paper solution

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Lonere - Raigad.
Winter Semester Examination - December - 2017
Subject - Advanced pre-stressed Concrete
Code - CVSE - E2 - 01.

prof. M.Z. shaikh [9762551333]

Q.1.

[a] what are the basic concept applied to Explain and Analyze prestressed concrete. Explain any one. [4-Marks]

⇒ Three diffrent concepts may be applied to Explain and analyse the Basic behaviour of prestressed concrete.

1) stress concept
2) strength concept
3) Balanced load concept.

1) stress concept : prestressing to transform concrete into an elastic material. This concept is credited to Eugene Freyssinet who visualised prestressed concrete as essentially concrete which is transformed from a brittle material into an elastic material by compression given to it. if an ordinary concrete, wheather plain or reinforced is subjected to only compressive stresses it behaves as a perfect elastic material because no Tension cracks are there. But it is subjected to Flexural stressed Some portion of it will be in Tension resulting in Tension cracks, the material under such circumstance no longer remains elastic.

In prestressed concrete, on the otherhand concrete is visualised as being subjected to two system of Forces. a) internal prestress, which is compressive and External load causing tensile stresses. The Tensile stresses are Counterbalanced by the compressive stress due to prestress, with the result that Final stress in the extreme

Fibre is either compressive or zero. Due to absence of final tensile stress, no tension cracks would be there in concrete. and it will thus be transformed from brittle to elastic material.

To illustrate this point, let us considered two cases

i) concentrically tendon (steel reinforcement)

ii) eccentric tendon.

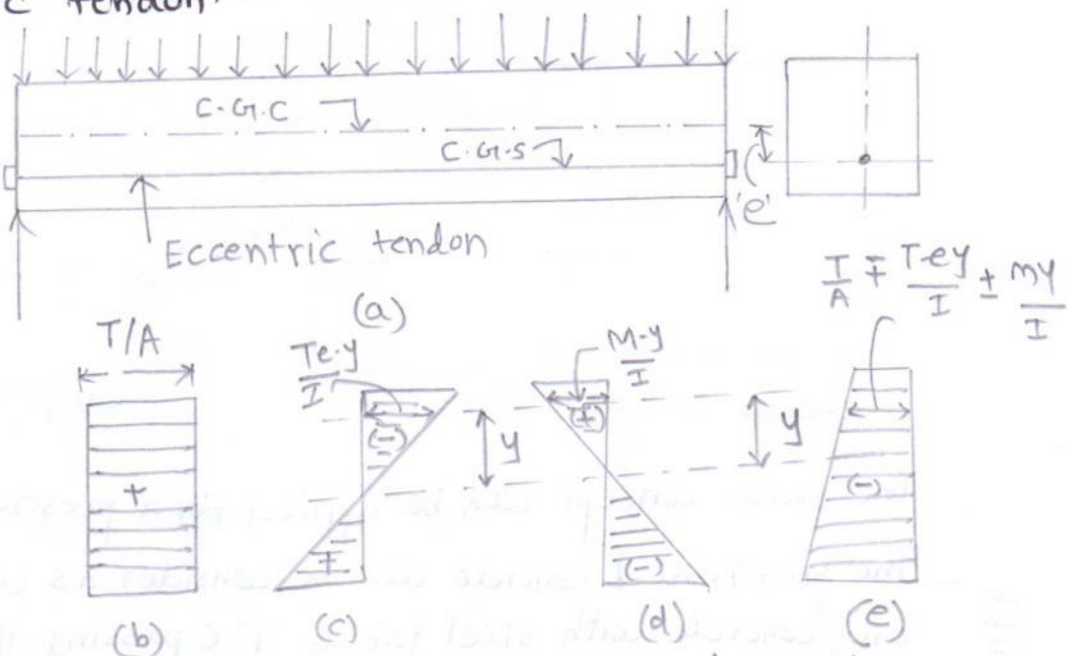

Figure- Eccentrically prestressed section.

The above figure shows a concentrically prestressed concrete beam, due to prestress force T in tendon, a uniform comp. stress $= T/A$ will be induced in concrete. if the beam is subjected to a moment M due to external load, inclusive of its own weight, the stress at any point will be $\frac{M}{I} \cdot y$

where,

y = The distance of the point from the centroidal axis

and I = moment of Inertia of the section.

The final stress at any point will be given by $F = \frac{T}{A} \pm \frac{M \cdot y}{I}$

in figure due to stress force T in tendon applied eccentrically the moment produced due to prestress will be $T \cdot e$. Hence the stress f' due to prestress at any point will be

$$f' = \frac{T}{A} \mp \frac{T \cdot e \cdot y}{I}$$

if M is the external moment, the final stress at any point

$$f = \frac{T}{A} \mp \frac{T \cdot e \cdot y}{I} \pm \frac{M \cdot y}{I}$$

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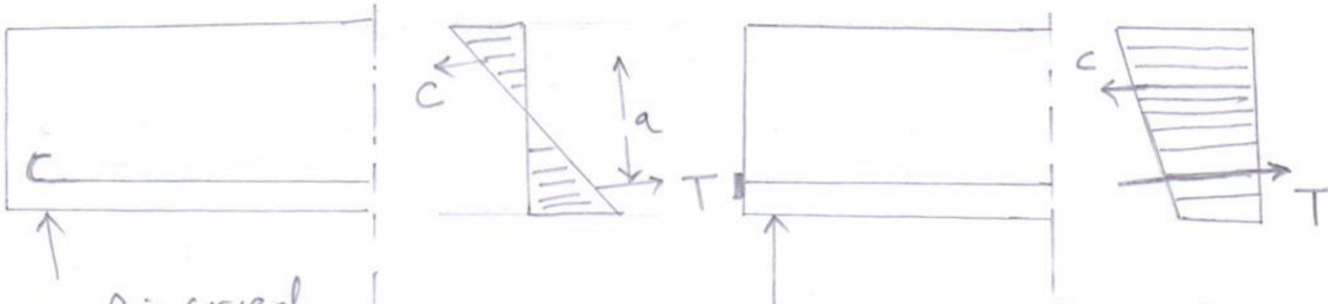
Q.1 [a] continue

2) Strength concept : prestressing For combination of High strength steel and concrete.

In reinforced concrete, steel takes tension while concrete takes compressive stress and the couple Formed by the resultant compressive force C and tensile force T (where C = T) resist the external moment. as shown in below figure.



(a) Reinforced Concrete section

(b) prestressed concrete

The same concept can be applied For a prestressed concrete section. The prestressed concrete can be consider as combination of steel and concrete. with steel taking T (passing through the tendon) and concrete taking compression C [passing through the C-G of the of the stress distribution shown shaded in Fig(a). So that the two material Form a resisting couple to resist the external moment. This concept is well utilised to determine the ultimate strength of prestressed concrete beam.

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Q.1 (b) 8 - Marks.

$\Rightarrow$ Solution : $P = 540\,kN$, $A = (250 \times 300) = 75 \times 10^3\,mm^2$

$e = 60\,mm$, $Z = \left[\frac{250 \times 300^2}{6}\right] = 375 \times 10^4\,mm^3$

at the centre of the span $M_q = (0.25 \times 68 \times 3) = 51\,kNm$

at the quarter span $M_q = (0.125 \times 68 \times 3) = 25.5\,kNm$

stresses due to prestressing force

$$\frac{P}{A} = \left[\frac{54 \times 10^4}{75 \times 10^3}\right] = 7.2\,N/mm^2$$

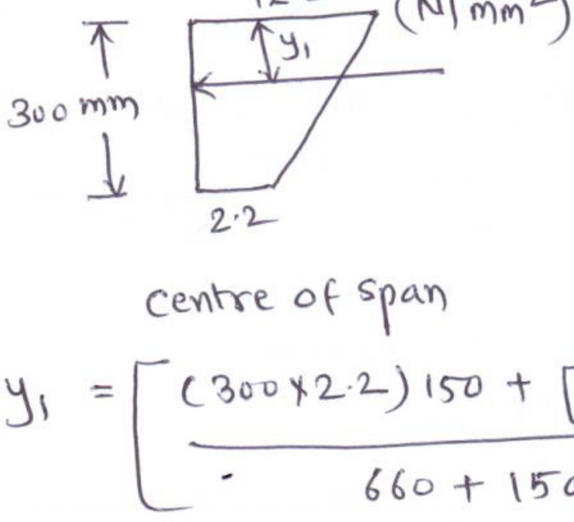
$$\frac{Pe}{Z} = \left[\frac{54 \times 10^4 \times 60}{357 \times 10^4}\right] = 8.6\,N/mm^2$$

stresses due to external loads

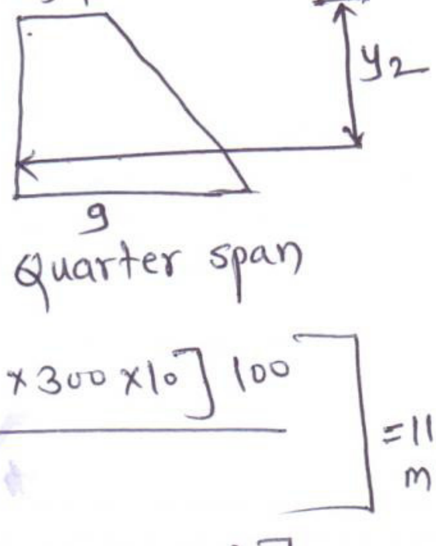
At the centre of span $\left[\frac{M_q}{Z}\right] = \left[\frac{51 \times 10^6}{375 \times 10^4}\right] = 13.6\,N/mm^2$

At the quarter of span $\left[\frac{M_q}{Z}\right] = \left[\frac{25.5 \times 10^6}{375 \times 10^4}\right] = 6.8\,N/mm^2$

The position of the resultant thrust from top fibre of the beam is can be obtained from



centre of span



Quarter span

$$y_1 = \left[\frac{(300 \times 2.2)150 + \left[\frac{1}{2} \times 300 \times 10\right]100}{660 + 1500}\right] = 115\,mm$$

$$y_2 = \left[\frac{(5.4 \times 300)150 + \left(\frac{1}{2} \times 300 \times 3.6\right)200}{1620 + 540}\right] = 162\,mm$$

1.1 [b]

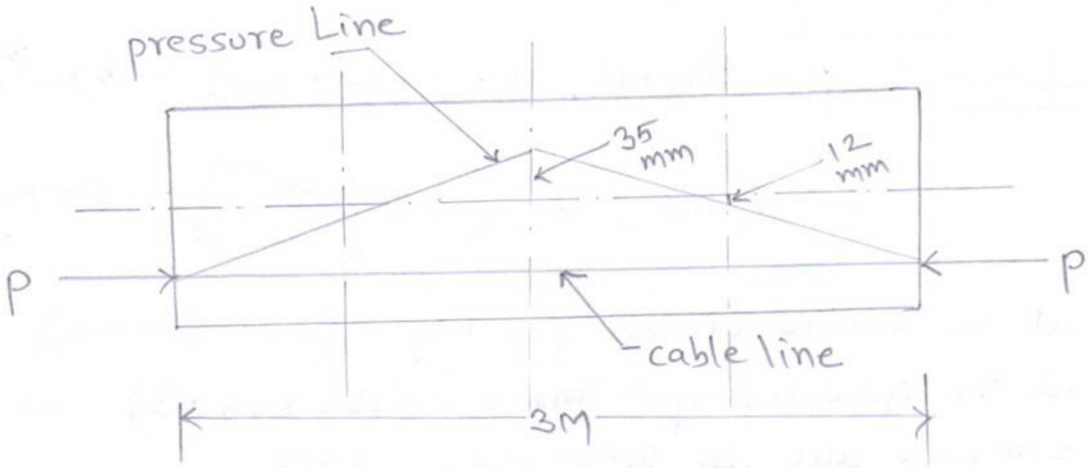


Figure – Location of pressure line in the prestressed beam.

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Q.2 [a] Magnel's method of End block design [4 Marks]

⇒ [a] Solution : In magnel's method, the end block is considered as a deep beam subjected to concentrated loads due to anchorages on one side and to Normal and tangential distributed loads from the linear direct stress and shear stress distribution from the other side.

The Force acting on the end block and the stresses acting on any other point on the horizontal axis parallel to the beam are shown in figure below.

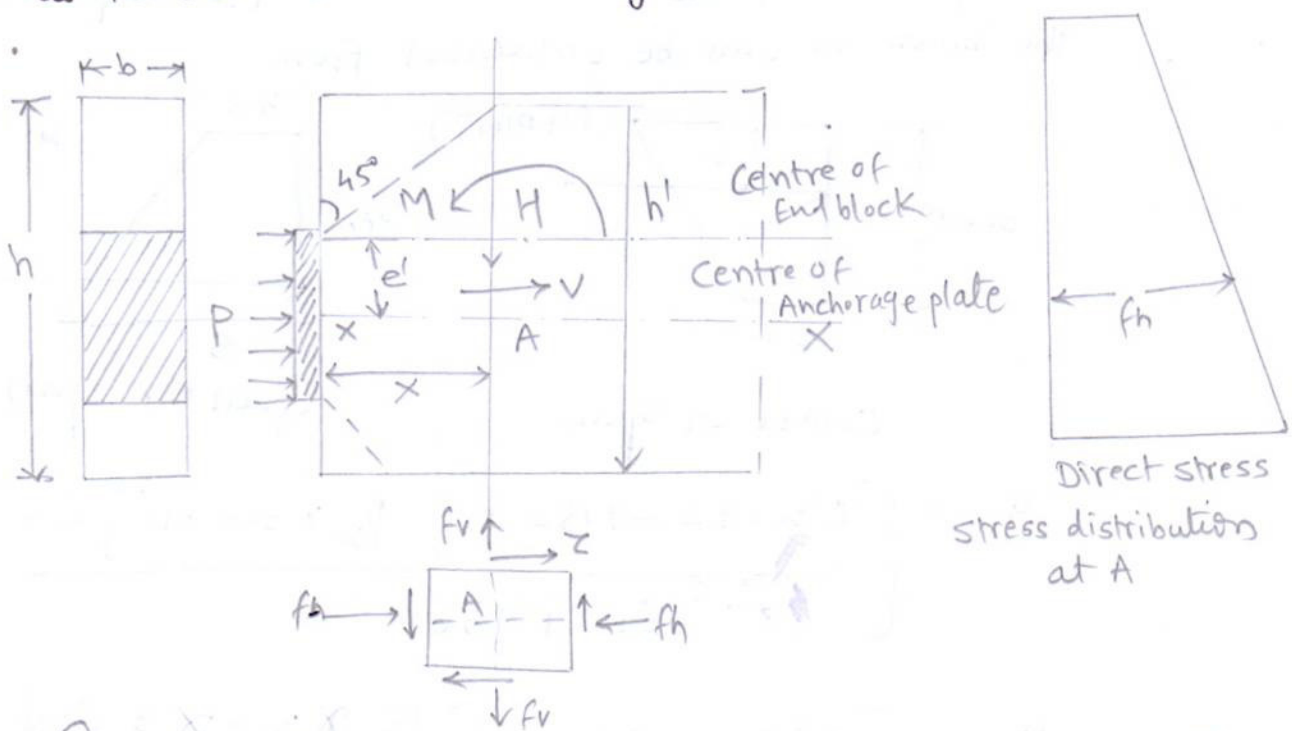

figure – Force Acting on the End block

Q.2
[9]

Where,

M = bending moment

H = direct Force (vertical) (+ve)

V = shear Force (Horizontal)

fv = vertical stress

fh = direct stress at point A. (shown in figure)

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The stress distribution across the section can be approximated by the following equations:

$$F_v = K_1\left[\frac{M}{bh^2}\right] + K_2\left[\frac{H}{bh}\right]$$

$$\tau = K_3\left[\frac{V}{bh}\right]$$

$$F_h = \frac{P}{bh}\left[1 + 12\frac{e'^2}{h'^2}\right]$$

Where K_1, K_2 & K_3 are constants.

The direct stress Fh is computed by assuming that the Concentrated load at 45° & considering the depth of the section intercepted between the dispersion lines at the required point on the horizontal axis.

The principal stresses acting at the point are computed by the general equations

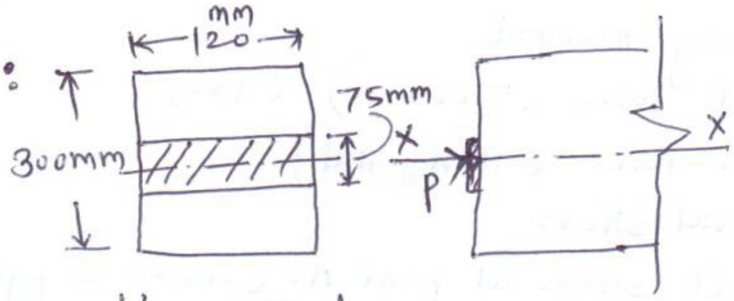
$$f_{max} \text{ or } f_{min} = \left[\frac{F_v + F_h}{2}\right] \pm \frac{1}{2}\sqrt{(F_h - F_v)^2 + 4\tau^2}$$

$$\tan 2\theta = \left[\frac{2\tau}{F_v - f_h}\right]$$

The bursting tension is computed from the distribution of principal tensile stress on the required axis and suitable reinforcements are designed to take up this tension.

Q.2 [b] [8 - Marks]

⇒ Solution :



[a] Magnel's method

$P = 250\ kN$

Direct stress $= f_h = \left(\frac{250 \times 10^3}{120 \times 300}\right) = 6.94\ N/mm^2$

vertical stress fv and principal stress are critical at $x = 0.5h$

At $\left(\frac{x}{h}\right) = 0.5$ Thus from Table-01

$K_1 = -5$

$K_2 = 2.00$

$K_3 = 1.25$

$$M = \left[(6.94 \times 150 \times 120)\left(\frac{150}{2}\right) - \left(\frac{250 \times 10^3}{2}\right)\left(\frac{75}{4}\right)\right]$$

$= 7026250\ N \cdot mm$

$V = 0$

$H = 0$

$$f_v = -5\left(\frac{7026250}{120 \times 300^2}\right) = -3.25\ N/mm^2$$

$f_h = +6.94\ N/mm^2$

$$\therefore f_{min} = \left(\frac{6.94 - 3.25}{2}\right) - \frac{1}{2}\sqrt{(6.94+3.25)^2 + 0}$$

$= -3.25\ N/mm^2$ acting at 150 mm

Tension

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Q.2 [b] Guyon Method

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$P = 250\ kN$

$2y_{p0} = 75\ mm$

$2y_0 = 300\ mm$

$\therefore$ Distribution ratio $= \left(\frac{y_{p0}}{y_0}\right) = \left(\frac{75}{300}\right) = 0.25$

$\therefore$ position of max. stress $= 0.33\,(2y_0) = 100\ mm$

From table-02

maximum tensile stress $= 0.345\left(\frac{P}{A}\right)$

$= \left(0.345 \times \frac{250 \times 10^3}{120 \times 300}\right)$

$= 2.39\ N/mm^2$

Q.3

Solution : [12 Marks]

span $= 12\ m$

Super imposed load $= 15\ kN/m$

Bending moment due to super imposed load $= \frac{15 \times 12^2}{8}$
$= 270\ kNm$

provide For 20% additional moment due to self weight $= 54\ kNm$

$\therefore$ Total bending moment $= 324\ kNm$

Adopting a depth of beam of $\frac{L}{16} = \frac{12}{16} = 0.75\ m$

prestressing force $= \frac{324 \times 10^6}{0.61 \times 750} = 708196.9\ N.$

Area of concrete required $= \left[\frac{708196.9}{0.85}\right] \times \frac{2}{14}$
$= 119024.6\ sq\,mm$

Q.3

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Since it is precast unit a symmetrical I section is assumed with top and bottom width of 300 mm and Flange depth of 150 mm and web width 120 mm.

$$\text{Area} = 144000\ mm^2$$

$$\text{Moment of Inertia} = 300 \times \frac{750^3}{12} - 180 \times \frac{450^3}{12}$$

$$= 1.0547 \times 10^{10} - 0.1367 \times 10^{10}$$

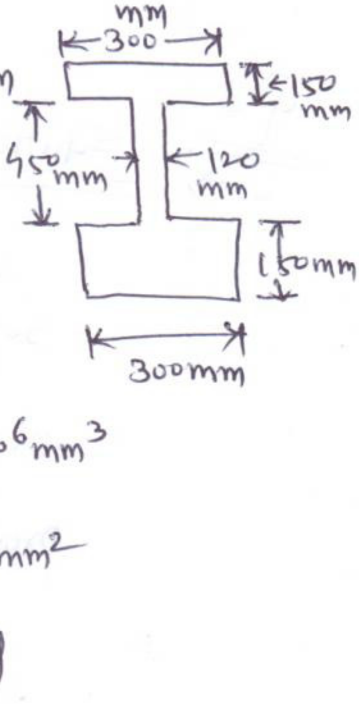
$$= 0.918 \times 10^{10}\ mm^4$$

$$Z_b = Z_t = \frac{0.918 \times 10^{10}}{375} = 24.48 \times 10^6\ mm^3$$

$$f_{cbrange} = 0.85 \times 14 - (-1.4) = 13.3\ N/mm^2$$

$$f_{ctrange} = 14 - 0.85(-1.4) = 15.19\ N/mm^2$$

$$M_g = \left[\frac{144000}{10^6}\right] 24 \times \frac{12^2}{8} = 62.208\ kNm$$



$$Z_b = \frac{[270 + 0.15 \times 62.208]}{13.3} = 21 \times 10^6\ mm^3$$

$$Z_t = \frac{270 + 0.15 \times 62.208}{15.19} = 18.39 \times 10^6\ mm^2$$

Z provided is more than what is required
Hence section adopted is safe.

prestressing force required at section load stage

$$P_\infty = A\left[\frac{f_{cbp} Z_b + f_{ctp} \cdot Z_t}{Z_t + Z_b}\right]$$

$$f_{cbp} = \frac{-1.4}{0.85} + \frac{(270 + 62.208) \times 10^6}{0.85 \times 24.48 \times 10^6}$$

$$= -1.647 + 15.966 = 14.32\ N/mm^2$$

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Q.3

$f_{ctb} = -1.4 - \frac{62.28}{24.48} = -1.4 - 2.501 = -3.941\ N/mm^2$

At transfer stage with dead load effects the stress will be come less 14 N/mm² at bottom and less than 1.4 N/mm² tension at top.

$$P_\infty = 144 \times 10^3 \left[\frac{(14.32 - 3.94)\, 24.48 \times 10^6}{2 \times 24.48 \times 10^6} \right]$$

$$= 747.288\ kN.$$

$$e = \frac{24.48 \times 24.48 \times 10^{12}}{144 \times 10^3} \left[\frac{14.32 - (-3.941)}{24.48 \times 10^6 \times (14.32 - 3.941)} \right]$$

$$e = 298.6\ mm$$

provide 300 mm eccentricity with a centre cover of 75mm for steel 747.288 kN of force can be got 24 nos of 7mm in 2 cable (12×7) stressed to 1000 N/mm² initially will give a force of 92.3161 kN. with a % loss of 15%. the final pressers will be 784.68 kN > 747.288 kN.

Hence the design is safe.

Q.4 [12 Marks]

Solution :

$L_x = 6m$ and $L_y = 9M$

Ratio of $\frac{L_x}{L_y} = 1.5$

Thickness of slab $= \left(\frac{span}{50}\right) = \left(\frac{6000}{50}\right) = 120\ mm$

self weight of slab $= (0.12 \times 24 \times 1) = 2.88\ kN/m^2$

Live load on slab $= 3.00\ kN/m^2$

filishes $= 0.12\ kN/m^2$

Total service load $= 6.00\ kN/m^2$

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Q.4

Total ultimate design load, $W_{ud} = (1.4 \times 3) + (1.6 \times 3)$
$= 9.00\ KN/m^2$

Taking α_x bending moment co-efficient for shortar span
$= 0.089$ & for longer span $= 0.056$

$M_{sx} = (0.089 \times 6 \times 6^2) = 19.3\ KN_m/m$

$M_y = (0.056 \times 6 \times 6^2) = 12.1\ KNm/m$

Total moment in the middle strip (x-direction)
$= (19.3 \times 0.75 \times 9) = 130\ KNM$

using minimum cover = 30 mm for Tendon at centre of slab,
The distance between the top kern and the centroid of cable

$= (120 - 20 - 40) = 50\ mm$

if p = Total prestressing Force required in x-direction

$(p \times 50) = (130 \times 10^6)$

$p = (26 \times 10^5)\ N = 2600\ KN$

∴ Force in each cable = 100 KN

∴ Number of cables in x direction (middle strip) = 26

spacing of cables $= \left[\dfrac{0.75 \times 9 \times 1000}{26}\right] = 260\ mm$

adopt a spacing of 250 mm (four cables per meter)

Total moment in middle strip (y direction)
$= (12.1 \times 0.75 \times 6) = 55\ KNm.$

providing a cover of 40mm to cables in y direction
Distance between cable and top kern

$= 120 - 40 - 40 = 40\ mm$

∴ prestressing Force required $= \left[\dfrac{55 \times 10^6}{40 \times 10^3}\right] = 1380\ KN.$

∴ Numbers of cable in y direction (middle strip) $= \dfrac{1380}{100}$
$= 14$

spacing of cables $= \left[\dfrac{0.75 \times 6 \times 1000}{14}\right]$

$= 320\ mm$

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The cable profile is parabolic with maximum eccentricity at centre and concentric at the supports.

Check for limit state of collapse

ultimate moment (x direction) $= 0.089 \times 9 \times 6^2 = 29$ kNm/m

$A_p = (4 \times 4 \times 20) = 320\ mm^2$

Q.4

$$\left[\frac{A_p f_p}{b d f_{ck}}\right] = \left[\frac{320 \times 1600}{1000 \times 90 \times 40}\right] = 0.142$$

Assumed $\left[\frac{f_{pu}}{0.87 f_p}\right] = 1$

$$f_{pu} = (0.87 \times 1600) = 1392\ N/mm^2$$

and $\left(\frac{x_u}{d}\right) = 0.29$

$$x_u = (0.29 \times 90) = 26.1\ mm$$

$$M_u = f_{pu} A_p (d - 0.42 x_u)$$

$$= 1392 \times 320 \left[\frac{90 - 0.42 \times 26.1}{10^6}\right] = 35.2\ kNm$$

The ultimate moment capacity of slab is higher than the minimum value required. a similar check may be made in y direction also.

Check for deflection under service load

The tendons following a parabolic profile in x and y directions induce uniformly distributed loads acting upwards are given by

equivalent load (x direction) $= \left[\frac{8 P e}{L_x^2}\right] = \left[\frac{8 \times 400 \times 0.03}{36}\right]$

$= 2.66$ kN/m

Equivalent load (y direction) $= \left(\frac{8 P e}{L_y^2}\right) = \left[\frac{8 \times 320 \times 0.02}{81}\right]$

$= 0.64$ kN/m

$\therefore$ unbalanced service load $= (6 - 2.66 - 0.64)$

$= 2.7\ kN/m^2$

$= 0.0027\ N/mm^2$

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using deflection co-efficient, for an aspect ratio of $\frac{L_y}{L_x} = 1.5$ The deflection is given by

$$a_{max} = \alpha \left[\frac{q L_x^4}{D} \right]$$

where, α = Coefficient = 0.00772

q = u.d.l = 0.0027 N/mm²

$$D = \text{Flexural rigidity} = \left[\frac{Eh^3}{12(1-V_c^2)} \right]$$

$$= \left[\frac{3800 \times 120^3}{12(1-0.15^2)} \right] = 5.62 \times 10^9$$

$$\therefore a_{max} = 0.00772 \left[\frac{0.0027 \times 600^4}{5.62 \times 10^9} \right] = 4.85 \text{ mm}$$

maximum permissible long term deflection $= \frac{6000}{250} = 24$mm

Check For stresses

unbalanced load = 2.7 kN/m²

moment due to this load (x direction) $= 0.089 \times 2.7 \times 6^2$
$= 8.7$ kNm

$$\text{stress developed} = \left[\frac{8.7 \times 10^6}{(1000 \times 120^2)/6} \right] = 3.33 \text{ N/mm}^2$$

Compression at top and tension at soffit of slab

Direct stress due to prestressing force $= \left[\frac{400 \times 1000}{1000 \times 120} \right]$
$= 3.66$ N/mm² (C)

$\therefore$ maximum compressive stress in concrete at the top of slab $= (3.66 + 3.33) = 7$. N/mm²

which is less than the permissible stress of 13 N/mm²

The maximum shear stress under ultimate load

$$= \left[\frac{0.424 \times 9 \times 6000}{1000 \times 90} \right]$$

$$= 0.26 \text{ N/mm}^2$$

which is negligibly small & hence no shear Reinforcement are neccessary.

Q.5 [a] [6-Marks]

⟹ Solution : Diameter of concrete pipe $= D = 1200mm$

Fluid pressure $= N_d = 1.6\ N/mm^2$

Thickness of concrete shell $= t = 100mm$

Diameter of high tensile wires $= d = 5mm$

Initial stress in wires $= F_s = 1000\ N/mm^2$

Compressive stress in concrete at transfer $= f_{ct} = 16\ N/mm^2$

Rasidual compressive at service loads $= f_{min,w} = 1\ N/mm^2$

Loss ratio $= \eta = 0.8$

The thickness of concrete pipe is evaluated using the relation

$$t > \left[\frac{N_d}{\eta f_{ct} - f_{min,w}}\right] > \left[\frac{1.6\,(0.5 \times 1200)}{(0.8 \times 14) - 1}\right] > 94.1\,mm$$

Thickness provided $= 100mm$. Hence the compressive stress in concrete is given by

$$f_c = \left[\frac{W_w D}{2\eta t} + \frac{F_{min.w}}{\eta}\right] = \left(\frac{1.6 \times 1200}{2 \times 0.8 \times 100} + \frac{1}{0.8}\right)$$

$$= 13.25\ N/mm^2$$

Number of turns HT wires winding is given by the relation

$$n = \left[\frac{4000\,(t + \alpha_e t_s)}{\pi d^2 f_s}\right] = \left[\frac{4000 \times 100 \times 13.25}{\pi \times 5^2 \times 1000}\right]$$

$$= 68 \text{ turns/meter}$$

$\therefore$ pitch of wire winding $= \dfrac{1000}{68} = 14.7\,mm$

Q.5 [B] [6-Marks]

⟹ Solution :

The design equations for the computations of the minimum wall thickness, circumferential prestress, spacing of wires and vertical prestress required are as follows.

1) Estimate the maximum ring tension, Nd and bending moment Mw in the walls of the tank using the IS code Tables.

2) Minimum wall Thickness $= \left[\dfrac{Nd}{\eta f_{ct} - f_{min.w}}\right]$

The thickness of the wall provided should be such that a minimum cover of 35mm is available to vertical prestressing cables. In practise the walls are seldom less than 120mm thick to ensure proper compaction of concrete.

3.) The circumferential prestress required is given by

$$f_c = \left[\frac{Nd}{\eta t} + \frac{f_{min.w}}{\eta}\right] \ N/mm^2$$

4) The spacing of wires required at any section is obtained by considerations of the hoop tension due to fluid pressure and hoop compression due to the circumferential wire winding as follows.

if A_s = c/s Area of wire winding mm^2

w_t = average radial pressure of wires at transfer section N/mm^2

D = Diameter of the tank, mm

s = spacing of wires at the given section, mm

f_s = stress in wires at transfers N/mm^2

t = thickness of the tank wall, mm

f_c = Compressive stress in concrete, N/mm^2

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∴ Hoop compression due to prestressing $= \dfrac{w_t \cdot D}{2}$

Equating $\dfrac{w_t D}{2} = \dfrac{f_s \cdot A_s}{s}$

$\therefore w_t = \left[\dfrac{2 f_s \cdot A_s}{s D}\right]$

Q.5 [b]

If Nd = hoop tension due to hydrostatic working pressure, Ww

Nt = hoop compression due to radial pressure of wires, wt

then $Nt = Nd \left[\frac{wt}{Ww}\right]$

Also, $Nt = t f_c$.

The spacing of the wires $s = \left[\frac{2Nd}{Ww} \times \frac{f_s \cdot A_s}{f_c \cdot Dt}\right]$ mm

5) vertical prestress required to resist the bending moments in the wall due to the circumferential wire winding and Hydrostatic pressure as a consequence of end restrained is computed as follows.

if Mt = vertical moment due to the prestress at transfer

& Mw = vertical moment due to Hydrostatic pressure

then $Mt = Mw \left(\frac{wt}{Ww}\right)$

The compressive prestress required in concrete

$$f_c = \left[\frac{f_{min \cdot w}}{\eta} + \frac{Mw}{\eta Z}\right]$$

where Z = section modulus of unit length of wall.

6) when tank is empty, the prestress required

$$f_c = \left[\frac{f_{min.w}}{\eta} + \frac{Mt}{Z}\right]$$

7) vertical prestressing force required is given by

$$P = f_c \cdot A_c$$

where, A_c = c/s area of Concrete per unit length along the circumference.

according to the I.S. code vertical prestressing force is to be designed for 30% of hoop compression.

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Q.6 [a] [6-Marks]

⟹ Solution :

In prestressed concrete members, cracks may developed due to a variety of reasons. concrete can cracks in its plastic phase, when it is still not set. due to plastic shrinkage and settlement. Concrete during its hardening phase is likely to develop cracks due to constraints to early thermal movement, drying shrinkage or due to differential settlement.

In hardened state, concrete can crack due to overload, Faulty construction, inedequate detailing, sulphate attack on cement concrete, Alkali agregate reaction and long term drying shrinkage.

1) plastic settlement cracks : such kind of cracks generally occur in Deep sections and Top of columns through and waffle slabs.
The primary cause of occurance of plastic settlement cracks is excess bleeding. and it may occur due to rapid early drying conditions.
remedial measure – Reduce bleeding do air entrainment or reviberate mildly.

2) plastic shrinkage cracks : The most common location of occurance of plastic shrinkage cracks is in roads and slabs, reinforced concrete slabs etc. it may occur due to low rate of bleeding and Fast surface.
remedial measure – Improve early curing.

3) Early thermal contraction cracks : its most common Location of occurance is in thick walls & in thick slabs. it generally occurs due to excess heat generation, excess temprature gradients.
remedial measure – reduced heat.

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4) Long term drying shrinkage cracks : occurs generally in thin slabs and in thin walls. it also occurs when there is excess shrinkage and inefficient curing.
Remedial measure :- Reduce water content and Improve curing of concrete.

5) Crazing cracks : it generally occurs only on surface of concrete. & over the fair faced concrete slabs. it may occur when impermeable form work over trowelling is done. it may occur in rich concrete mixes with low or poor curing.
Remedial measure — Improve curing and smoth finishing.

6) Cracks due to alkali agregate reaction — it generally occurs in a place where dampness is present. it occurs when reactive silicates & carbonates present in agglegate reacts with alkalis present in cement.
Remedial measure — use of inert agglegate and low alkali content cement.

Q.6 [b] [6- marks]

⟹ Solution : When the dormant cracks wider than about one meter, it is more economical to use the grouting and sealing technique.
This method involves enlarging the crack along its exposed face and then the crack is cleaned and grouted. The surface is then sealed with a suitable joint sealant. types of sealant is selected based on the amount of movement and the limitations imposed by the size of the exposed face which can be cut together with the type of crack ie either vertical or horizontal.
Following three types of sealantes are generally used depending upon their suitability in given situation.
1) Mastics 2) Thermoplastic
3) Elastomers.

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1) mastics — mastics are a asphalts with a low melting point with added Fillers or fibers.
They are recommonded when the total movements will not exceed 15% of the width of the groove.
The groove should be cut so that it has a depth to width ratio of 2:1.
mastics are cheapest of the sealants but their use should be restricted to vertical situations.

Q.6 [b]

2) Thermoplastics : It comprosing asphalts, pitches and coal tar become liquid or semi viscous when heated.
the pouring temprature is usually above 100°c. the groove depth to width ratio should be 1:1 and the total design movements is of the order of 25 percent of the groove width.
These materials soften less than mastics but they may excrude under high ambient tempratures and they may be degraded by ultraviolet light, losing elasticity after a few years of exposure to direct sunlight.

3) Elastomers : It include a wide range of materials such as polysulphides, epoxy polysulphides, polyurethane, silicones and acrylics.
These materials are advantageous since they can be used without heating.
The groove depth to width ratio should be 1:2. The material should be prevented from adhering to the bottom so that the crack remains free as a live crack.

END