UNIT - IV

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TENSION MEMBERS

pipes and tanks are prestressed by providing a prestress wires round a core of concrete. The wires are covered with pneumatic mortar. Some times longitudinal prestress is also done by pretensioning the longitudinally placed wires. After placing the concrete is tensioned by applying a high pressure inside the pipe. After the hardening of the concrete the inside pressure is released. The tension wires compress the concrete

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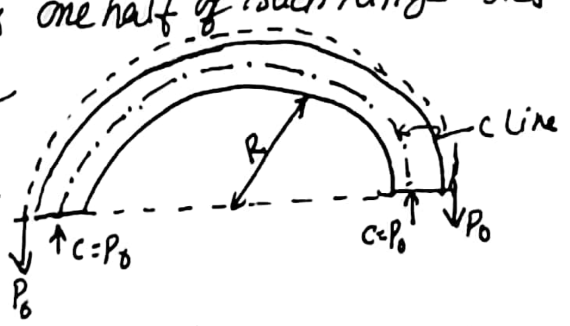
Principles of Circumferential prestressing

In a tank ~~of~~ a circumferential prestress is provided to resist hoop tension produced by internal liquid pressure. The tank may be taken as consist of a number of circular rings of unit height each of which is designed for hoop tension. The fig shows one half of such rings. This ring is sorrounded by prestressing wire carrying an initial circumferential prestressing force P_0. The C-line or the line of prestress will coincide with the circumferential centroidal line



Let P_0 = initial prestressing force

compressive stress in concrete due to initial Ps force =

$($ P_0

After allowing the losses if the net PS force is P, then the Comp. stress in concrete due to final PS force

$$= f_c = \frac{P}{A_c}$$

If an internal pressure p is applied, hoop tension due to this internal pressure is pR.

This Hoop tension is resisted by Concrete and Steel. using the principle of transformed section based on Elastic theory,

Equivalent Concrete area $= A_e = A_c + m A_{st}$

where m = modular ratio.

Tensile stress in Concrete due to internal liquid pressure

$$= pR/A_e$$

Resultant stress in Concrete under the action of final PS force and internal liquid pressure $= \frac{P}{A_c} - \frac{pR}{A_e}$

UNIT-IV

DESIGN OF ~~CIRCULAR~~ PRESTRESSED CONCRET TANK

Steps.

1) Estimate the max. Ring tension N_d and BM M_w in the walls of tank using IS code (3370 part IV) Tables.

2. Min. wall thickness = $\frac{N_d}{\eta f_{ct} - f_{min\,w}}$.

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The thickness of the wall provided should be such that a minimum cover of 35mm is available to the vertical prestressing cables. In practice the walls are less than proper Compaction of concrete. 150mm thick to Ensure

3. The Circumferential prestress reqd is given by

$$f_c = \frac{N_d}{\eta t} + \frac{f_{min.w}}{\eta} \text{ N/mm}^2$$

4 The spacing of wires reqd at any section is obtained by consideration of hoop tension due to fluid pressure and hoop compression due to the Circumferential wire winding as follows

If A_w = c/s area of wire winding, mm²
w_t = avg. radial pr. of wires at transfer at a given sectio N/mm²
D = dia. of tank in mm
f_s = stress in wires at transfee
f_c = comp. stress in concrete

Hoop Compression due to prestressing $= \frac{N_t D}{2}$

Equating $\frac{N_t D}{2} = \frac{f_s A_s}{S}$

$$\therefore N_t = \frac{2 f_s A_s}{s D} \quad -①$$

If N_d = hoop tension due to hydrostatic working pressure, N_w

N_t = hoop compression due to Radial pr. of wires N_t.

then $N_t = N_d\left[\frac{N_t}{N_w}\right]$ — ②

also $N_t = t \cdot f_c$

from Eqn ① & ②, the spacing of the wires

winding $$S = \frac{2Nd}{N_w} \cdot \frac{f_s A_s}{f_c D t} \text{ mm}$$

5) The vertical prestressing reqd to resist the BM in the wall due to circumferential wire winding and hydrostatic pressure as a consequence of end restraint is computed as follows:

If M_t = vertical moment due to the prestress at transfer

and M_w = vertical moment due to hydrostatic pressure

then $M_t = M_w\left[\frac{N_t}{N_w}\right]$

The compressive prestress reqd in concrete is expressed as

$$f_c = \frac{f_{min\,w}}{\eta} + \frac{M_w}{\eta Z}$$

when the tank is empty, the Ps reqd is

$$f_c = \frac{f_{min.w}}{\eta} + \frac{M_t}{Z}$$

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⇒ The vertical Ps force reqd is given by

$$P = f_c \cdot A_c$$

According to IS, the vertical prestressing force is to be designed for 30% of the hoop compression.

The walls of the tank should be suitable reinforced, since the circumferential wire winding is generally perform prior to the vertical prestressing of walls.

#1) A cylindrical PSC water tank of internal diameter 30m is required to store water over a depth of 7.5m. The per. comp. stress in concrete at transfer is 13N/mm² (fc and min comp stress under working pressure (fmin.w) is 1N/mm². The loss Ratio (η) is 0.75. wires of 5mm ∅ with an initial stress of 1000N/mm² (pi) are available for circumferential winding and Freyssinet Cables made up of 12 wires of 8mm-∅ stressed to 1200N/mm² are to be used for vertical prestressing. Design the tank walls assuming the base as fixed. fck = 40N/mm

Sol For the reqd depth of storage of 7.5m & D = 30m, an average wall-thickness of 150mm is tentatively assumed based on Table (33 - 90)

$D = 30m$; $H = 7.5m$ and $t = 150mm$; $\eta = 0.75$

$$\frac{H^2}{Dt} = \frac{7.5^2}{30 \times 0.15} = \underline{12.5}$$

& $w_w = wH$ (kN/m²)

$= (10 \times 7.5)$ kN/m²

$= \underline{0.075}$ N/mm²

referring to tables 16:

the man ring tension (N_d) + moment in tank walls (M_w) for the fixed base condition are:

Coeff ω H R

$N_d = (0.64 \times 10 \times 7.5 \times 15) = 720$ kN/m $= 720$ N/mm

$M_w = (0.01 \times 10 \times 7.5^3) = 42.5$ kN/m

Coeff × ωH³

$= \underline{42500}$ N/mm

Step ii

Min. wall thickness $t = \frac{N_d}{\eta f_{ct} - f_{min.w}} = \frac{720}{(0.75 \times 13) - 1} = 82.3mm$

Net thickness available (allowing for vertical cables of dia 30mm) is $(150 - 30) = \underline{120mm}$

Step iii

Required Circumferential Prestress is

$$f_c = \frac{N_d}{\eta t} + \frac{f_{min.w}}{\eta}$$

$$\therefore f_c = \frac{720}{0.75 \times 120} + \frac{1}{0.75} = 9.4 \text{N/mm}^2$$

Step iv

Spacing of Circumferential wire windings at base is

$$S = \frac{2 N_d \cdot f_s A_1}{N_w f_c D t} = \frac{2 \times 720}{0.075} \times \frac{1000 \times 20}{9.4 \times 30 \times 10^3 \times 120} = 11.4mm$$

19.63

No: of wires/m $= \underline{87}\#$ (i.e 1000/11.4)

Ring tension N_d at 0.1H (0.75m) from top is

$$N_d = (0.097 \times 10 \times 7.5 \times 15) = 109\ kN/m = 109\ N/mm$$

$$f_c = \frac{109}{0.75 \times 120} + \frac{1}{0.75} = 2.5 N/mm^2$$

$$S = \frac{2 \times 109}{0.075} \times \frac{1000 \times 20}{2.5 \times 30 \times 10^3 \times 120} = 64mm$$

No: of wires/m at the top of tank $= 16 \rightarrow \frac{1000}{64}$

Step v

Max. Radial pressure due to prestress is $\rightarrow w_t = \frac{2 f_s A_s}{s \cdot D} =$ [illegible] $= 0.115$

Max vertical moment due to PS is $\rightarrow M_t = M_N \left(\frac{N_t}{N_N}\right) = 42500 \left(\frac{0.114}{0.075}\right) = 67000\ Nmm/mm$

$$= 67 \times 10^6\ Nmm/m$$

Considering one metre length of tank along the circumference, the section modulus is

$$Z = \frac{1000 \times 150^2}{6} = \underline{375 \times 10^4}\ mm^3$$

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∴ Vertical PS required is

$$f_c = \frac{f_{min \cdot w}}{\eta} + \frac{M_t}{Z} = \frac{1}{0.75} + \frac{67 \times 10^6}{375 \times 10^4} = \underline{19.2 N/mm^2}$$

Since this stress exceeds the per. value of $f_{ct} = \underline{13 N/mm^2}$, the thickness of tank wall at base is increased to 200mm. Thus

$Z = \frac{1000 \times 200^2}{6} = 666 \times 10^4 mm^3$

$f_c = \frac{1}{0.75} + \frac{67 \times 10^6}{666 \times 10^4} = 12 N/mm^2$

Vertical Ps force $= f_c A = \frac{(12 \times 1000 \times 200)}{(1000)} = 2400 kN$

using 8mm-φ ~~of~~ (12 nos) Freyssinet Cables.

Force/cable $= \frac{(50 \times 12 \times 1200)}{1000} = 720 kN$

∴ Spacing of vertical cables $= \frac{\overset{1m}{1000} \times 720}{2400} = \underline{300 mm}$.

· The approximate vertical-Ps reqd to counteract winding stresses as per IS code is $= \underset{(30\%)}{0.3 f_c} = 0.3 \times 9.4 = \underline{2.82 N/mm^2}$

∴ Vertical Ps force reqd $= \frac{2.82 \times 1000 \times 200}{1000} = \cancel{2} 564 kN$

ultimate tensile ~~stress~~ force in wires at base of tank

$\frac{\text{no of wires} \times a_{st} \times f_{pu}}{1000} = \frac{87 \times \overset{\pi(5)^2/4}{20} \times \overset{\cancel{1000}}{1500}}{1000} = \overset{1740}{\cancel{2610}}\ 2610$

∴ load factor against collapse $= \frac{\overset{\cancel{1740}}{2610}}{720} = \overset{\cancel{2.41}}{3.6}$

Direct tensile strength of concrete $= 0.267 \sqrt{\overset{f_{ck}}{40}} = 1.7 N/mm^2$

Cracking load $= \frac{(1000 \times 200)(\overset{0.75 f_c + f_c}{0.75 \times 9.4 + 1.7})}{1000} = 1760 kN$.

∴ Factor of safety against cracking $= \frac{1760}{720} = 2.45$

Nominal Reinforcement of 0.2% of the c/s area are to be provided in the circumferential and longitudinal directions. This requirement will be fulfilled by providing 8mm ø mild steel bars @ 300mm spacing on both faces at a cover of 20mm.

Design the above Circular tank when the base Connection to be hinged.

Sol From tables (tanks with base hinged), the max Ring tension and moments are obtained for the tank parameter, $\frac{H^2}{DT} = 12.5$

$$N_d = (0.75\times10\times7.5\times15) = 840\ \text{N/mm}$$

$$M_w = (0.0039\times10\times7.5^3) = 16.5\ \text{kNm}$$
$$= 16500\ \text{Nmm/mm}$$

Thickness of wall, $t = \frac{840}{(0.75\times13)-1} = 96\text{mm}$

Thickness adopted for practical reasons of housing wire cables of 30mm ø = 150mm

Net thickness available = (15030) = 120mm

Circumferential Prestress,

$$f_c = \frac{840}{0.75\times120} + \frac{1}{0.75} = 10.75\ \text{N/mm}^2$$

Spacing of 5mm wires $S = \frac{2 \times 840}{0.075} \times \frac{1000 \times 20}{1075 \times 30 \times 10^3 \times 12}$

11.6mm

No. of wires/met = <u>86</u>

Similarly no. of wires/mt reqd towards the top of the tank = 16.

Radial pr. due to PS is : $w_t = \left[\frac{2 \times 1000 \times 20}{11.6 \times 30 \times 10^3}\right] = 0.115$ N/mm²

Max. vertical moment due to PS = $M_t = 165 @ \left[\frac{0.115}{0.075}\right]$

$= 25400$ Nmm/mm

$= 25.4 \times 10^6$ N mm/m.

Considering one metre length, section modulus

$Z = 375 \times 10^4 \text{ mm}^3$

Vertical PS reqd = $f_c = \frac{1}{0.75} + \frac{25.4 \times 10^6}{375 \times 10^4} = 8.2$ N/mm²

Vertical PS Force = $(8.2 \times 1000 \times 150)/1000 = 1230$ kN

Using 8mm-Ø bar-1200 Force/cable = $(50 \times 12 \times 1200)/1000 = 720$ kN

Spacing of Freyssinet cables containing 12 wires of 8mmØ

$= \frac{1000 \times 720}{1230} = 585$ mm

The min value of load-factor against collapse of 2 and against cracking of 1.2 are available to satisfy the IS code requirements of strength & serviceability.

sep.2012 A prestressed concrete circular cylindrical tank is required to store water to a depth of ~~5m~~ 10.5m and internal dia. 50m. The permissible compressive stress in concrete at transfer should not exceed 14 N/mm² and the minimum compressive stress under working pressure should not be less than 1 N/mm² The loss ratio is 0.8 high tensile steel wires of 7mm ϕ with an initial stress of [illegible] 1000 N/mm² are available for winding round the tank Freyssinet cables of 10 wires of 8mm ϕ which are stressed to 1100 N/mm² are available for vertical prestressing. The cube strength of concrete is 40 N/mm². Design the tank wall when the base is hinged.

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jan/feb - 2013 A cylindrical PSC water tank of internal dia. 30m to store water to a depth of 9.5m is to be designed using M40 grade concrete. The thickness of the ~~wall of~~ tank wall is 150mm. and 5mm dia high tensile wires are available for use. The per. comp stress in concrete at transfer is limited to 13 N/mm² and no tensile stresses are allowed under service load conditions. The initial stress in high tensile wires is 1200 N/mm². The loss of PS is limited to be 18%. Assume the base slab as fixed. Design the tank wall

COMPRESSION MEMBERS · PIPES

DESIGN OF NON-CYLINDRICAL PIPES

According to IS:784, the design of PSC pipes should cover the following five stages:

1) circumferential prestressing, winding with or without longitudinal prestressing.

2) handling stresses with or without longitudinal prestres

3) condition in which a pipe is supported by sad. at extreme points with full water load but zero hydrostatic pressure

4) full working pressure confirming to the limit state of serviceability

5) The first crack stage corresponding to the limit state of total damage.

* DESIGN OF NON-CYLINDRICAL PIPES

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steps:

1) Determine the min-thickness of concrete reqd and pitch of the circumferential wire winding on the pipe

if N_d = hoop tension developed under working pr.

t = thickness of conc. pipe

D = dia of pipe

ww = hydrostatic pr

f_{ct} = per. comp stress in conc @ transfer

$f_{min,w}$ = per. stress in conc under working pr we have $\frac{N_w D}{2l} < (\eta f_{ct} - f_{min.w})$

$$t \geq \left[\frac{N_w D/2}{\eta f_{ct} - f_{min,w}}\right] > \left[\frac{N_d}{\eta f_{ct} - f_{min,w}}\right]$$

In Case of liquid retaining Structure, to ensure water tightness, the value of $f_{min.w}$ is either zero or a min. Comp. Stress of 20 percent of the ultimate Comp. Strength of Concrete.

(ii) If f_c = actual Compressive in Concrete

$$f_c = \frac{\overset{\rightarrow N_w D/2}{N_d}}{\eta t} + \frac{f_{min.w}}{\eta}$$

At transfer, the p.s force per mt length of the pipe is given by

$P = 1000 \times 2 t r f_c$ where t is in mm
f_c is in N/mm²

(iii) If A_s = c/s area of wire/m length of pipe

f_s = stress in wire @ transfer

n = no. of turns of circumferential wire winding/m length of pipe

d = dia. of pipe

$$A_s = \frac{2\pi d^2 (n)}{4} = \frac{2000 t f_c}{f_s}$$

Since $A_s f_s = P \Rightarrow \left(\frac{2\pi d^2 n}{4}\right) f_{sc} = 2000 t \cdot f_c$

$$\Rightarrow \boxed{n = \frac{4000 t f_c}{}}$$

<u>losses of prestress</u> Due to elastic deformation of concrete during circumferential wire winding, there is a loss of prestress which depends upon the modular ratio α_e and reinforcement ratio ρ

If f_{si} = initial winding stress in steel

f_{se} = eff. stress in steel after winding for compatibility of strains

then $\dfrac{f_{si} - f_{se}}{E_s} = \dfrac{f_c}{E_c} = \left(\dfrac{A_s f_{se}}{2000t}\right) \dfrac{1}{E_c}$

if $\dfrac{A_s}{2000t} = \rho = \dfrac{f_c}{f_s}$ and $\dfrac{E_s}{E_c} = \alpha_e$ $\quad (\rho f_{se}) \cdot \dfrac{1}{E_c} = f_{si}$ $\quad \dfrac{E_s}{E_c} + f$ $\quad f_{se}$ $\quad f$

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then $f_{si} = (1 + \alpha_e \rho) f_{se}$

for PSC pipes, the % reinforcement varies between 0.5 and 1 percent and the modular ratio b/w 5 & 6. Hence the loss due to elastic deformation is about 3 to 6 percent of the initial stress

In addition to the above losses, various other loss shall also be considered

Design a non cylinder PSC pipe of 600mm internal diameter to withstand a working hydrostatic pressure of $1.05 N/mm^2$, using a 2.5mm high tensile wire stressed to $1000 N/mm^2$ at transfer. Per. max & min stresses in con at transfer and service loads are 14 & 0.7 N/mm

The loss ratio is 0·8 . calculate also the test pressure required to produce a tensile stress of 0·7N/mm² in concrete when applied immediately after tensioning and also the winding stress in steel if $E_s = 210 kN/mm^2$ & $E_c = 35 kN/mm^2$

sol

i) $t > \frac{Nd}{\eta f_{ct} - f_{min.w}} > \frac{(w_N D/2)}{\eta f_{ct} - f_{min w}}$

$\geqslant \frac{105(600/2)}{(0.8 \times 14 - 0.7)}$

$\geqslant$ 30mm

for a 30mm thick concrete pipe, the actual comp. stress in concrete f_c = 14 N/mm²

The no: of turns of 2·5 mm wire stressed to 1000N/mm² per m length of the pipe is given by

$$n = \frac{4000 t f_c}{\pi d^2 f_s} = \frac{4000 \times 30 \times 14}{\pi (2.5)^2 \times 1000} = 86 \text{ turns/m}$$

spacing/ pitch of circumferential wire winding = $\frac{1000}{86}$ = 11·6 mm

If w_w = test pr. reqd immediately after winding ($\eta = 1$)

from Eqn $f_c = \frac{w_N D}{2\eta t} + \frac{f_{min.w}}{\eta}$

$$w_w = \frac{2t}{D}(f_c - f_{min.w}) = \frac{2 \times 30}{600}\left[14 - (-0.7)\right]$$

$= 1 \cdot 47 \; N/mm^2$

If f_{si} = winding stress in steel,

$$f_{si} = (1+\alpha_e \rho)\, f_{se}$$

$$\alpha_e = 6;\ \rho = \frac{f_c}{f_s} = \frac{14}{1000} = 0.014$$

$$\therefore f_{si} = (1+6\times 0.014)\,1000 = \underline{1084\,N/mm^2}$$

Mar-2011 Design a non cylindrical PSC pipe of 550mm int. dia. having a working hydrostatic pressure of 1.02 N/mm², using a 2mm high tensile wire stressed to 800 N/mm² at transfer permissible max & min stresses in concrete ~~at~~ transfer and service loads are 13 N/mm² & 0.5 N/mm². The loss ratio is 0.8. calculate also the test pr. reqd to produce a tensile stress of 0.5 N/mm² in concrete when applied immediately after tensioning and also the winding stress in steel if $E_s = \underline{210}\,kN/mm$ and $E_c = \underline{35}\,kN/mm^2$.

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DESIGN OF CYLINDRICAL PIPES

The design principles of cylindrical pipes are similar to those of the noncylinder pipes, except that the required thickness of concrete is computed by considering the equivalent area of light guage steel pipe embedded in concrete.

If t_s : thickness of steel pipe

α_e = modular ratio = $\frac{E_s}{E_c}$

thickness of concrete pipe reqd is given by

$$t = \frac{N_d}{\eta f_{ct} - f_{min\,w}} - \alpha_e t_s$$

The PS reqd in the concrete at transfer is

$$f_c = \frac{N_d}{\eta(t + \alpha_e t_s)} + \frac{f_{min\,w}}{\eta}$$

The no. of turns of circumferential wire winding per meter length of pipe is

$$n = \frac{4000(t + \alpha_e t_s) f_c}{\pi d^2 f_s}$$

** → The failure of non-cylinder pipes is due to the excessive cracking of concrete, reducing in the decrease of internal fluid pressure.

- the mechanism of failure is one of progressive collapse due to excessive leakage without any sudden fracture of steel

However in case of <u>Cylindrical pipes</u> there are possibilities of pipe bursting due to the yielding of the steel cylinder accompanied by the Excessi elongation or fracture of the circumferential wire winding

The bursting fluid pressure is estimated from eqn

$$P_u = \frac{f_{pu} A_{B} + f_y \cdot A_{cs}}{D}$$

Since $A_B = \frac{\pi d^2}{2} n = 1.57 d^2 n \ mm^2/m$.

$= 0.00157 d^2 n \ mm^2/m$

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and $A_{cs} = 2t_s$

$$P_u = \frac{0.00157 d^2 n f_{pu} + 2t_s f_y}{D}$$

where P_u = bursting Pr., N/mm^2

d = dia of wire winding, mm

n = no: of turns per m length of pipe

f_{pu} = tensile strength of wire winding, N/mm^2

f_y = yield stress of steel cylinder, N/mm^2

t_s = thickness of steel cylinder, mm

D = dia of steel cylinder, mm.

A PSC Cylinder pipe is to be designed using a steel cylinder of 1000mm int. dia and thickness 1.6mm. The Circumferential wire winding consist of a 4mm high tensile wire, initially tensioned to a stress of 1000N/mm². ultimate tensile strength of the wire = 1600N/mm², yield stress of steel cylinder = 280N/mm². The max. permissible Comp. stress in Concrete at transfer is 14N/mm² and no tensile stresses are permitted under working pr. of 0.8N/mm². Determine the thickness of Concrete lining Required, the no of turns of circumferential wire winding and the factor of safety against bursting. Assume

$\alpha_e = 6$

Sol

$$t > \frac{Nd}{2f_{ct} - f_{min} \cdot N} - \alpha_e t_s$$

$$> \frac{0.8\,(1000/2)}{0.8 \times 14 - 0} - 6 \times 1.6 \quad > \underline{\underline{25.9\,mm}}$$

using 26mm thick lining, $f_c = 14\,N/mm^2$

$$n = \frac{4000 \times (26 + 6 \times 1.6)\,14}{\pi\,(4^2)\,(1000)} = \underline{\underline{40\ turns/metre}}$$

Bursting pressure, $P_u = \frac{(0.00157 \times 4^2 \times 40 \times 1600) + (2 \times 1.6 \times 280)}{1000}$

$$\boxed{P_u = 2.516\,N/mm^2}$$

Factor of Safety against bursting = $\frac{\text{Bursting Pr}}{\text{Working Pr}} = \frac{2.516}{0.80} = \underline{\underline{3.14}}$

Prestressed Concrete poles

are widely used for railway powerlines & signal lines antenna mast, telephone transmission, low & high voltage Electric power transmission and substation towers.

Advantages

i) resistance to corrosion
ii) freeze & thaw resistance in Cold Region
iii) Easy handling due to less weight
iv) fire-resistant
v) Easily installable
vi) Clean & neat in appearenc
vii) have increased crack resistance.

Design Considerations

The poles are generally designed for the following Critical load Considerations:

i) Bending due to wind load on the cable
ii) combined bending & torsion due to eccentric snappi of wires
iii) Max. torsion due to skew snapping of wires.
iv) Bending due to falling of all wires on one side of t
v) handling & erection stresses.

PSC PILES

Advantages

i) High load & moment Carrying Capacity

ii) Standardization in the design for mass production

iii) Excellent durability

iv) Crack free characteristics

v) Resistant to tensile load due to uplift

vi) Combined load-moment Capacity

vii) Piles can be lengthed by Splicing

viii) ease of handling, transporting & driving.

ix) Overall Economy in production

x) Advantageous for deep foundations to carry heavy loads on weak soils.

UNIT-V

SLABS

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TYPES OF PSC FLOOR SLABS

PSC slabs are ideally suited for floor & roof Constru of industrial buildings where the L.L ~~to~~ to be supporte are of higher order and uninterrupted floor space is desirable for which reason longer spans b/w the support elements are required.

Design of ONE WAY SLAB or (similar to Beam) (SPANNING B/W PARALLEL SUPPORTS)

The deck slab of a road bridge of span 10m is to be designed as a one way PSC slab, with parallel post tensi cables in each of which the force at transfer is 500k If the deck slab is required to support a uniformly distributed L.L of 25 kN/m², with the compressive and tensile force in concrete at any stage not exceeding 15 and 0 N/mm² respectively. Calculate the max. horizo spacing of the cables and their positions at the mid spa section. Assume the loss ratio as 0.8.

Sol Step 1
The L.L & D.L moments are computed considering one metre width of slab. $wl^2/8$

$$M_q = \frac{25 \times 10^2}{8} = 312.5 \text{ kN-m.}$$

Let h = over all depth of slab in mm
b = width of slab in mm

$$\frac{b\times h\times 24\times 10^{2}}{(10^{3})\times 8}$$

$$M_d = \frac{w_d l^2}{8} \qquad w_d = \frac{\overset{\text{Area}\times\rho}{b\times h\times 24}}{\cancel{10^6}}\ \text{kN/mm} = \frac{300bh}{10^6}\ \text{kN-m}$$

$$= \frac{\overbrace{bh\times 24}^{w_d}\times 10^2}{10^6\times 8\times \cancel{10}} = \cancel{300\ \text{kN-m}}\ \frac{300\,bh\ \text{kN.m}}{10^6} = 300bh\ \text{N-mm}$$

300bh N-m

$f_{ct} = f_{cw} = 15\ \text{N/mm}^2$

$f_{tt} = f_{tw} = 0\ \text{N/mm}^2$

$$M_d = \frac{bh\times 24\times 10^2}{10^6\times 8}$$

$$M_d = \frac{300\times bh}{10^6}\ \text{kN-m}$$

↓

$300bh$ N-mm ①

Range of stress $f_{br} = \cancel{\eta\times f_{ct} + f_{tt}} = (\eta f_{ct} + f_{tw})$

$= 0.8\times 15 + 0$

$\boxed{f_{br} = 12\ \text{N/mm}^2}$

Hence
Min. section modulus

$$Z_b = \frac{\overset{1m}{b}h^2}{6} = \left[\frac{M_L + (1-\eta)M_d}{f_{br}}\right]$$

$$\frac{1000\times h^2}{6} = \left[\frac{\cancel{312.5\times 10^6\ (1-0.8)\ 300\times 1000\times h}}{12}\right]$$

$\Rightarrow \cancel{h^2 - 30h + 156250 = 0}$

$$Z_b = \frac{1000h^2}{6} = \left[\frac{(312.5\times 10^6) + (1-0.8)\,300\times 1000\times h}{12}\right]$$

$\Rightarrow h^2 - 30h - 156250 = 0$

$\Rightarrow \boxed{h = 410\text{mm}}$

$$A = \overset{b\times h}{1000 \times 410} = 410000\,mm^2 . = 41\times10^4$$

$$Z_b = Z_t = 1000 \times \frac{410^2}{6} = 28\times10^6\,mm^3$$

& $M_d = 300 \times 1000 \times 410$ N-mm (from ①)

$= 123\times10^6$ N-mm

resultant (top) stress @ fibres

$$\boxed{f_{sup} = \cancel{0} \left[f_{tt} - \frac{M_d}{Z_t}\right]}$$

$$= \left[0 - \frac{123\times10^6}{28\times10^6}\right] = -4.4\,N/mm^2$$

$$\boxed{\underset{\text{(bot) fibres}}{f_{inf}} = \left[\frac{f_{tw}}{\eta} + \frac{M_d+M_L}{\eta Z_b}\right]}$$

$$= 0 + \frac{(312.5+123)\times10^6}{0.8\times28\times10^6} = 19.4\,N/mm^2$$

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∴ The min. p.s force reqd is given by

$$P = A\left(\frac{f_b Z_b + f_t Z_t}{Z_t + Z_b}\right)$$

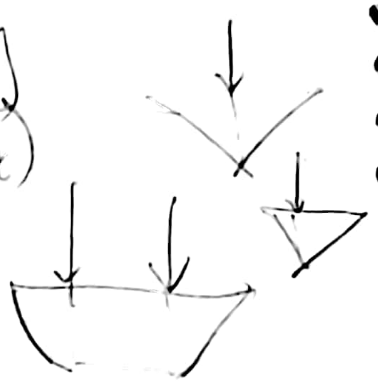
$$P = \left[\frac{A(f_{inf} Z_b + f_{sup} \times Z_t)}{Z_t + Z_b}\right] \quad \because Z_t = Z_b$$

$$= \left[\frac{A\cdot \cancel{Z_b}[f_{inf}+f_{sup}]}{2\cancel{Z_b}}\right] = \frac{A}{2}(f_{inf} + f_{sup})$$

$$= \frac{41\times10^4}{2}(19.4 - 4.4) = 3075\times10^3\,N = 3075\,kN$$

$$\boxed{P = 3075\,kN}$$

$$\frac{Z}{A}\cdot\frac{(f_b - f_t)}{(f_b + f_t)}$$



Eccentricity

$$e = \frac{Z\,(f_{inf_b} - f_{sup_t})}{A\,(f_{inf} + f_{sup})}$$

$$= \left[\frac{28\times10^6\,(19.4 + 4.4)}{41\times10^4\,[19.4 - 4.4]}\right] = \underline{\underline{109\,mm}}$$

$$Spaing = \frac{\cancel{\quad}\ \times \text{prestress @ transfer}}{P}$$

$$\therefore\ \text{Spacing of Cables} = \frac{\overset{\text{per m}}{1000} \times \overset{\text{force @ transfer}}{500}}{3075} = \underline{\underline{162\,mm}}$$

DESIGN OF PSC TWO WAY SLAB (SUPPORTED ON ALL 4 SIDES)

(refer BS 8110 Tables)

\# Design a post tensioned PSC two way slab 6m by 9m with discontinuous edges, to support an imposed load of 3kN/m². Cables of four wires of 5mm ϕ carrying an effective force of 100kN are available for use. Design the spacing of cables in two directions and check for the safety of the slab against collapse and excessive deflection at service loads. Assume $f_{ck} = 40N/mm^2$; $f_p = 1600N/mm^2$ & $E_c = 38\,kN/mm^2$, $\nu_c = 0.15$ & $\alpha = 0.0772$.

sol $l_x = 6m$ & $l_y = 9m$

$\frac{l_y}{l_x} = 1.5$ ∴ TWO Way slab

thickness of slab $= \frac{Span}{50} = \frac{6000}{50} = 120mm$

$\left\{ \frac{Span}{40} \text{ for Flat slabs} ;\ \frac{Span}{50} \text{ for normal PSC slabs in PSC} \right\}$

Self wt of slab $= 0.12 \times 24 \times 1 = 2.88$ kN/m²

L.L on slab $= 3.00$ kN/m²

f.f $=$ $\underline{0.12 \text{ kN/m}^2}$

3k 50kN

Total service load $=$ $\underline{6.0 \text{ kN/m}^2}$ i.e 6kN/m for 1m width of slab

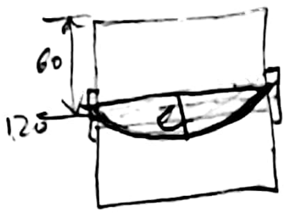
Total ultimate load $= W_{ud} = 1.5 \times 6$

$= 9.0$ kN/m²

Referring to table (IS 456) working moments in the middle strips are given by

$\boxed{M = \alpha w l_x^2}$

$M_{sx} = \alpha\, l_x^2\, W_s$ service $\alpha_x W_s l_x^2$



$= \underline{0.089} \times 6 \times 6^2 = \underline{19.3 \text{ kN m/m}}$

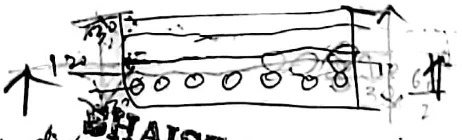
$M_{sy} = \underline{0.056} \times 6.0 \times 6^2 = 12.1$ kN m/m $\alpha_y W_s l_x^2$

Total moment in the middle strip (x direc)

$M_x = 19.3 \times 0.75 \times 9 = \underline{130 \text{ kNm}}$ ($\frac{3}{4} l_y$ middle strip)

using a min cover of $\underline{30\text{mm}}$ for the tendons at the centre of slab, the dist b/w the top kern and centroid of cable

$= 120 - 30 - 40 = \underline{50\text{mm}}$

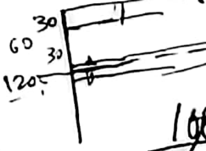


If P = Total PS force Reqd in x-dir

$\boxed{M = Pe}$

$P \times e \quad M_x$

$P \times 50 = 130 \times 10^6 \Rightarrow \underline{P = 2600 \text{ kN}}$



$\frac{100}{2600} \times 1000$

Force in each cable $= \underline{100 \text{ kN}}$

∴ No of cables in x-direction (middle strip) $= 26$

$$\frac{P_T}{P} = \frac{2600}{100} = 26$$

Spacing of cables $= \frac{\overset{\frac{3}{4} l_y}{0.75 \times 9 \times 1000}}{26} = \underline{260 mm}$. $\leftarrow \frac{\frac{3}{4} l_y}{\text{no. of cables}}$

Adopt a spacing of 250mm (4 cables/mt) $\frac{1000}{250} = 4$

Total moment in middle strip (~~x~~ y dir)

$M_{y4} \times \frac{3}{4} l_x l_2$

$M_y = 12.1 \times 0.75 \times 6 = 55$ kN-m

Providing a cover of $\underline{40mm}$ in the cable in Y-dir

Distance b/w cable and top kern $=$ ~~$\frac{55 \times 10^6}{40 \times 10^3}$~~ $120 - \underline{40} - \underline{40}$

$= \underline{40mm}$.

$\therefore$ PS force reqd $= \frac{55 \times 10^6}{40 \times 10^3} = \underline{1380}$ kN

$P \times e = M_y$

$P = \frac{M}{e} = \frac{55 \times 10^6}{40}$ N

$= \frac{1.375 \times 10^6}{10^3}$ N $= 138$

$\therefore$ No: of cable in Y-dir. (middle strip)

$= \frac{1380}{100} = \underline{14}$

Spacing of cables $= \frac{0.75 \times \overset{l_x}{6} \times 1000}{14} = \underline{320mm}$

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

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Check for limit state of collapse

$\alpha_x (W_{ud}) l_x^2$

Ult. moment (x-dir) $= \overset{\alpha_x}{0.089} \times \underset{\uparrow \text{ultimate}}{9} \times 6^2 = \underline{29 \text{ kN m/m}}$

$\frac{\pi (5)^2}{4} = 19.63$

$A_p = \overset{\text{cable wires } 4 \times \frac{\pi d^2}{4}}{(4 \times 4 \times 20)} = \underline{320 mm^2}$

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

$$\overset{2}{(120-30)} \leftarrow d = (D - \text{cover})$$

$$D = 120$$
$$d = 120 - 30$$
$$\boxed{d = 90}$$

ref. Table (pg. 59)

$$\left(\frac{f_{pu}}{0.87 f_p}\right) = \underline{1.0}$$

$$f_{pu} = 1 \times 0.87 \times 1600 = \underline{1392\ N/mm^2}$$

Also $\left(\frac{x_u}{\underset{120-30}{d}}\right) = \underline{0.29}$

$$x_u = 0.29 \times 90 = \underline{26.1\,mm}$$

$$\boxed{M_u = f_p A_p (d - 0.42 x_u)} \quad (pg. 59)$$

$$= 1392 \times 320 \left(\frac{90 - 0.42 \times 26.1}{10^6}\right) = \underline{35.2\ kN\cdot m} > 29\ kN\,m/m$$

The ult. moment capacity of slab is higher than the min. value reqd. Similar check may be done in y-dir. also.

Check for deflection under service load

The tendons following a parabolic profile in x & y directions induce u.d.l.s acting u/w which are given by

M. Pe

$P \times e = \frac{Wl^2}{8}$

Equivalent load (x-dir) $= \frac{8Pe}{L_x^2} = \frac{8 \times 400 \times 0.03}{6^2} = 2.66$ kN/m

Equivalent load (y-dir) $= \frac{8Pe}{L_y^2} = \frac{8 \times 320 \times 0.01}{9^2} = 0.64$ kN/m

Unbalanced service load $= 6.00 - 2.66 - 0.64$

$= 2.7$ kN/m $= 0.0027$ N/mm²

Using deflection coefficients recommended by Timoshenko

for $\frac{L_y}{L_x} = 1.5$, deflection is given by

$$a_{max} = \alpha \left[\frac{q L_x^4}{D} \right] = \frac{5wl^4}{384EI} = \frac{5 \times 0.0027 \times 6000^4}{384 \times 38000}$$

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$$\frac{5 \times 2.7 \times 6^4}{384}$$

where

α = coeff = 0.00772

q = u.d.l = 0.0027 N/mm²

D = flexural rigidity $= \frac{Eh^3}{12(1 - \nu_c^2)}$

$$= \frac{38000 \times 120^3}{12(1 - 0.15^2)} = 5.62 \times 10^9$$

$\therefore\ a_{max} = 0.00772 \left(\frac{0.0027 \times 6000^4}{5.62 \times 10^9} \right) = 4.85$ mm

Max perm. long term deflection $= \frac{6000}{250} = 24$ mm $> a_{max}$

Hence safe.

Check for Stresses

unbalanced load = 2.7 kN/m²

$\alpha \times w \times l_x^2$

moment due to this load (x-dir) $= 0.089 \times 2.7 \times 6^2 = 8.7$ kN-m

Stress developed $= \dfrac{8.7 \times 10^6}{(1000 \times 120^2)/6} = 3.33$ N/mm²

$\frac{M}{Z}$ $\quad \frac{bh^2}{6}$

(comp at top + tension at soffit of slab)

Direct stress due to PS Force

$\frac{P}{A} = \dfrac{400 \times 10^3}{1000 \times 120} = 3.66$ N/mm² (comp)

$b \times h$

∴ Max. comp stress in concrete at the top of slab

$= 3.66 + 3.33 = 7$ N/mm² < per. stress of 15 N/mm²

∴ safe.

~~The max shear stress under the ult. load is~~

~~$= \dfrac{0.424 \times 9.8 \times 6000}{(1000 \times 90)} = 0.26$ N/mm² which~~

~~is negligibly small and hence no shear Reinforcement~~

~~are necessary.~~

$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}$

$f = \frac{My}{I}$

$f = \frac{M}{I/y}$

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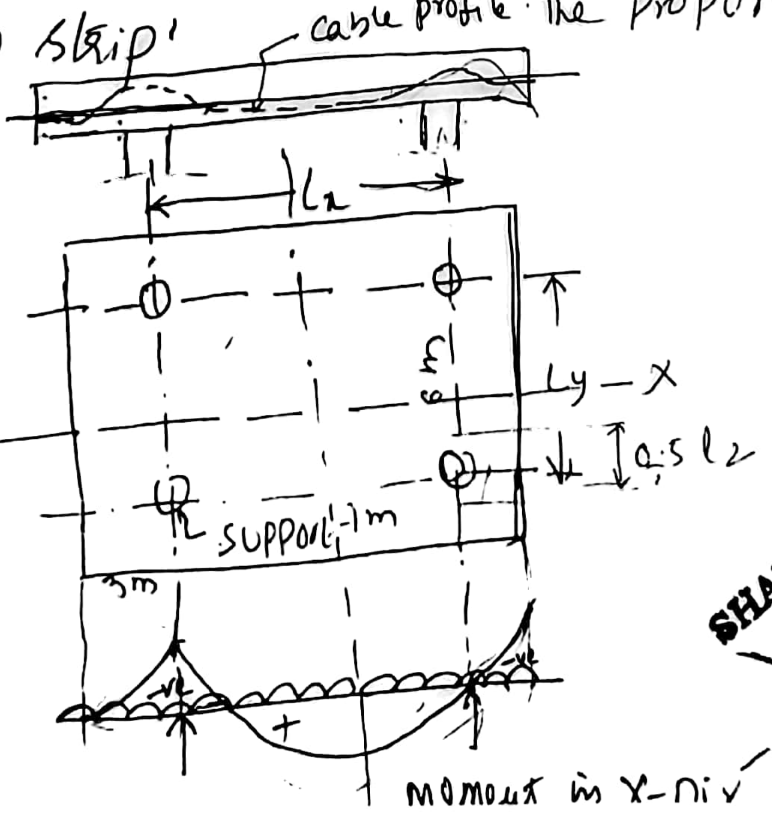
5m

FLATSLABS : A simple PS flat slab is generally supported by a network of columns without beams and prestressed in two perpendicular directions.

The design of a flat slab involves the analysis of moment in the two principal directions so that the cables may be arranged to resist these moments. The slab is analyzed as one way slab and the total number of cables reqd to resist the moments in each of the two principal directions are determined.

The column strip being stiffer than the middle strip, a greater percentage of tendons are housed in the column strip. The proportioning



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of tendons between the columns and middle strips may be based on the provision of codes such as IS1156 and BS 8110, where column strips share a higher proportion of the total ~~Amount~~ moment.

The total number of cables required in any direction is apportioned in the ratio of 65 and 35 percent b/w the column and middle strip

#A simple flat slab 12m by 9m is supported by four columns so placed as to form a symmetrical rectangular grid 7m x 6m. The cantilevers formed are 2.5m and 1.5m in the long and short directions of the slab. The live load on the slab is 1kN/m². Prestressing cables consisting of four wires of 5mm carrying an effective force of 100kN are available for use. Design the number of cables required and arrange them suitably in the two principal directions.

Sol Thickness of slab $= \frac{Span}{40} = \frac{6000}{40} = 150mm$

self wt of slab $= 0.15 \times 1 \times 1 \times 24 = 3.6 \, kN/m^2$

L.L on slab $= 1.0 \, kN/m^2$

finishes $= 0.4 \, kN/m^2$

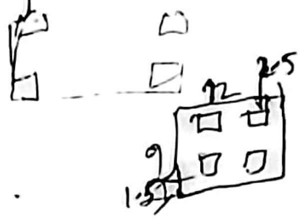
Total $5.0 \, kN/m^2$

Total load on four columns = $5 \times \overset{A}{\widehat{12 \times 9}}$
= 540 kN

$e = \frac{M}{P}$
$= \frac{\cancel{131} \times 10^3}{\cancel{1820} \times 10^3}$

Reaction on Each column = 135 kN $= \frac{540}{4}$

The slab is analysed for +ve & -ve moments in the long span (x-direc) and short span (y-direc)



Moments in the direction of long span ✓

Positive moment (centre of slab), $M_{xp} = (\overset{wl}{270} \times 3.5) - (270 \times 3)$?
= 135 kN-m

Negative moment (supports), $M_{xn} = (\overset{w \times L \times \frac{L}{2}}{2.5 \times 9 \times 5}) \times 1.25$
= 141 kN-m

The PS force reqd is designed to resist the max. moment of 141 kN-m in the x-direction.

The Cables are provided at a distance of 30 mm from the edge of the slab at critical sections. The cable profile is parabolic along the span so that the eccentricity is proportional to the moment at the section (ref. fig).

Total PS force reqd in the x-direction is given by $\boxed{e = 60}$

$P = \overset{M/e}{\frac{141 \times 10^3}{70}} = 2020\,kN$

Thickness of slab = $\frac{\text{span}}{40} = \frac{6000}{40} = 150\text{mm}$

Self wt of slab = 0.15 × 24 = 3.6 kN/m²

L.L on slab = 1.0 kN/m²

floor finish 0.4 kN/m²

5.0 kN/m²

ult. load $w_u = 1.5 \times 5.0$

$= 7.5 \text{ kN/m}^2$

Total

Moment along longer direction (x-dir) $l_1 = 7\text{m}$ $l_2 = 6\text{m}$

$$\text{Total B.M} = \frac{w_u l_x}{8} = \frac{w_u . l_2 . l_n^2}{8}$$

∵ l_n c/c dist after deduc. col. head

$$= \boxed{\frac{w_u . l_2 . l_1^2}{8}}$$

$$= \frac{7.5 \times 6 \times 7^2}{8} = 275.625 \text{ kN-m}$$

~~65% of moment is resisted by column strip~~

~~35% " " " " " middle strip~~

∴ ~~Total moment~~

As per IS 456 (31.4.3.2)

Total -ve design moment $M_{uo_{-ve}} = 0.65 \times 275.625 \simeq 180 \text{kNm}$

Total +ve " " $M_{uo_{+ve}} = 0.35 \times 275.625 \simeq 95.625\text{k}$

Distribution of Moment in column Strip & Middle Strip

-ve moment in col. Strip $M_{col} = 0.75 \times M_{uo}$

$= 0.75 \times 180 = -135$ kN-m — ①

-ve moment in middleship $M_{mid} = 0.25 \times 180 = -45$ kN-m -②

+ve moment in col Strip $= +\overset{0.63}{0.75} \times 95.625 = +71.72$ kN-m ③

+ve moment in middl " $= +0.25 \times 95.625 = +23.90$ kN-m ④

Highest $\overset{(-ve)}{\text{moment}}$ 135 kN-m in Col. Strip is considered for design.

Width of column Strip for this moment $\boxed{\not> 0.5 l_2 \not> 0.5 l_1}$

$0.5 \times 6 \not> 0.5 \times 7$

$= 3m$ $\quad 3.5$

∴ Total P.S force reqd $P \times e = M$ $(e = 70mm)$

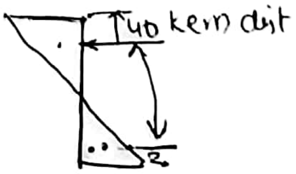
$$P = \frac{135 \times 10^3}{70} = 1928.57 \text{ kN}.$$

∴ No: of cable in col. Strip in x direction $= \dfrac{1928.57}{100\cancel{0}}$

$\simeq$ 20 bars in 3m width

Spacing $= \dfrac{3 \times 1000}{20} \approx 150$mm

$\dfrac{l}{n}$



Similarly along shorter directions (Y) $l_1 = 6m$; $l_2 = 7m$.

Total BM $= \frac{Nu.\, l_2 l_1^{2}}{8} = \frac{7.5 \times 7 \times 6^2}{8} = 236.25$ kN-m

Total -ve design moment $M_{uo_{-ve}} = 0.65 \times 236.25 = 153.56$k

Total +ve design moment $M_{uo_{+ve}} = 0.35 \times 236.25 = 82.68$k

<u>Distribution of moment in Col. Strip & middle strip</u>

-ve moment in col. strip $= 0.75 \times 153.56 = -115.17$ kN-m — ①

-ve moment in mid strip $= -0.25 \times 153.56 = -38.39$ kN-m — (2

+ve moment in col strip $= +0.75 \times 82.68 = 62.01$ kN-m — (3

+ve moment in midstrip $= +0.25 \times 82.68 = +20.67$ kN-m — ((

Highest (-ve) moment 115.17 kNm in Col. Strip is consid

width of column strip for this moment $= 0.5 l_2 \ngtr 0.5 l_1$

$= \underline{3m}$

$= 0.5 \times 7 \ngtr 0.5 \times 6$

$= 3.5 \ngtr 3m$

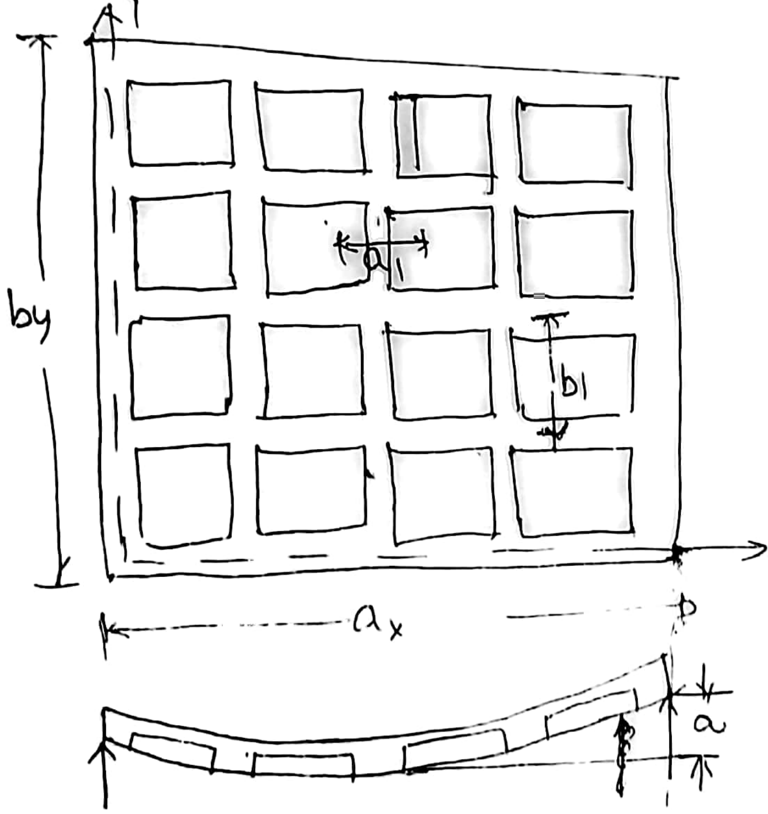
Total PS force Reqd $P = \frac{115.17 \times 10^3}{70} = 1645.28$ kN

No. of cables in col. strip (Y-dir) $= \frac{1645.28}{100} \simeq 16$

$\therefore$ Spacing $= \frac{3 \times 1000}{16} = 187 \simeq 190$ mm.

***ANALYSIS & DESIGN OF PS CONCRETE GRID FLOORS

A psc grid floor consists of ribs at close interval in two mutually perpendicular directions and connected by slab in b/w the ribs. It can be considered as an orthotropic plate freely supported on four sides



Def. Char. of Grid floods

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The vertical deflection 'a' at any point of the grid shown in fig: is expressed as

$$a = \frac{16 W d}{\pi^6} \frac{\sin\left(\frac{\pi x}{a_x}\right)\sin\left(\frac{\pi y}{b_y}\right)}{\left(\frac{D_x}{a_x^4}\right)+\left(\frac{C_x+C_y}{a_x^2 b_y^2}\right)+\left(\frac{D_y}{b_y^4}\right)}$$

Where w_d = total u.d.l. per unit area
a_x, b_y = length of plate in x & y directions
D_x, D_y = flexural rigidities in x & y Directions
C_x, C_y = torsional Rigidities.

If a_1, b_1 are the spacing of ribs in x & y directions then

$$D_x = \frac{EI_1}{b_1} \quad ; \quad C_x = \frac{C_1}{b_1}$$

$$D_y = \frac{EI_2}{a_1} \quad ; \quad C_y = \frac{C_2}{a_1}$$

Where EI_1 & EI_2 are flexural Rigidities of the effective section in x & y directions. " " " "
C_1 & C_2 are torsional Rigidities "

The moments and shears are computed using the following expressions.

$$M_x = -D_x\left(\frac{\partial^2 a}{\partial x^2}\right) \quad ; \quad M_y = -D_y\left(\frac{\partial^2 a}{\partial y^2}\right)$$

$$T_{xy} = -\frac{C_1}{b_1}\left(\frac{\partial^2 a}{\partial x \partial y}\right) \rightarrow T_{yx} = \frac{-C_2}{a_1}\left(\frac{\partial^2 a}{\partial x \partial y}\right)$$

$$Q_x = \frac{-\partial}{\partial x}\left[D_x\left(\frac{\partial^2 a}{\partial x^2}\right) + \frac{C_2}{a_1}\left(\frac{\partial^2 a}{\partial x \partial y}\right)\right]$$

$$Q_y = \frac{-\partial}{\partial y}\left[D_y\left(\frac{\partial^2 a}{\partial y^2}\right) + \frac{C_1}{b_1}\left(\frac{\partial^2 a}{\partial x \partial y}\right)\right]$$

Max BM occur @ centre of span while max. torsional moments are generated at the corners of the grid and max. SF develop at midpoints of longer side supports.

→ The min PS force & eccentricity reqd at the centre of span along the mutually perpendicular directions are determined.

Design a PSC grid floor to cover a rectangular panel of an office floor 12m x 7m. The SI load may be taken as 3.0kN/m^2 M40 grade concrete & Fressynet cables of 18-5mmø and 12-7mmø are available for use. Design the grid & sketch the details of profile of the cable in the ribs

Not Reqd.