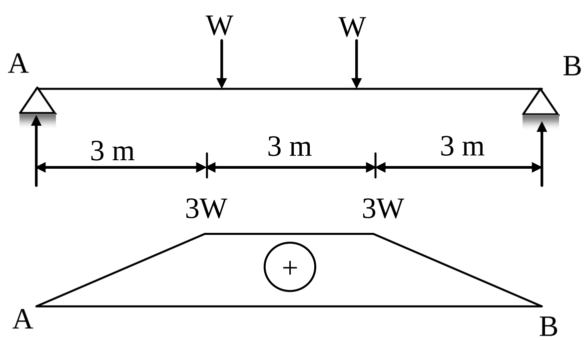
$= 0.15 \times 212.043 + 0.65 \times 150 = 129.306$ mm

$M_R = 0.36 \times 25 \times 300 \times 212.043 \times (700 - 0.42 \times 212.043) + 0.446 \times 25 \times (1358.82 - 300) \times (0.15 \times 212.043 + 0.65 \times 150) \times (700 - 64.6532)$

= 349,774,096.8 + 969,902,258.6

= 1319.677 kN-m

Now, to calculate the magnitude of point load, equating the bending moment to moment of resistance



$W_d = 0.48 \times 25 = 12$ kN/m

$\frac{M_U}{1.5} = M_W = 879.8$ kN-m

$879.8 = \frac{W_d \ell^2}{8} + \frac{W\ell}{3} = \frac{12 \times 9^2}{8} + \frac{W\ell}{3}$

W = 252.8 kN

02(b).

(i)

Sol: A building that lacks symmetry and has discontinuity in geometry mass (or) load resisting element is called irregular.

Irregularities are of two types

(a) Horizontal irregularities

(b) Vertical irregularities

Horizontal irregularities refer to as symmetrical plan shapes (or) discontinuities in horizontal resisting elements such as large openings, re-entrant corners cut-outs.

There are numerous examples of past earthquakes in which the cause of failure of reinforced concrete building has been ascribed to irregularities in configurations.

Irregularities are mainly categorized as

(i) Horizontal Irregularities

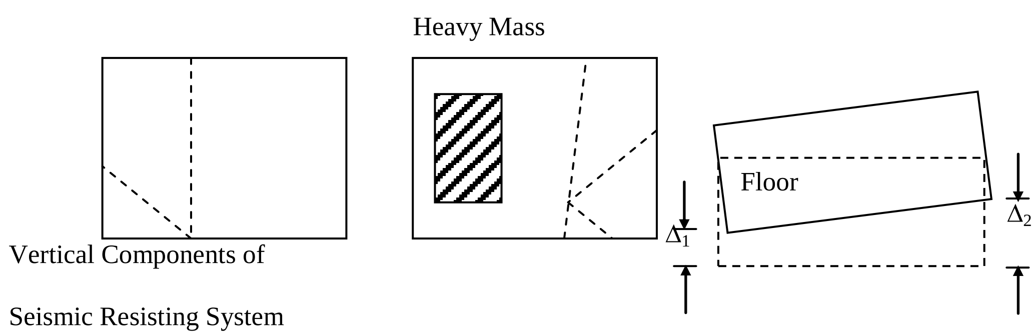
(ii) Vertical Irregularities

Horizontal Irregularities

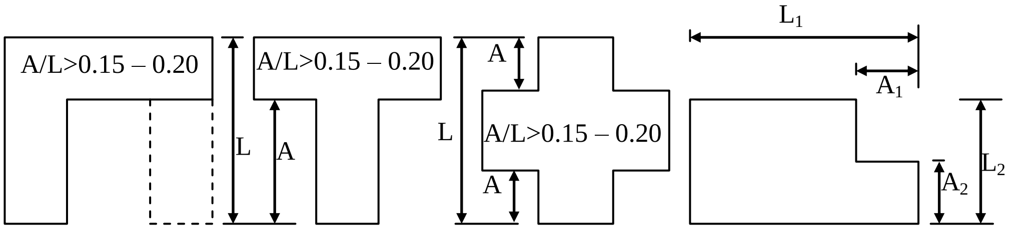
Horizontal irregularities refer to asymmetrical plan shapes (e.g. L-, T-, U-, F-) or discontinuities in the horizontal resisting elements (diaphragms) such as cut-outs, large openings, re-entrant corners and other abrupt changes resulting in torsion, diaphragm deformations, stress concentration.

Irregularity Type and Description

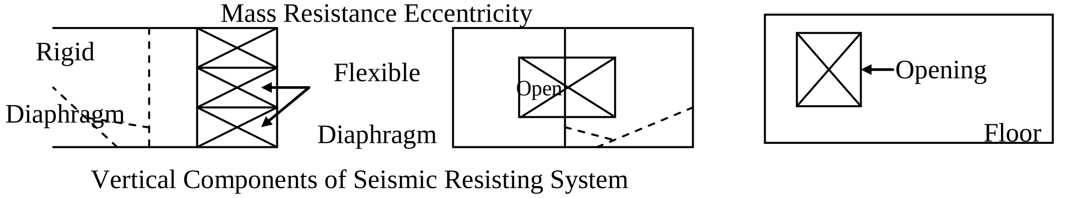
(i) **Torsion Irregularity:** To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure.



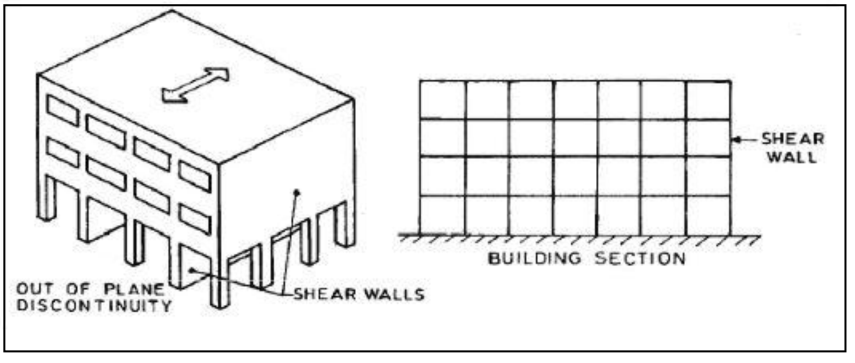
(ii) Re-entrant Corners: Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the reentrant corner are greater than 15 percent of its plan dimension in the given direction



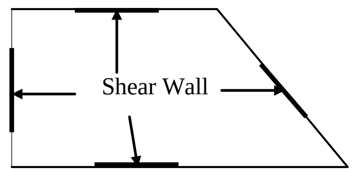
(iii) Diaphragm Discontinuity: Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next



(iv) Out-of-Plane Offsets: Discontinuities in a lateral force resistance path, such as out-of plane offsets of vertical elements



(v) **Non-parallel Systems:** The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements



02(b).

(ii)

Sol: Base shear $V_B = A_h W$

A_n = design horizontal acceleration spectrum

$$= \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

Z = zone factor = 0.24

I = Importance factor = 1.5 for hospital

R = Response reduction factor = 5

$\frac{S_a}{g}$ = Spectral acceleration

Natural period of RC frame building = $0.075\ h^{0.75}$

$= 0.075 \times 15^{0.75}$

$= 0.57$

For $0.4 \le T \le 4.00$, $\frac{S_a}{g} = \frac{1}{T} = \frac{1}{0.57} = 1.75$

W = total seismic weight = 2000 + 4(3000)

= 14000 kN

$\therefore$ Total base shear $= \frac{0.24}{2} \times \frac{1.5}{5} \times 1.75 \times 14000$

V_B = 882 kN

Distribution of Base Shear:

Base shear is distributed along the height of building as per $Q_i = V_B = \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}$

$\therefore$ **For first floor:**

$$Q_1 = 882 \times \frac{3000 \times 3^2}{[3000 \times 3^2 + 3000 \times 6^2 + 3000 \times 9^2 + 3000 \times 12^2 + 2000 \times 15^2]}$$

$$= 882 \times \frac{3000 \times 3^2}{1260000} = 18.9 \text{ kN}$$

For Second Floor:

$$Q_2 = 882 \times \frac{3000 \times 6^2}{1260000} = 75.6 \text{ kN}$$

For Third Floor:

$$Q_3 = 882 \times \frac{3000 \times 9^2}{1260000} = 170.1 \text{ kN}$$

For Fourth Floor:

$$Q_4 = 882 \times \frac{3000 \times 12^2}{1260000} = 302.4 \text{ kN}$$

For Fifth Floor:

$$Q_5 = 882 \times \frac{2000 \times 15^2}{1260000} = 315 \text{ kN}$$

02 (c).

Sol: Given mix proportion 1 : 1.2 : 2.1 (volume batching)

w/c = 0.39

$V_c : V_{FA} : V_{CA} = 1 : 1.2 : 2.1$ & $w_w / w_c = 0.39$

$V_{FA} = 1.2\ V_c$; $V_{CA} = 2.1\ V_c$; $w_w = 0.39\ w_c$

Net volume of concrete $= 1 - \frac{2}{100}$

$= 0.98 \text{ m}^3$

$$\Rightarrow 0.98 = \frac{w_w}{9.81} + \frac{w_c}{3.15 \times 9.81} + \frac{w_{FA}}{2.6 \times 9.81} + \frac{w_{CA}}{2.5 \times 9.8}$$

$$= \frac{0.39 w_c}{9.81} + \frac{14.4 v_c}{315 \times 9.81} + \frac{16.2 V_{FA}}{2.6 \times 9.81} + \frac{15.8 V_{CA}}{2.5 \times 9.81}$$

$$= \frac{0.39 \times 14.4 v_c}{9.81} + \frac{14.4 v_c}{3.15 \times 9.81} + \frac{16.2 \times 1.2 v_c}{2.6 \times 9.81} + \frac{15.8 \times 2.1 v_c}{2.5 \times 9.81}$$

$\Rightarrow v_c = 0.311 \text{ m}^3$

$$w_c = \frac{0.311 \times 14.4 \times 1000}{9.81} \text{kg} \qquad w_w = 0.39\ w_c = 178 \text{ kg}$$

$= 457$ kg

$\Rightarrow v_{FA} = 1.2\ v_c = 0.311 \times 1.2 = 0.373\ m^3$

$$w_{FA} = \frac{0.373 \times 16.2 \times 1000}{9.81} \text{kg}$$

$= 616$ kg

$\Rightarrow v_{CA} = 2.1\ v_c = 0.311 \times 2.1 = 0.653\ m^3$

$$w_{CA} = \frac{0.653 \times 15.8 \times 1000}{9.81} \text{kg}$$

$= 1052$ kg

03(a)

(i)

Sol: When a reinforcing bar is embedded in concrete, concrete adheres to its surface and resists any force that tries to cause slippage of bar relative to its surrounding concrete, this phenomenon is called bond.

Significance:

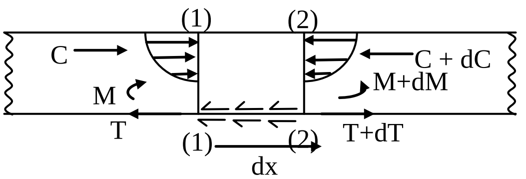
- Bond is responsible for transfer of axial force from reinforcing bar to the surrounding concrete, thereby providing strain compatibility and responsible for composite action of concrete and steel.
- Hence the assumption plane section remains plane even after bending is valid only if bond is fully effective.
- The axial stress in a reinforcing bar can undergo variation from point to point along the length through the action of bond. This is due to variation of bending moment along length of flexural member.

03. (a)

(ii)

Sol: Consider 2 sections (1)-(1) and (2)-(2) a beam subjected to a bending moment

The distance between the sections = 'dx'



At section (1)-(1): $M = C_j d = T_j d \rightarrow (1)$ (jd = lever arm)

At section (2)-(2): $M + dM = (C + dC)\, jd = (T + dT)jd \rightarrow (2)$

From equation (1) and (2)

$dM = dC\ jd = dT\ jd$

$\therefore dT = \frac{dM}{jd}$

This unbalanced force is transferred to surrounding concrete by flexural bond developed along interface.

Assuming flexural bond stress (u_f) is uniformly distributed

From equilibrium: $dT = u_f \times$ area of interface

$\Rightarrow dT = u_f \times (\pi\phi) \times n \times dx$ [n is no. of bars]

$$\Rightarrow u_f = \frac{dT}{\pi\phi n dx}$$

$$\Rightarrow u_f = \frac{\left(\frac{dM}{jd}\right)}{\Sigma 0 dx}$$

$$\Rightarrow u_f = \frac{(dM/dx)}{\Sigma 0(jd)}$$

$$\Rightarrow u_f = \frac{V}{\Sigma 0(jd)}$$

03.(a)

(iii)

Sol: **Limit State:**

It is an acceptable limit for the safety and serviceability requirements before failure occur. It can be classified into the following type.

1. Limit State of Collapse: It is the limit state on attainment of which the structure is likely to collapse. It relates the stability and ultimate strength of the structure. Design to this limit state ensure safety of structure from collapse.

2. Limit state of Serviceability: Limit state of serviceability relate to performance (or) behaviour of structure at working loads and are based on causes affecting serviceability of the structure.

Limit State

Limit State of Collapse	Limit State of Serviceability
(i) Flexure	(i) Deflection
(ii) Shear	(ii) Cracking
(iii) Torsion	(iii) Vibrations
(iv) Compression	
(v) Bond	

03(b).

Sol: Given : capacity = 500 kilo litre

Depth = 4.2 m; free board = 200 mm

M20 grade concrete, Fe 415 steel,

Design:

Step 1: dimensions:-

Effective depth = H = 4.2 – 0.2 = 4 m

Assuming ‘D’ as diameter of tank

$$\frac{\pi}{4}D^2 \times 4 = \frac{500 \times 10^3}{10^3}$$

$\Rightarrow$ D = 12.6 m

$\therefore$ Provide diameter of 13.0 m

Step 2: Reinforcement required:

Maximum hoop tension = $w\dfrac{HD}{2} = 9800 \times 4 \times \dfrac{13}{2}$

= 254800 N/m ht at base

$\sigma_{st} = 150 \text{ MPa}$

$\therefore$ Area of hoop steel = $\dfrac{254800}{150} = 1698.67 \text{ mm}^2/\text{m height}$

Using 20 mm ϕ bars, spacing = $\dfrac{1000 \times \frac{\pi}{4} \times 20^2}{1698.67}$

= 184.94 mm

$\therefore$ Provide 20 mm hoops @ 180 mm c/c

$\therefore$ Actual steel = $\dfrac{1000}{180} \times \dfrac{\pi}{4} \times 20^2 = 1745.33 \text{ mm}^2$

Assuming t as thickness of wall:

$$\sigma_{ct} = \frac{wH\frac{D}{2}}{1000T + (m-1)A_{sh}}$$

$$\Rightarrow 1.2 = \frac{254800}{1000T + (13-1)\times 1745.33}$$

$\Rightarrow$ T = 191.4 mm

From empirical formula T = (3H + 5) cm = (3 × 4 + 5) = 17 cm

= 170 mm

$\therefore$ Provide thickness of 195 mm

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

Step 3: Vertical reinforcement

Distribution and temperature reinforcement in vertical direction

= 0.3% for 100 mm thick section to 0.2% for 450 mm thick section

For 195 mm thick section = 0.3 – 0.1 $\dfrac{(195-100)}{(450-100)}$

= 0.272%

$$A_{st} = \frac{0.272}{100} \times 195 \times 1000 = 530.4 \text{ mm}^2$$

Using 10 mm ϕ ; spacing = $= \dfrac{1000 \times \frac{\pi}{4} \times 10^2}{520.6} = 148\,\text{mm}$

∴ Provide 10 mm ϕ @ 140 mm c/c. n vertical direction

Step 4: Tank floor:

Since the tank is resting on firm ground, provide a nominal thickness of 150 mm.

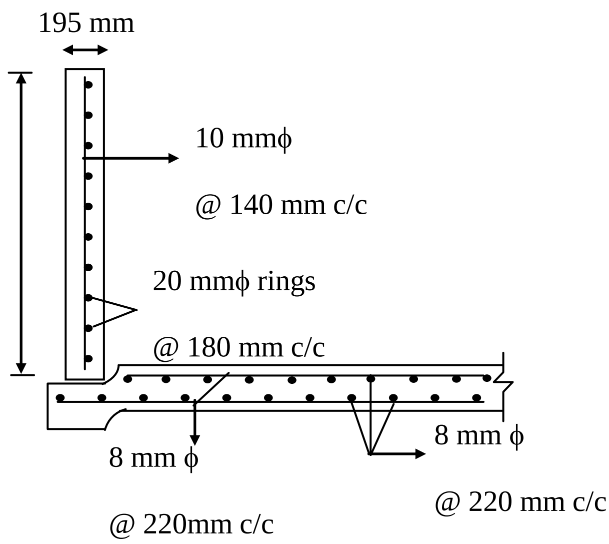
Area of steel required = $\dfrac{0.3}{100} \times 150 \times 1000 = 450 \text{ mm}^2$ in each direction

Area of reinforcement near each face = $\dfrac{450}{2} = 225 \text{ mm}^2$

Using 8 mm ϕ bars spacing = $\dfrac{1000}{225} \times \dfrac{\pi}{4} \times 8^2 = 223.4 \text{ mm}$

∴ Provide 10 mm ϕ @ 220 mm c/c.

Step 5: Detailing:



03(c).

Sol: Given mix proportion = 1 : 1.5 : 3 (weigh batching)

w/c = 0.5

$w_c : w_{FA} : w_{CA} = 1 : 1.5 : 3$

$\Rightarrow w_{FA} = 1.5\, w_c$; $w_{CA} = 3\, w_c$; $w_w = 0.5\, w_c$

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

$$1 = \frac{0.5w_c}{9.81} + \frac{w_c}{3.15\times 9.18} + \frac{1.5w_c}{2.6\times 9.81} + \frac{3w_c}{2.5\times 9.81}$$

$\Rightarrow w_c = 3.781$ kN $\quad v_C = 3.781/14.4 = 0.263\ m^3$

$$\Rightarrow w_c = \frac{3.781\times 1000}{9.81}\text{kg} = 385\text{ kg}$$

$W_w = 0.3\, w_c = 193$ kg

$w_{FA} = 1.5 \times 3.781$ kN

$= 5.672$ kN $= 578$ kg

$V_{FA} = 5.672/16.2\ m^3$

$= 0.35\ m^3$

$w_{CA} = 3 \times 3.781 = 11.343$ kN $\Rightarrow 1156$ kg

$V_{FA} = 11.343 / 15.8 = 0.718\ m^3$

Mix proportion based on volume batching

$$= 1 : \frac{0.35}{0.263} : \frac{0.718}{0.263}$$

= 1 : 1.33 : 2.73

04(a).

(i)

Sol:

The different industrial applications of Fly Ash are as follow:

1. Manufacture of PPC:

- Because of its pozzolanic properties, fly ash is widely used in the production of Portland Pozzolanic Cement (PPC).
- Compared to OPC, PPC has less rate of early strength gain, but due to its pozzolanic action, the ultimate strength of PPC is more than OPC.

- Due to the absorption of Ca(OH)2 in the pozzolanic action, PPC is more resistant against chemical attack.

2. Manufacture of Fly ash Bricks:

- Along with pozzolanic properties, fly ash also has ceramic properties.
- Therefore, it can also be used for the production of Fly ash bricks.
- The bricks produced using fly ash are generally superior in quality and have higher compressive strength compared to ordinary clay bricks.

3. Manufacture of Asphalt concrete:

- Asphalt concrete is a composite material consisting of an asphalt binder and mineral aggregate.
- Because of its hydrophobic nature, use of fly ash in the production of asphalt concrete makes it more resistant against deterioration caused by water.
- Because of pozzolanic action, it also improves the strength of the Asphalt concrete.

4. Construction of Embankments:

- Fly ash can also be used for the construction of embankments, thus saving top soil which otherwise is conventionally used.

5. Soil Stabilization:

- Soil stabilization is the process of improving the engineering properties of soil.
- Use of fly ash, mainly class C, can increase the shear strength of the soil, thus improving its load bearing capacity.

6. Admixture:

- Fly ash is also used as an admixture in the preparation of concrete.
- Use of fly ash reduces segregation, bleeding and shrinkage of cement.

04 (a)

(ii).

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

1. Flakiness Index Test.
2. Elongation Index Test.
3. Aggregate Crushing Value Test.
4. Aggregate Impact Value Test.
5. Aggregate Abrasion Value Test.
6. Soundness Test.

The toughness of coarse aggregates is assesed based on the Aggregate Impact Value Test.

Aggregate Impact Value Test:

- This test gives the Aggregate Impact Value (AIV), which is an index of the resistance of aggregates to sudden shock or impact.
- The apparatus used for this test is called Impact Testing Machine.
- A sample of aggregates, in surface dry condition, which pass through the 12.5 mm sieve and are retained on 10 mm sieve are taken for this test (W1).
- The whole sample is compacted in three layers with 25 blows each time.
- Now the hammer of the Impact Testing Machine is raised until its lower face is 380 mm above the upper surface of the aggregate sample and allowed to fall freely.
- The test sample is subjected to a total of 15 blows, each being delivered at an interval of not less than one second.
- The sample is now sieved on a 2.36 mm sieve and the fraction passing through the sieve is weighed (W2).
- Aggregate Impact Value (AIV) = (W_2/W_1)x100.

04(b).

Sol: M20 $\rightarrow f_{ck} = 20$ N/mm^2

Fe-415 $\rightarrow f_y = 415$ N/mm^2

(i) Factored Load, $P_u = 1.5$ P

$= 1.5 \times 1000 = 1500$ kN

(ii) Size of the column

Assume longitudinal steel is 1% of A_g

$P_u = 1.05\ [0.4\ f_{ck}\ A_c + 0.67 f_y\ A_{sc}]$

$$1500 \times 10^3 = 1.05\left[0.4 \times 20 \times \left(A_g - \frac{1}{100}A_g\right) + 0.67 \times 415 \times \frac{1}{100}A_g\right]$$

$= 11.236\ A_g$

$A_g = 133214.9$ mm^2

$$\frac{\pi}{4}D^2 = 133214.9$$

Dia of column, D = 411.8 mm

Provide D = 420 mm

(iii) Check effective length of column,

$L = 0.65l$

$= 0.65 \times 3 = 1.95$ m

Slenderness ratio, $\lambda = \dfrac{L}{D} = \dfrac{1950}{420}$

$= 4.6 < 12$

$\therefore$ It is designed as short column

$$\frac{D}{20} = \frac{420}{20} = 21 \text{ mm}$$

Minimum eccentricity

$$e_{min} = \frac{\ell}{500} + \frac{D}{30} \text{ (or) 20 mm which ever is max}$$

$$= \frac{3000}{500} + \frac{420}{30} = 20 \text{ mm}$$

$$e_{min} < \frac{D}{20}$$

$\therefore$ It is designed as axially loaded column

(iv) Longitudinal Steel:

$$A_{sc} = \frac{1}{100} A_g$$

$$= \frac{1}{100} \times 133214.9 = 1332.149 \text{ mm}^2$$

No. of 16 mm ϕ bars required

$$n = \frac{A_{sc}}{a_{sc}} = \frac{1332.149}{\frac{\pi}{4} \times 16^2} = 6.6$$

Provide 7-16 mmϕ

(v) Spiral Reinforcement:

(a) Diameter of Helix:

$$\left.\begin{array}{l} \bullet\ \frac{1}{4}\phi_{LL} = \frac{1}{4} \times 16 = 4 \text{ mm} \\ \bullet\ 6\text{mm} \end{array}\right\} \text{max}$$

Provide 6 mm dia helix

(b) Pitch Calculation:

$D_c = D - 2 \times \text{cover} + 2\phi_h$

$= 420 - 2 \times 40 + 2 \times 6 = 352$ mm

$$V_h = \pi(D_c - \phi_h)\frac{\pi}{4}\phi_h^2$$

$$= \pi(352 - 6)\frac{\pi}{4} \times 6^2$$

$$V_c = \frac{\pi}{4} D_c^2 . P = \frac{\pi}{4} \times 352^2 \times P$$

$$\frac{A_g}{A_c} = \frac{\frac{\pi}{4} D^2}{\frac{\pi}{4} D_c^2} = \frac{D^2}{D_c^2} = \frac{420^2}{352^2} = 1.42$$

$$0.36\left[\frac{A_g}{A_c} - 1\right]\frac{f_{ck}}{f_y} = 0.36[1.42 - 1] \times \frac{20}{415} = 7.286 \times 10^{-3}$$

$$\frac{V_h}{V_c} \geq 0.36\left[\frac{A_g}{A_c} - 1\right]\frac{f_{ck}}{f_y}$$

$$\frac{\pi(352 - 6)\frac{\pi}{4} \times 6^2}{\frac{\pi}{4} \times 352^2 \times P} \geq 7.286 \times 10^{-3}$$

$P \leq 43.35$ mm

Check:

(i) < 75 mm

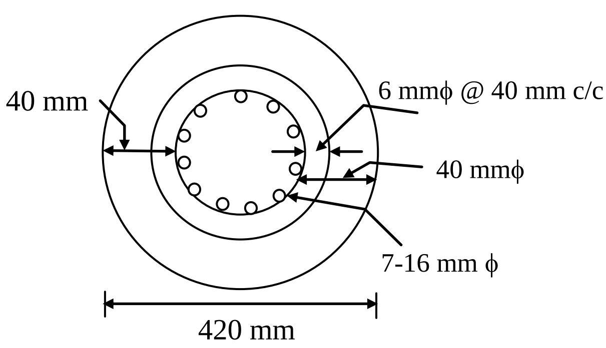
(ii) $< \frac{1}{6} D_c \left(\frac{1}{6} \times 352 = 58.6 \text{ mm}\right)$

(iii) > 25 mm

(iv) > $3\phi_h$ ($3 \times 6 = 18$ mm)

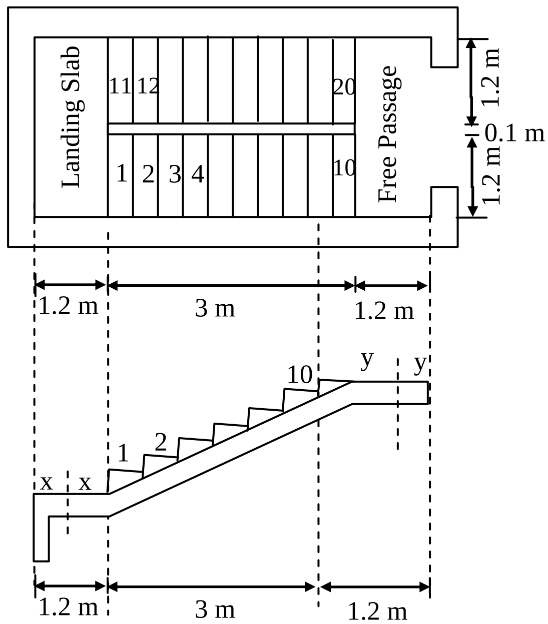
Provide 40 mm c/c

(vi) Sketch:



04(c).

Sol:



(i) Effective span (L):

Total going length (G) = No. of treads $\times$ Tread

$= 10 \times 300 = 3000$ mm

$= 3$ m

Width of landing slab

$$X = Y = \frac{1.2}{2} = 0.6\,\text{m} < 1\text{m} \quad \therefore \text{ O.K}$$

when x < 1 m

y < 1 m then

span L (m) is taken as L = G + X + Y]

L = G + X + Y

= 3 + 0.6 + 0.6 = 4.2 m

(ii) Assume thickness of waist slab:

Take $\frac{L}{d} = 30$

$$d = \frac{L}{30} = \frac{4200}{30}$$

$= 140$ mm

Overall thickness of waist

$$w = 140 + \text{clear cover} + \frac{\phi}{2}$$

$$= 140 + 20 + \frac{10}{2}$$

$= 165$ mm

(iii) Loads:

$$B = \sqrt{R^2 + T^2} = \sqrt{0.18^2 + 0.3^2}$$

$= 0.35$ m

Self weight (waist + steps + floor finishes)

Assume thickness of finishes = 15 mm

$$w_D = \left[wB + \frac{1}{2}RT\right]\frac{25}{T} + [f.f \times T]\frac{23.5}{T}$$

$$= \left[0.165 \times 0.35 + \frac{1}{2} \times 0.18 \times 0.3\right]\frac{25}{0.3} + [0.015 \times 0.3]\frac{23.5}{0.3}$$

$= 7.06 + 0.375 = 7.435$ kN/m^2 on plan

Design load = 1.5(7.435 + 5)

$= 18.65$ kN/m^2

(iv) Design Bending Moment:

$$M_u = \frac{w_u L^2}{10} = \frac{18.65 \times 4.2^2}{10}$$

$= 32.9$ kN-m

(v) Thickness of Waist Slab Required:

$$d=\sqrt{\frac{M_u}{R_u b}}=\sqrt{\frac{32.9\times10^6}{0.138\times20\times1000}}$$

$= 109.18$ mm < 140 mm $\therefore$ O.K

Provide d = 140 mm, w = 165 mm

(vi) Area of Steel required in Waist:

(a) Main Steel:

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

$$=\frac{0.5\times20}{415}\left[1-\sqrt{1-\frac{4.6\times32.9\times10^6}{20\times1000\times140^2}}\right]1000\times140$$

$= 730.24\ mm^2$

No. of 10 mm ϕ bars required

$$n=\frac{A_{st}}{a_{st}}=\frac{730.24}{\frac{\pi}{4}\times10^2}$$

$= 9.3 \simeq 10$

Provide in 1.2 m, 12 numbers

(b) Distribution Steel:

0.12% of gross c/s

$$\frac{0.12}{100}\times1000\times165=198\ mm^2$$

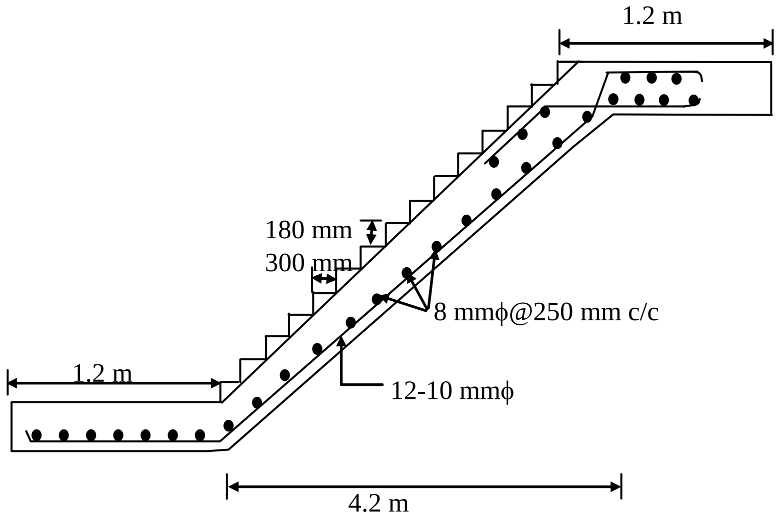
Spacing:

$$S=\frac{1000\,a_{st}}{A_{st}}=\frac{1000\times\frac{\pi}{4}\times8^2}{198}$$

$= 253.8$ mm

Provide 8 mm ϕ @ 250 mm c/c

(vii) Sketch:



05(a).

Sol: Given: Size of beam = 200 mm × 400 mm

Live load = 6 kN/m

Span = 8 m

Prestressing force = 400 kN

Resultant stress at bottom = 0 under service conditions

Step 1: Section Properties:

Area of cross-section 'A' = 200 × 400 = 8×10^4 mm^2

Section modulus Z $= \dfrac{BD^2}{6} = \dfrac{200 \times 400^2}{6} = 5.33 \times 10^6$ mm^3

Step 2: Moment Calculation:

UDL due to self weight = 25 × 0.2 × 0.4 = 2 kN/m

UDL live load = 6 kN/m

Total load under service conditions = 8 kN/m

Bending moment at midspan $= \dfrac{w\ell^2}{8} = \dfrac{8 \times 8^2}{8} = 64$ kNm

Step 3: Stress Calculation:

Under Service conditions

Resultant stress at bottom $= \frac{P}{A} + \frac{Pe}{Z} \frac{-(M_{DL}+M_{LL})}{Z} = 0$

$$\Rightarrow \frac{500\times10^3}{8\times10^4} + \frac{500\times10^3\times e}{5.33\times10^6} - \frac{64\times10^6}{5.33\times10^6} = 0$$

$\Rightarrow$ e = 61.37 mm

$\therefore$ Location of cable from bottom $= \frac{D}{2} - e$

$$= \frac{400}{2} - 61.375 = 138.625 \text{ mm}$$

05(b).

Sol: Given: Size of column = 450 mm × 600 mm

Area of steel bars = $A_{st} = 4\times\frac{\pi}{4}\times20^2 = 1256.63 \text{ mm}^2$

Effective length = 3.0 m

M25 grade concrete Fe 415 grade steel

Step 1: Minimum Eccentricities:

$$e_{x,min} = \frac{\ell}{500} + \frac{D_x}{30}$$

$$= \frac{3000}{500} + \frac{600}{30}$$

$$= 26 \text{ mm}$$

$$e_{y,min} = \frac{\ell}{500} + \frac{D_y}{30}$$

$$= \frac{3000}{500} + \frac{450}{30} = 21 \text{ mm}$$

$0.05\ D_x = 0.05 \times 600 = 30 \text{ mm} > e_{x,\,min}$

$0.05\ D_y = 0.05 \times 450 = 22.5 \text{ mm} > e_{y,\,min}$

$\therefore$ Minimum eccentricities are less than 0.05 times lateral dimension in both sides

Hence it is designed as axially loaded column.

Step 2: Capacity of Column:

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

$A_g = 450 \times 600 = 270000\ mm^2$

$A_{sc} = 1256.63\ mm^2$

$\therefore\ P_u = (0.4 \times 25 \times 270000) + (0.67 \times 415 - 0.4 \times 25) \times 1256.63$

$= 3036.84 \times 10^3\ N$

$= 3036.84\ kN$

$\therefore$ Load carrying capacity of column under working loads $= \dfrac{3036.84}{1.5} = 2024.56\ kN$

05(c).

Sol: Given:

$F_{ck} = 20\ MPa;\ f_y = 415\ MPa$

LL = 14 kN/m; DL = 9.5 KM/m ; d/b = 2

Span (l) = 6 m

- Adopting $l/d = 10 \Rightarrow d = 600\ mm\ ;\ b = 300\ mm$
- Loadings:

LL = 14 kN/m; DL = 9.5 kN/m

Self wt. of beam = 25 × 0.3 × 0.6 kN/m

= 4.5 kN/m

$\Rightarrow$ Total load = 14 + 9.5 + 4.5 = 28 kN/m

$\Rightarrow$ Total factored $BM_{max} = 1.5 \times 28 \times \dfrac{6^2}{8}$

$(M_u) = 189\ kN.m$

- For a beam having $\dfrac{d}{b} = 2\ \&\ x_{u\ lim} = 0.48d$

$M_u = M_{R\ lim} = 0.36\ f_{ck}\ B.\ x_{u\ lim}\ (d - 0.42\ x_{u\ lim})$

$189 \times 10^6 = 0.36 f_{ck} \left(\dfrac{d}{2}\right)(0.48d)(d - 0.42 \times 0.48d)$

$\Rightarrow d^2\ (d - 0.2016\ d) = 109.375 \times 10^6$

$d^3 = 136.99 \times 10^6$

Solving d = 515.5 mm as per flexure criteria

Adopting d = 520 mm, effective cover = 50 mm

Total depth D = 520 + 50 = 570 mm < 600 mm (O.K)

$$b = \frac{d}{2} = 260 \text{ mm}$$

- A_{st} required

Since a balance section is designed $A_{st\ req} = \dfrac{M_u \text{ or } M_{R\lim}}{0.87 f_y (d - 0.47 x_u)}$

$= 1260.87 \text{ mm}^2$

$$P_t = \frac{A_{st}}{bd} \times 100 = 0.933\% \leq Pt_{lim} \text{ (O.K)}$$

05(d).

Sol: Ordinary Portland Cement (OPC) is the most widely used cement in the construction industry. The manufacturing of Ordinary Portland Cement is as follows.

1. The raw materials used for the manufacturing of Ordinary Portland Cement are Limestone and Clay.
2. There are two types of processes commonly employed for the grinding of the raw materials, they are dry and wet processes.
3. In the wet process, the limestone brought from the quarries is first crushed to smaller fragments. Then it is taken into a ball or tube mill where it is mixed with clay, and grounded to a fine consistency of slurry with the addition of water.
4. In the dry process, the raw materials are grounded in dry state itself.
5. This grinded raw material is stored in tanks under constant agitation and fed into huge firebrick lined rotary kilns.
6. By the time the material rolls down to the lower end of the rotary kiln, it undergoes a series of chemical reactions until finally, in the hottest part of the kiln where the temperature is of the order of 1500^{o}C, about 20 to 30 percent of the material gets fused.
7. The fused mass turns into nodular form of size 3 mm to 20 mm, known as clinker, has its own physical and chemical properties.
8. This clinker is mixed and crushed with 3 to 5 percent of gypsum and fed into a tube mill for final grinding. The finished product, known as Ordinary Portland cement, is packed in bags weighing 50 Kg per bag.

05(e).

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

In concrete mix design based on IS 10262:2009, workability based on slump cone test is used.

Slump Cone Test:

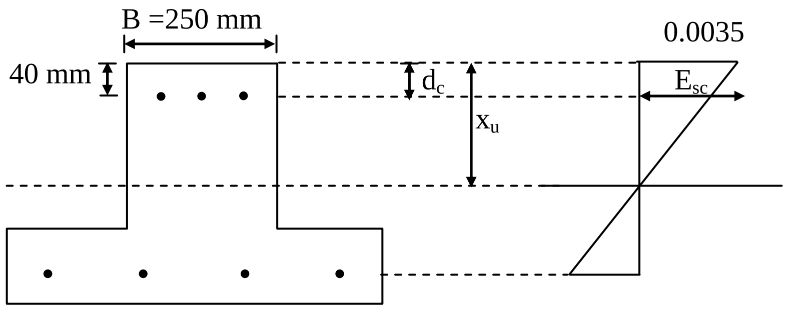
- This test is the most commonly used method for determining the degree of workability of concrete.
- This test can be used either in laboratory or at site of work.
- This test is not very useful for measuring the workability of very dry, very wet and stiff concrete mix.
- The apparatus used for this test is called Slump Cone.
- The dimensions of the slump cone are bottom diameter 200 mm, top diameter 100 mm and height 300 mm.
- The slump cone is filled with fresh concrete in four layers, with each layer compacted with 25 strokes using a tamping rod (Diameter = 16mm and length = 600mm).
- The concrete on top of the sump cone is struck off using a trowel.
- The slump cone is now removed immediately by raising it in a vertical direction.
- Now the concrete, also called as slump, is allowed to subside and the slump value is measured by determining the difference between the height of the mould and that of the highest point of the slump.
- The difference in height in mm is taken as slump value of concrete.
- More the slump value, better is the workability of the concrete.

06(a).

Sol: Given: $A_{sc} = 3\times\frac{\pi}{4}25^2 = 1472.62\ mm^2$

$$A_{st} = 4\times\frac{\pi}{4}25^2 = 1964\ mm^2$$

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



From above figure $E_{sc} = 0.0035\left(1-\frac{d_c}{x_u}\right)$

Stress in compression steel (f_{sc}) = $700\left(1-\frac{40}{x_u}\right)$

- Assuming x_u < 300 mm

Total compression = Total tension

$\Rightarrow 0.36\ f_{ck}.\ B.\ x_u + (f_{sc} - 0.45\ f_{ck})\ A_{sc} = 0.87\ f_y\ .A_{st}$

$$\Rightarrow 0.36 \times 25 \times 250 \times x_u + \left[\left\{700\left(1-\frac{40}{x_u}\right)-0.45\times 25\right\}\times 1472.6\right]$$

$$= 0.87 \times 415 \times 1964$$

$$\Rightarrow 0.36\times 25\times 250 x_u + \left[700-\frac{28000}{x_u}-11.25\right]\times 1472.6$$

$$= 0.87 \times 415 \times 1964$$

$\Rightarrow 2250\ x_u^2 + 305151.05\ x_u - 28000 = 0$

Solving x_u = 108.6 mm < 300 mm

Hence assumption is correct

- MR :

$MR = 0.36\ f_{ck}.\ B.\ x_u\ (d - 0.42\ x_u) + (f_{ck} - 0.45\ f_{ck})\ A_{sc}\ (d - d_c)$

For $x_u = 108.6$ mm

$f_{sc} = 335.4$ MPa

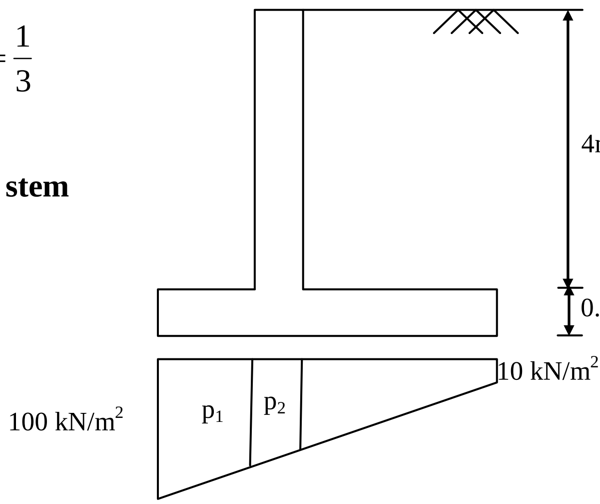
$\Rightarrow MR = 2250\ x_u\ (460 - 0.42\ x_u) + (335.45 - 11.25) \times 1472 \times (460 - 40)$

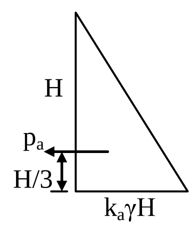
$= 301.8$ kN.m

06(b).

Sol: Active earth pressure coefficient

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$





(i) Checking the thickness of stem

Total soil pressure on stem

$$P_a = \frac{1}{2}(K_a \gamma H)H$$

$$= K_a \gamma \frac{H^2}{2} = \frac{1}{3} \times 18 \times \frac{4^2}{2}$$

$= 48$ kN

Bending moment at base

$$M = P_a \frac{H}{3} = 48 \times \frac{4}{3}$$

$= 64$ kN-m

Design B.M, $M_u = 1.5 \times 64$

$= 96$ kN-m

Effective depth of stem available $= 400 - 60$

$= 340$ mm

Effective depth of stem required

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{96 \times 10^6}{0.138 \times 20 \times 1000}}$$

$= 186$ mm < 340 mm $\therefore$ O.K

(ii) Reinforcement in stem:

(a) At bottom:

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 96\times 10^6}{20\times 1000\times 340^2}}\right]\times 1000\times 340 = 823.85\text{ mm}^2$$

Minimum steel:

$$\frac{0.12}{100}\times 1000\times 400 = 480\text{ mm}^2 < 823.85\text{ mm}^2 \therefore \text{ O.K}$$

(b) 50% of reinforcement is curtailed at a height h

$$\frac{h}{H} = \left[\frac{A_{st}'}{A_{st}}\times\frac{d'}{d}\right]^{1/3}$$

$$h = 4\left[\frac{1}{2}\times 1\right]^{1/3} = 3.17\text{ m}$$

But actual cut off point will be at d (or) 12ϕ which ever is max from theoretical point of cutoff

3.17 – 0.34 = 2.83 m from top

Spacing of main steel

$$S = 1000\,\frac{a_{st}}{A_{st}} = \frac{1000\times\frac{\pi}{4}\times 16^2}{823.85} = 244\text{ mm}$$

Provide 16 mmϕ @ 240 mm c/c at base

16 mm ϕ @ 480 mm c/c at above 1.47 m from base

Spacing of distribution steel

$$S = \frac{1000\times\frac{\pi}{4}\times 10^2}{480} = 163.5\text{ mm}$$

Provide 10 mm dia @ 160 mm c/c

(iii) Shear Force at Critical Section

Pressure at the junction between stem and heel slab

$$p_2 = 10 + \frac{1.6}{3}(100 - 10) = 58 \text{ kN/m}^2$$

w_1 = weight of soil above heel slab

w_2 = weight of heel slab

w_3 = total soil pressure below heel slab

$w_1 = 1.6 \times 4 \times 1 \times 18 = 115.2$ kN ($\downarrow$)

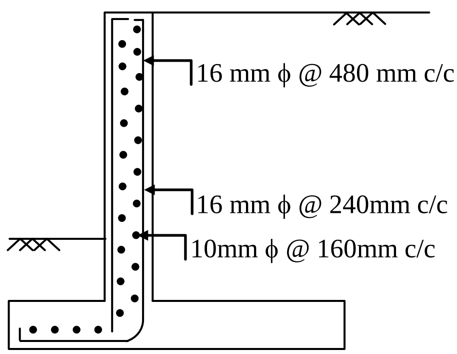
$w_2 = 1.6 \times 0.3 \times 1 \times 25 = 12$ kN ($\downarrow$)

$w_3 = \left[\frac{10+58}{2}\right]1.6 = 54.4$ kN ($\uparrow$) (upward pressure)

S.F, V = 115.2 + 12 – 54.4

= 72.8 kN

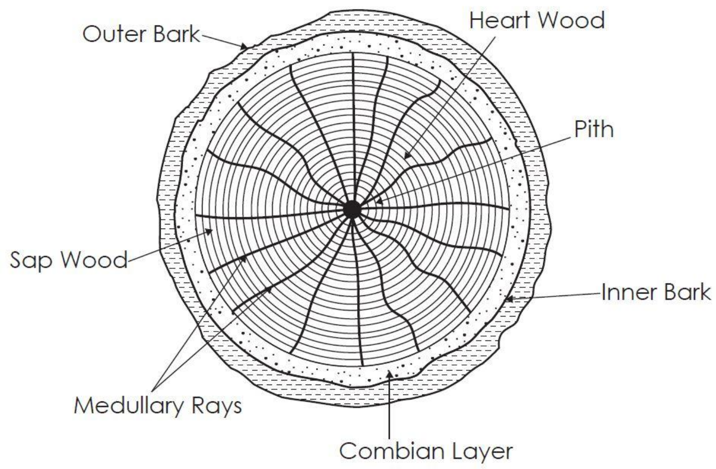
Sketch:



06(c).

(i)

Sol: The section of a typical exogenous tree is as follows:



1. **Pith:** This is the inner most part or core having soft tissues. It is found near about the center of log or a timber. Its size varies from 1.5 mm to 10 mm in dia.

2. **Heart Wood:** The inner part of the tree surrounding the pith is called Heart Wood. It imparts rigidity to tree and provides strong and durable timber for various engineering purposes.

3. **Sap Wood:** It is the area between the heart wood and cambium layer. It contains living cells and takes active part in growth of tree. It is light in colour and weight.

4. **Cambium Layer:** It is the thin layer between sap wood and inner bark. It indicates sap which has yet to be converted into sap wood.

5. **Inner Bark:** It is the Inner skin or layer covering the cambium layer. It gives protection to the cambium layer.

6. **Outer Bark:** It is the outer most protective layer of the tree. It consists of cells of wood fiber and it is known as cortex.

7. **Medullary Rays:** These are thin radial fibers extending from pith to cambium layer. It holds together the annual rings of heart wood and sapwood. One ring is added every year, which decides the age of the tree.

06(c).

(ii)

Sol: The physical and mechanical properties of Aluminum with respect to other metals are as follows:

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.

Industrial Applications of Aluminium:

1. Aluminium being very light, has a low handling and transportation cost, further it also possesses ease of fabrication and assembly. Aluminium structures have a low dead weight and a low maintenance cost.
2. Because of its high ductility, aluminium is highly suitable at extremely low temperatures. This characteristic of aluminium is particularly useful for outer space and Antarctica region expeditions.
3. A finished aluminium surface is very smooth and attractive and is used for decorative structures.
4. Aluminium has high scrap value compared to other metals as it is very cheap to recycle.

07(a).

Sol: Given:

Reactions at support:

BM due to DL = 10 kN-m/m

BM due to LL = 15 kN-m/m

SF at support = 15 kN/m

M25 grade concrete, Fe 415 steel

Effective length of slab = clear span + $\frac{1}{2}$ (width of support)

$$= 1500 + \frac{1}{2}(400) = 1700 \text{ mm}$$

Step 1: Design reactions: (Assuming width = 1000 mm)

Total bending moment at support

$$= 10 + 15 = 25 \text{ kN-m}$$

Factored bending $M_u = 1.5 \times 25 = 37.5$ kN-m

Factored shear force at support = 1.5 × 15 = 22.5 kN

Step 2: Depth of slab required

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{37.5 \times 10^6}{0.138 \times 25 \times 1000}}$$

$$= 104.25 \text{ mm}$$

$\therefore$ Provide 110 mm as effective depth of slab

Given : Effective cover = 30 mm

$\therefore$ total depth = 110 + 30 = 140 mm

Step 3:

(a) Reinforcement required:

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

$$= 0.5\times\frac{25}{415}\left[1-\sqrt{1-\frac{4.6\times 37.5\times 10^6}{25\times 1000\times 110^2}}\right]\times 1000\times 110$$

$= 1141.23\ mm^2$

Minimum steel = 0.12% bD = $\frac{0.12}{100}\times 1000\times 140 = 168\ mm^2$

[Provide A_{st} = 1141.23 mm^2]

Using 12 mm diameter bars : spacing required $= \frac{1000}{1141.23}\times\left(\frac{\pi}{4}\times 12^2\right)$

= 99.10 mm

Check :

Spacing $\not> 3d = 3\times 110 = 330$ mm

$\not>$ 300 mm

$\therefore$ Provide 12 mm ϕ bars @ 90 mm c/c/

(b) Distribution bars:

$$A_{st} = 0.12\% bD = 168\ mm^2$$

Using 12 mm dia bars; spacing required $= \frac{1000}{168}\times\frac{\pi}{4}\times 12^2$

= 673.2 mm

Check:

Spacing $\not> 5d = 5\times 110$ = 550 mm

$\not>$ 450 mm

$\therefore$ Provide 12 mm bars at 450 mm c/c

Step 4:

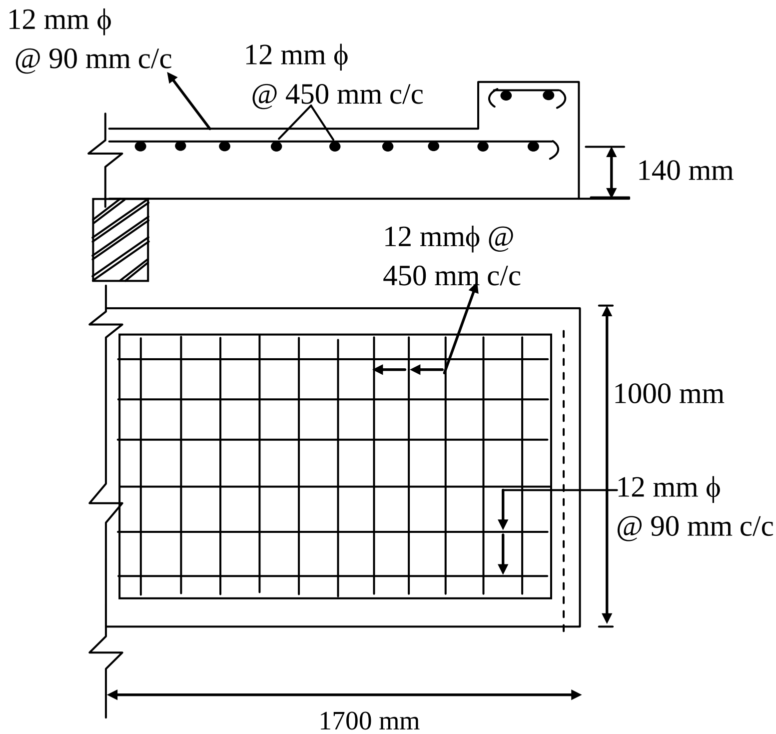
Check for shear:

Nominal shear stress = $\tau_{vu} = \frac{V_u}{bd} = \frac{22.5\times1000}{1000\times110} = 0.20$ MPa

Shear strength of concrete = τ_c = 0.25 MPa > τ_{vu}

Hence safe in shear.

Design details:



07(b).

Sol:

Given

Span = 10 m; live load = 20 kN/m

b = 300 mm; loss in prestress = 15%

f_c = 16 MPa; f_S = 1500 MPa, f_{ct} = 0

depth d = 600 mm

self weight = 25 × 0.3 × 0.6 = 4.5 kN/m

Bending moment due to dead load = $\frac{w_d \ell^2}{8}$

$M_d = 4.5 \times \frac{10^2}{8} = 56.25$ kNm

Bending moment due to live load M_e = $20 \times \frac{10^2}{8}$ = 250 kNm

At transfer:

Stress at top = $\frac{P}{A} - \frac{Pe}{z} + \frac{M_d}{z} = 0 \quad \rightarrow (1)$

Stress at bottom = $\frac{P}{A} + \frac{Pe}{z} - \frac{M_d}{z} = f_c \quad \rightarrow (2)$

At service:

Stress at top = $\frac{\eta P}{A} - \frac{\eta Pe}{z} + \left(\frac{M_d + M_\ell}{z}\right) = f_c \quad \rightarrow (3)$

Stress at bottom = $\frac{\eta P}{A} + \frac{\eta Pe}{z} - \left(\frac{M_d + M_\ell}{z}\right) = 0 \quad \rightarrow (4)$

Eq (3) – k (Eq (1))

$\Rightarrow \frac{M_d}{z} + \frac{M_\ell}{z} - \frac{\eta M_d}{z} = f_c$

$\Rightarrow \quad z = \frac{M_d(1-\eta) + M_\ell}{f_c}$ [η = 1 – 0.15 = 0.85]

$= \frac{56.25(1-0.85)\times 10^6 + 250 \times 10^6}{16}$

= 16.15×10^6 mm^3

Depth required = $\sqrt{\frac{62}{8}} = \sqrt{\frac{6 \times 16.15 \times 10^6}{300}}$

= 568.37 mm

Provided depth = 600 mm

Hence safe.

Step 2:

Area = $300 \times 600 = 18 \times 10^4$ mm^2

From e q (3) + eq (4)

$$\Rightarrow \frac{2\eta P}{A} = f_c$$

$$\Rightarrow P = \frac{f_c A}{2\eta} = \frac{16\times18\times10^4}{2\times0.85} = 1694.117\times10^3\,N$$

Step 3:

$$\text{Area of steel} = \frac{P}{f_s} = \frac{1694.117\times10^3}{1500} = 1129.41\ \text{mm}^2$$

Step 4:

$$\text{Eq (1)} \times k = \frac{\eta P}{A} - \frac{\eta P_e}{z} + \frac{\eta M_d}{z} = 0$$

$$\Rightarrow \frac{\eta P_e}{z} = \frac{\eta P}{A} + \frac{\eta M_d}{z}$$

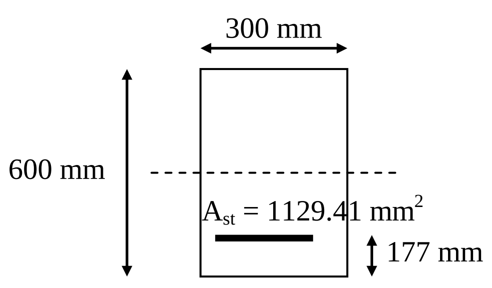
$$\text{Eq (4)} \Rightarrow \frac{\eta Pe}{z} = -\frac{\eta p}{A} + \left(\frac{M_d + M_\ell}{z}\right)$$

$$\text{Adding} \Rightarrow \frac{2\,\eta Pe}{z} = \frac{M_d(1+\eta) + M_\ell}{z}$$

$$\Rightarrow e = \frac{M_d(1+\eta) + M_\ell}{2\,k\,P}$$

$$\Rightarrow e = \frac{(56.25(1+0.85)+250)\times10^6}{2\times0.85\times1694.11\times10^3} = 122.93\ \text{mm}$$

$$\therefore \text{location of cable from bottom} = \frac{600}{2} - 122.93 = 177\ \text{mm}$$



07 (c).

(i)

Sol: Admixtures are substances which are mixed with concrete to improve certain desired characteristics. Admixtures are broadly divided into two types – Chemical and Mineral Admixtures.

Plasticizers:

- These are the chemical admixtures which improve the workability of concrete without increasing the water cement ratio.
- These can reduce water requirement in a range of 5 to 15% without affecting the workability.
- These also have some retarding tendency.
- These are used for the producing of higher grades of concrete.
- Most commonly used plasticizers are Hydro carbonic acid and Ligno sulphates.

Mineral Admixtures:

1. Fly Ash.
2. Ground Granulated Blast Furnace Slag.
3. Burnt Rice Husk.
4. Silica Fume.
5. Surkhi.
6. Metakoline.

07 (c).

(ii)

Sol: The following are the different tests conducted on bricks:

1. Compressive Strength Test.
2. Water Absorption Test.
3. Saturation Factor Test.
4. Efflorescence Test.
5. Warpage Test.
6. Soundness Test.

The test which determines the freezing and thawing resistance of a brick is Saturation Factor test. This test has two components.

1. **Cold water test:** The brick is oven dried at a temperature of 105 to115^{O}C till it attains a constant mass (M_1). It is then immersed in water which is at a temperature of 27 ± 2^{O}C for 24 hours. The brick is then removed and weighed (M_2) and the water absorbed is calculated.

   24 hour water absorption by weight (W_{24}) = $(M_2 – M_1)/M_1$ x 100.

2 **Boiling water test:** The dried brick specimen is taken and is immersed in a water bath which is heated to boiling in one hour and boiled continuously for five hours. The water is then allowed to cool to 27^{o}C by natural loss of heat for 16 to 19 hours. The specimen is again weighed (M_3) and the water absorbed is calculated.

   5 hour boiling water absorption by weight (W_5) = (M3 – M1)/M1 x 100.

   Now the Saturation Factor is calculated as W_{24}/W_5

08(a).

Sol: M25 $\rightarrow f_{ck} = 25$ N/mm^2

Fe-415 $\rightarrow f_y = 415$ N/mm^2

(i) Effective Depth of Beam:

d = D – effective cover

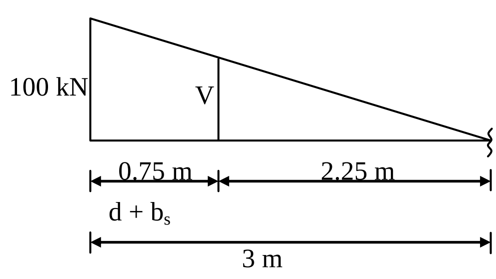
= 500 – 50 = 450 mm

(ii) Design Shear Force at Critical Section:

Assume width of support b_s = 300 mm

$$V = \frac{2.25}{3} \times 100 = 75 \text{ kN}$$

$V_u = 1.5V = 1.5 \times 75 = 112.5$ kN



(iii) Nominal Shear Stress:

$$\tau_v = \frac{V_u}{bd} = \frac{112.5 \times 10^3}{300 \times 450} = 0.83 \text{ N/mm}^2$$

$\tau_v < \tau_{c\,max}$ $\therefore$ O.K

(iv) Shear Strength of Concrete:

Available steel at critical section 3-20ϕ

$$p_t = 100\frac{A_{st}}{bd}$$

$$= \frac{100\times 3\times\frac{\pi}{4}\times 20^2}{300\times 450} = 0.7\%$$

$$\tau_c = 0.49 + \frac{0.57-0.49}{0.75-0.50}\times(0.7-0.5)$$

$$= 0.55\ \text{N/mm}^2$$

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

Hence design the shear reinforcement use vertical stirrups

(v) Design Shear Force of Stirrups:

$V_{us} = V_u - \tau_c bd$

$= 112.5 \times 10^3 - 0.55 \times 300 \times 450$

$= 38.25$ kN

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

$$38.25\times 10^3 = 0.87\times 415\times 2\times\frac{\pi}{4}\times 8^2\times\frac{450}{S_v}$$

$S_v = 427$ mm > 300 mm $\therefore$ Not safe

(vi) Minimum Shear Reinforcement:

$$\frac{A_{sv}}{b.S_v}\geq\frac{0.4}{0.87f_y}$$

$$\frac{2\times\frac{\pi}{4}\times 8^2}{300\times S_v}\geq\frac{0.4}{0.87\times 415}$$

$S_v = 302$ mm

(vii) Check with Maximum Spacing:

$$\left.\begin{array}{l}\text{(i) } 0.75d = 0.75 \times 450 = 337.5\,\text{mm} \\ \text{(ii) Cal } S_v = 437 \text{ mm} \\ \text{(iii) } S_{v\min} = 302 \text{ mm} \\ \text{(iv) } 300 \text{ mm}\end{array}\right\}\text{Smaller}$$

Provide 2 legged 8 mm dia @ 300 mm c/c

08(b).

(i)

Sol: The compounds present in cement are called Bogue Compounds. The properties of these compounds are as follows:

1. Tricalcium Silicate (C_3S): This compound is supposed to have the best cementing properties among the different compounds of cement. It has high rate of hydrolysis and is responsible for 7 day hardness and strength of cement. However, high C_3S content leads to high heat of hydration. It has a heat of hydration of about 500 J/gm.

2. Dicalcium Silicate (C_2S): This compound hydrates and hardens slowly and takes longer time to contribute to the strength compared to C_3S. But in a period of one year, its contribution to the strength and hardness is proportionately equal to C_3S. It also imparts resistance to chemical attack, as the hydration of this compound releases less $Ca(OH)_2$ compared to C_3S. However, high C_2S content reduces the rate of early strength gain, decreases resistance to freezing and thawing and decreases the heat of hydration. It has a heat of hydration of about 260 J/gm.

3. Tricalcium Aluminate (C_3A): This compound rapidly reacts with water and may lead to immediate stiffening of the cement paste, known as flash set. This action is regulated by the addition of 2 – 3 % gypsum at the time of grinding of clinkers. It is responsible for the initial set, a greater tendency to volume changes causing cracking and has high heat of hydration. Raising the C_3A content reduces the setting time, weakens resistance to sulphate attack and lowers the ultimate strength. It has a heat of hydration of about 865 J/gm.

4. Tetracalcium Alumino Ferrite (C_4AF): This compound has poor cementing value and is also responsible for flash set, but to a lesser extent compared to C_3A. It generates less heat of hydration compared to C_3A. It has a heat of hydration of about 420 J/gm.

08(b).

(ii)

Sol: The process of drying timber or reducing moisture of sap present in a freshly felled timbers under more or less controlled conditions is called 'seasoning of timber'. A well-seasoned piece of wood should have 10% to 12% of moisture equal to atmospheric humidity of a place.

Artificial Seasoning:

1. **Boiling:**
   In this method, the Timber is immersed in boiling water or exposed to the action of steam. This method is a quick process of seasoning the timber. The timber seasoned by this method becomes brittle and easy to break.
2. **Chemical seasoning:**
   An aqueous solution of chemical salts like common salt or urea have low vapour pressure. If the outer layers of timber are treated with such chemicals the vapour pressure will reduce and a vapour pressure gradient is created. The interior of timber, containing no salts, retains its original vapour pressure and therefore tends to dry rapidly.
   Corrosive effect of common salt is a drawback of this method.
3. **Electrical seasoning:**
   The logs of wood are placed in such a way that their two ends touch the electrodes. Electricity is passed through this setup. Wood being a bad conductor of electricity, resists the flow of electricity. In this process heat is generated which results in drying of timber. The drawback is that the wood may split due to overheating.
4. **Kiln Seasoning**:
   Carried out in an air tight chamber under controlled conditions of circulating air, relative humidity and temperature.
   Desired degree of moisture content in attained.
   Time required 12-20 days.

5. **Water Seasoning:**

The timbers like bamboo are placed in the flowing water. The flow of water takes away the sap wood. After one week the bamboo poles are removed from water and then dried out under shade. Timber loses its elasticity and becomes brittle as it is losing sap wood completely.

08(c).

(i)

Sol: **Thermo-plastics:** 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 plastics 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.

Examples: Polyethylene, polyvinyl chloride, polystyrene, acrylic, nylon and Teflon.

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.

Examples: Epoxy resin, Melamine Formaldehyde, Polyester resin and Urea Formaldehyde.

08(c).

(ii)

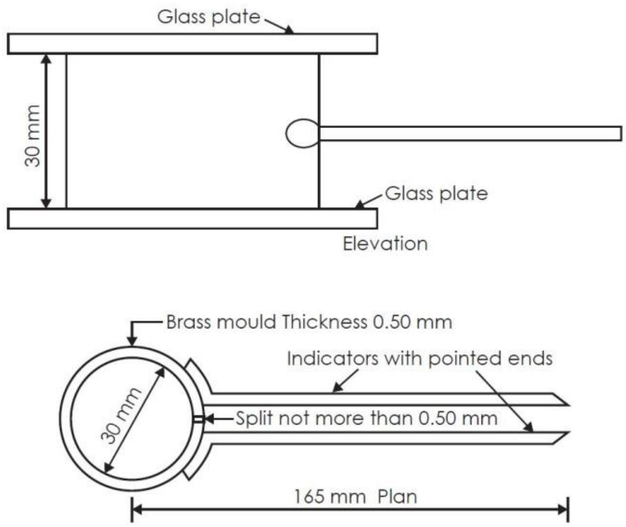
Sol: The following are the tests which are done in the field to assess the quality of the cement.

- It should be dark grey in colour with a slight greenish shade.
- It should be free from lumps.
- It should feel smooth when a small quantity of it is rubbed between the fingers.
- It should feel cool when hand is inserted in a bag or a heap of cement.
- When a small quantity of cement is thrown in water, it should sink and not float on the water.

The test used to determine the soundness of cement due to free lime only is Le Chatelier's test.

Test Apparatus:

The apparatus consists of a small split cylinder made of spring brass which is 30 mm in diameter and 30 mm in height. The split should not be more than 0.5 mm wide. Two indicator arms, which extend 165 mm from the center of the cylinder, are attached on either side of the split.



Test Procedure:

- A cement paste is prepared by adding water 0.78P% by weight of cement.
- The cement paste is filled into the split cylinder and kept on a glass plate.
- The top of the cylinder is covered with another glass plate.
- This assembly is completely immersed in water which is at a temperature of $27 \pm 2^{O}C$ and kept there for 24 hours.
- The assembly is taken out after 24 hours and the distance between the two indicator arms is measured.
- The assembly is now kept inside a water bath which is brought to boiling point in 25 to 30 minutes and kept boiling for 3 hours.
- The assembly is taken out of the water bath and allowed to cool and the distance between the two indicator arms is measured.
- The difference between the two measurements indicates the expansion of cement due to the presence of free lime in it.
- The difference between the two measurements should not be more than 10 mm for Ordinary Portland Cement.