

DESIGN FOR SHEAR & TORSIONShear and principle stresses:

The shear distribution in an uncracked structural member for which the deformation is assumed to be linear is the function of shear force and the properties of the cross-section of the member.

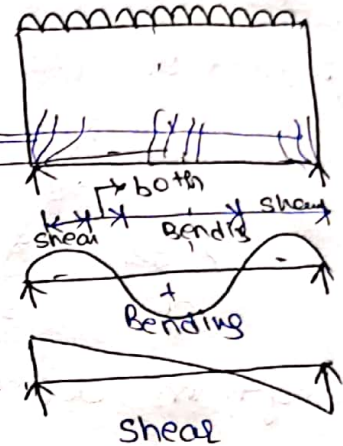
The shear stress at a point is expressed as

$$\tau_v = \frac{VA\bar{y}}{Ib} \quad (\text{or})$$

$$\tau_v = \frac{3}{2} \frac{Vu}{bd}$$

bending cracks.

Shear crack failure.



Where;

τ_v = Shearing stress due to transverse load.

V = Shear force

I = moment of Inertia.

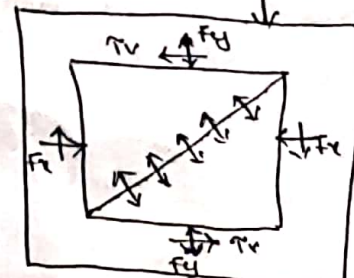
b = breadth of member at given point.

In a PSC member the shear stress is generally accompanied by a direct stress in the axial direction of the member.

If transverse, vertical prestressing is adopted the compressive stress in the direction perpendicular to the axis of the member will be present in addition to the axial pre-stress. ($V_u - P \sin \theta$)

The most general case often an element is subjected to a two-dimensional stress diagram shown in the figure.

Prestress in PSC member.



The maximum and minimum principle stress developed are given by σ_{max} , σ_{min} .

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

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

where;

σ_x & σ_y are direct stresses and τ_{xy} is shear stress acting at a point. in

In the PSC member the direct stresses σ_x & σ_y are compressive the magnitude of the principle tensile stresses is considerably reduced and in even some cases even eliminated. so that under working loads both major & minor principle stresses are compressive thereby eliminating the risk of diagonal tensional cracks.

In general there are 3 ways of improving the shear resistance of structural member concrete member by prestressing technique.

1. Horizontal (or) axial prestressing.
2. Prestressing by inclined (or) sloping.
3. Vertical (or) transverse Prestressing.

Sol: 10000

1. A PSC beam of span 10m of rectangular section 180mm wide & 300mm deep is axially prestressed by a cable carrying an effective force of 180kN. The beam supports a total UDL of 5kN/m which includes the selfweight of the member. compare the magnitude of the principle tension developed in the beam with and without axial prestressing.

Sol:

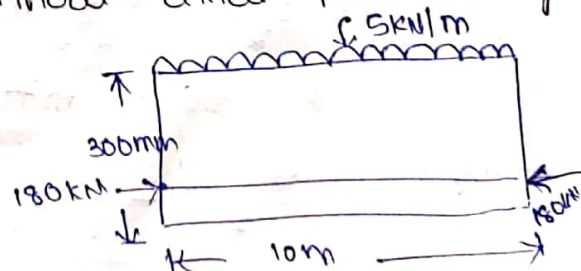
Span (L) = 10m.

Wide (b) = 180mm

deep (d) = 300mm.

Force (F) = 180 kN/m

UDL = 5 kN/m.



a) Without axial Prestressing:

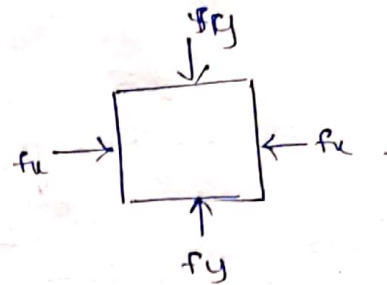
$$f_x = f_y = 0.$$

$$\tau_v = \frac{3}{8} \frac{VU}{bd}$$

$$V_u = \frac{Wl}{2} = \frac{5(10)}{2} = 25 \text{ KN.}$$

$$\tau_v = \frac{3}{8} \frac{(25 \times 10^3)}{120 \times 300}$$

$$\tau_v = 1.04 \text{ N/mm}^2$$



$$f_{\max}/f_{\min} = \frac{f_x + f_y}{2} \pm \sqrt{\left(\frac{f_x - f_y}{2}\right)^2 + \tau_v^2}$$

Here; $f_x = f_y = 0.$

$$f_{\max} (or) f_{\min} = \pm 1.04 \text{ N/mm}^2$$

b) With axial Prestressing

$$f_x = \frac{\text{Stress} \times \text{Area}_y}{\text{Area}_x} \quad f_y = 0.$$

$$\frac{P}{A} = \text{stress}$$

$$\frac{180 \times 10^3}{36 \times 10^3} = \text{stress}$$

$$f_x = 5 \text{ N/mm}^2$$

Principle stress

$f_{\max}, f_{\min}$:

$$f_{\max} = \frac{f_x + f_y}{2} + \sqrt{\left(\frac{f_x - f_y}{2}\right)^2 + \tau_v^2}$$

$$= \frac{5+0}{2} + \sqrt{\left(\frac{5-0}{2}\right)^2 + (1.04)^2}$$

$$= 5.2 \text{ N/mm}^2$$

$$f_{\min} = \frac{f_x + f_y}{2} - \sqrt{\left(\frac{f_x - f_y}{2}\right)^2 + \tau_v^2}$$

$$= \frac{5+0}{2} - \sqrt{\left(\frac{5-0}{2}\right)^2 + (1.04)^2}$$

$$f_{\min} = -0.804 \text{ N/mm}^2$$

19/2 Comparing magnitude of principal tension.

$$\frac{T_v - f_{min}}{T_v} \times 100$$

$$= \frac{1.041 - 0.2}{1.041} \times 100$$

$$= 80.78\%$$

2) From the above problem instead of axial prestressed cable having an eccentricity 100mm at the centre of the span & reducing to zero at supports is used. The effective force in the cable being 180kN. Estimate the percentage reduction in the principal tension in comparison with the case of axial Prestressing.

sol: Given data,

Eccentricity (e) = 100mm.

Force (F) = 180 kN.

l = 10m.

width (b) = 120mm

depth (d) = 300mm.

UDL = 5kN/m.

$\sqrt{u} = P \sin \theta$

$\sin \theta$ will be negligible ' θ '

$\sqrt{u} = P \theta$

$$\theta = \frac{4e}{L}$$

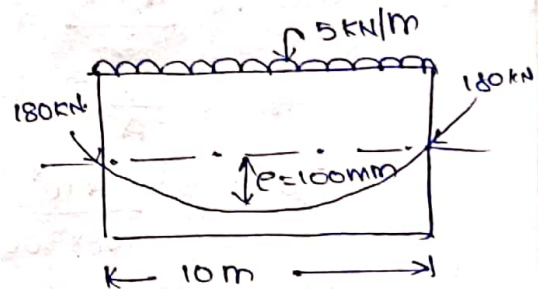
$$= \frac{4 \times 100}{10 \times 10^3} = 0.04 \text{ rad.}$$

The vertical component of the Prestressing force
= $P \sin \theta$.

For smaller values of θ , $\sin \theta$, similar to ' θ ' = $P \theta$.

$$= 180 \times 10^3 \times 0.04$$

$$P \theta = 7.2 \text{ kN}$$



Net shear at supports $V_u - P_0$.

$$V_u = \frac{w_l}{2} \\ = \frac{5(10)}{2} = 25 \text{ kN}$$

$$V_u - P_0 = 25 - 7.2$$

$$V_u = 17.8 \text{ kN}$$

maximum shear stress (τ_v)

$$\tau_v = \frac{3}{2} \frac{V_u}{bd} \\ = \frac{3}{2} \frac{17.8 \times 10^3}{120 \times 300}$$

$$\tau_v = 0.741 \text{ N/mm}^2$$

The maximum & minimum principal stress.

$$f_{\max} = \frac{f_x + f_y}{2} + \sqrt{\left(\frac{f_x - f_y}{2}\right)^2 + (\tau_v)^2}$$

$$f_x = \frac{P}{A} \\ = \frac{180 \times 10^3}{120 \times 300} \\ = 5 \text{ N/mm}^2$$

$$f_{\max} = \frac{5+0}{2} + \sqrt{\left(\frac{5-0}{2}\right)^2 + (0.741)^2} \\ = 5.10 \text{ N/mm}^2 \text{ (compression)}$$

$$f_{\min} = \frac{5+0}{2} - \sqrt{\left(\frac{5-0}{2}\right)^2 + (0.741)^2} \\ = -0.107 \text{ N/mm}^2 \text{ (tension)}$$

Compression both $f_{\min}$

$$\frac{+0.207 - (-0.107)}{0.207} \times 100$$

$$= 48\%$$

=

3. A concrete beam rectangular c/s section has a width of 250mm & depth of 600mm. The beam is prestressed by a parabolic cable carrying an effective force of 1000kN. The cable is concentric at supports and has maximum eccentricity 100mm at the centre of the span. The beam spans over 10m & supports a UDL live load 20kN/m. (a) Assuming the density of concrete is 24kN/m³, estimate the maximum principal stresses developed in a section of beam at a distance of 300mm from the supports. (b) The prestressing force required to nullify the shear force due to the dead & live loads at the support section.

sol:

Given data

$$P = 1000 \text{ kN}$$

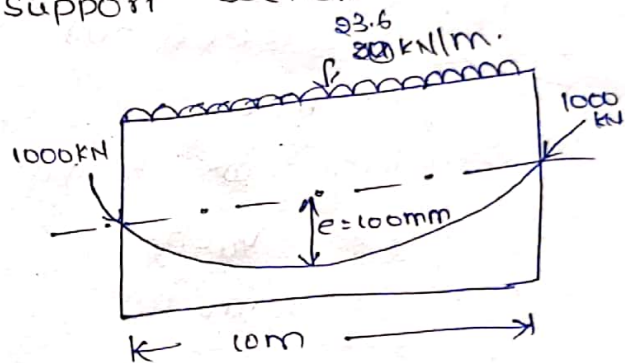
$$w = 20 \text{ kN/m}$$

$$e = 100 \text{ mm}$$

$$\rho = 24 \text{ kN/m}^3$$

$$\text{c/s} = 250 \times 600 \text{ mm}$$

$$l = 10 \text{ m}$$



The selfweight of the beam = Area * density of concrete.

$$= 0.25 \times 0.6 \times 24$$

$$= 3.6 \text{ kN/m}$$

Total load on beam (w) = both dead + live load

$$= 3.6 + 20$$

$$= 23.6 \text{ kN/m}$$

Shear force at support section.

$$V_u = \frac{wl}{2}$$

$$= \frac{23.6(10)}{2}$$

$$V_u = 118 \text{ kN}$$

Shear force at a section of 300mm from support

$$V_{at 300} = V_u - d w$$
$$= 118 - (0.3)(23.6)$$

$$V_{at 300} = 110.92 \text{ kN}$$

$$\theta = \frac{ue}{l} = \frac{4(100)}{10 \times 10^3}$$

$$\theta = 0.004 \text{ rad}$$

Vertical Component of the Prestressing force

$$P \sin \theta \approx P \theta$$

$$= 1000(0.004)$$

$$= 40 \text{ kN}$$

Net Shear force at 300mm from the support =

$$= V_u - P \theta$$

$$= 110.92 - 40$$

$$\boxed{V_u = 70.92 \text{ kN}}$$

a) The maximum shear stress at a distance of 300mm at the supports.

$$\tau_v = \frac{3}{2} \frac{V_u}{bd}$$

$$= \frac{3}{2} \frac{[70.92 \times 10^3]}{250 \times 600}$$

$$\boxed{\tau_v = 0.70 \text{ N/mm}^2}$$

The direct Prestressing force

$$f_x = \frac{P}{A} = \frac{1000 \times 10^3}{250 \times 600}$$

$$= 6.66 \text{ N/mm}^2$$

$$(\because f_y = 0)$$

$$f_{\max} = \frac{f_x + f_y}{2} + \sqrt{\left(\frac{f_x - f_y}{2}\right)^2 + \tau_v^2}$$

$$= \frac{6.66 + 0}{2} + \sqrt{\left(\frac{6.66 - 0}{2}\right)^2 + (0.70)^2}$$
$$= 6.73 \text{ N/mm}^2$$

$$f_{min} = \left(\frac{f_k + f_d}{2} \right) - \sqrt{\left(\frac{f_k - f_d}{2} \right)^2 + TV^2}$$

$$= \left(\frac{6.66 + 0}{2} \right) - \sqrt{\left(\frac{6.66 - 0}{2} \right)^2 + (0.70)^2}$$

$$= -0.79 \text{ N/mm}^2$$

b) If Prestressing force required to nullify the shear force at the ~~face~~ of the support due to dead and live load.

$$V_u - P \sin \theta = 0$$

$$V_u = P \sin \theta$$

$$= 1800(0.04)$$

$$\frac{118}{0.04} \times 10^3 = P$$

$$P = 2950 \text{ kN}$$

25/3/2020

4) A Prestressed T-section has the following properties:
 Area = $55 \times 10^3 \text{ mm}^2$; Second moment of Area (I) = $189 \times 10^7 \text{ mm}^4$;
 Statical moment about centroid = $468 \times 10^4 \text{ mm}^3$ ($A\bar{y}$)
 Thickness of web = 50mm. It is Prestressed horizontally by 24 wires of 5mm diameter and vertically by similar wires at 150mm centre, all the wires carry a tensile stress of 900 N/mm^2 . Calculate the principle stresses at the centroid when the shearing force of 80kN acts upon this section.

Q: Given data;

$$\text{Area (A)} = 55 \times 10^3 \text{ mm}^2$$

$$\text{Second moment of Area (I)} = 189 \times 10^7 \text{ mm}^4$$

$$A\bar{y} = 468 \times 10^4 \text{ mm}^3$$

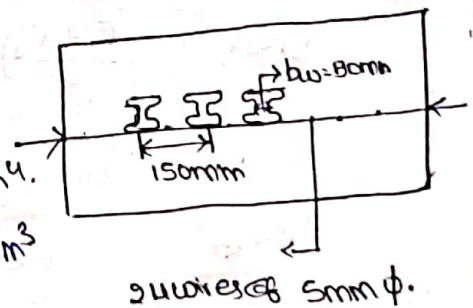
$$\text{Thickness of web} = 50 \text{ mm}$$

horizontal = 24 wires of 5mm ϕ

vertical = 24 wires of 5mm ϕ at 150mm centre.

$$\text{tensile stress} = 900 \text{ N/mm}^2$$

$$\text{shearing force} = 80 \text{ kN}$$



$$\tau_v = \frac{V A \bar{y}}{I b w A \bar{y}}$$

$$= \frac{80 \times 10^3 (468 \times 10^4)}{189 \times 10^7 (50) (468 \times 10^4)}$$

$$\tau_v = 3.96 \text{ N/mm}^2$$

Horizontal Prestress at centre:

$$f_x = \frac{P}{A}$$

P = stress $\times$ Area

$$= 900 \times 24 \cdot \frac{\pi}{4} (5)^2$$

$$= 424 \text{ kN}$$

$$= 424.11 \text{ kN}$$

$$f_x = \frac{424.11 \times 10^3}{55 \times 10^3} = 7.71 \text{ N/mm}^2$$

$$f_x = 7.71 \text{ N/mm}^2$$

$$f_y = \frac{P}{A} \Rightarrow P = \text{stress} \times \text{Area}$$

$$= 900 \times 150 \times 50 \cdot \frac{\pi}{4} (5)^2$$

$$= 6750 \text{ kN} = 17.67 \times 10^3 \text{ N}$$

$$f_y = \frac{17.67 \times 10^3}{750 \times 50}$$

$$f_y = 2.356 \text{ N/mm}^2$$

maximum principal stress:

$$f_{\max} = \left(\frac{f_x + f_y}{2} \right) + \sqrt{\left(\frac{f_x - f_y}{2} \right)^2 + \tau_v^2}$$

$$= \left(\frac{7.71 + 2.35}{2} \right) + \sqrt{\left(\frac{7.71 - 2.35}{2} \right)^2 + (3.96)^2}$$

$$f_{\max} = 9.81 \text{ N/mm}^2$$

minimum principal stress:

$$f_{\min} = \left(\frac{f_x + f_y}{2} \right) - \sqrt{\left(\frac{f_x - f_y}{2} \right)^2 + \tau_v^2}$$

$$= \left(\frac{7.71 + 2.35}{2} \right) - \sqrt{\left(\frac{7.71 - 2.35}{2} \right)^2 + (3.96)^2}$$

$$f_{\min} = 0.85 \text{ N/mm}^2$$

5) A cantilever portion of a prestressed concrete bridge has a rectangular cross-section 600mm wide & 1650mm deep is 8m long. carries a reaction of 350kN from the suspended span at free end together with a UDL of 60kN/m inclusive of its own weight. The beam is prestressed by 7 cables each carrying a force of 1000kN of which 3 are located at 150mm, 3 at 400mm & 1 at 750mm from the top edge. calculate the magnitude of the principle stresses at a point 550mm from the top cantilever at the support.

Section.

24/2/2020

sol: Given data.

c/s section of beam = 600mm x 1650mm.

long(l) = 8m.

Point load = 350kN at free end.

UDL = 60kN/m.

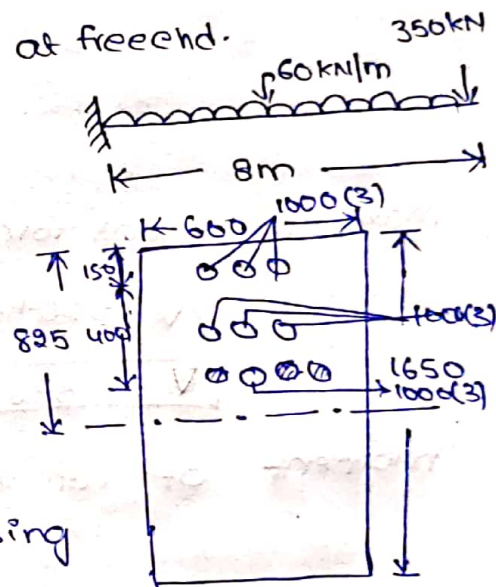
No of cables = 7.

3 at 150mm;

3 at 400mm;

1 at 750mm;

Force (F) = 1000kN.



4/3/2020

Centroid of the prestressing force from the top edge.

$$\bar{y} = \frac{A_1 x_1 + A_2 x_2 + A_3 x_3}{A_1 + A_2 + A_3}$$

$$y_1 = A_1 x_1 = 3000(150)$$

$$y_2 = A_2 x_2 = 3000(400)$$

$$y_3 = A_3 x_3 = 1000(750)$$

$$\bar{y} = \frac{A_1 x_1 + A_2 x_2 + A_3 x_3}{A_1 + A_2 + A_3}$$

$$= \frac{3000(150) + 3000(400) + 1000(750)}{3000 + 3000 + 1000}$$

$$\bar{y} = 342.85\text{mm}$$

$$\bar{y} = 342.85\text{mm}$$

$$\text{Eccentricity } (e) = \frac{d}{2} - \bar{y}$$

$$= \frac{1650}{2} - 342.815$$

$$e = 482.15 \text{ mm}$$

$$\text{Total Prestressing force } (P) = 3000 + 3000 + 1000$$

$$P = 7000 \text{ kN}$$

moment due to prestressing force = load $\times$ 11 and distance

$$m = P \times e$$

$$= 7000 \times 482.15$$

$$m = 3.37 \times 10^6 \text{ kN-mm}$$

$$m = 3.37 \times 10^6 \text{ kN-m}$$

$$m = 3375.05 \text{ kN-m}$$

maximum shear force of cantilever beam

$$P \times 350 + 60(8)$$

$$V = 830 \text{ kN}$$

moment of cantilever beam (m) = moment $\times$ distance

$$= 3375.05 \times \frac{8}{2}$$

$$= 350(8) \times \frac{8}{2}$$

$$m = 4720 \text{ kN-m}$$

moment due to live load + dead load is 4720 kN-m

$$f_x = \frac{P}{A} + \frac{Pe y}{I} - \frac{m y}{I} \quad (\text{max. resultant direct stress at } 550 \text{ mm from the top edge of support section}).$$

$$I = \frac{bd^3}{12} = \frac{600(1650)^3}{12}$$

$$I = 224.60 \times 10^9 \text{ mm}^4$$

$$f_x = \frac{830}{160}$$

$$y = 825 - 550$$

$$y = 275 \text{ mm}$$

$$= \frac{1650}{2}$$

$$= 825$$

$$F_x = \frac{7000 \times 10^3}{600(1650)} + \frac{3378 \times 10^6 \times 10^3 \times 275}{224.60 \times 10^9} - \frac{4720 \times 10^6 (275)}{224.60 \times 10^9}$$

$$= 7.07 + 4.12 - 5.4$$

$$F_x = 5.79 \text{ N/mm}^2$$

The maximum shear stress of 550mm from the top of the flange.

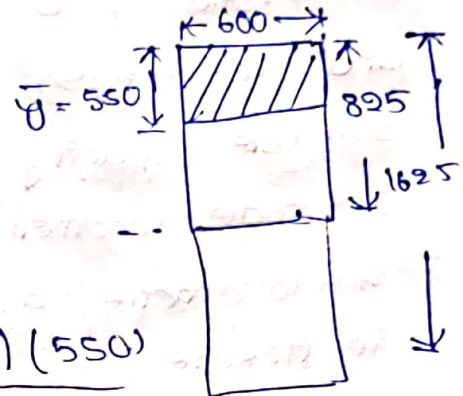
$$\tau_v = \frac{3}{2} \frac{V \bar{A} \bar{y}}{I_b}$$

$$A = 550(600)$$

$$A = 330 \times 10^3 \text{ mm}^2$$

$$\tau_v = \frac{(830 \times 10^3)(550 \times 600)(550)}{224.60 \times 10^9 \times 600}$$

$$\tau_v = 1.11 \text{ N/mm}^2$$



$$\therefore f_y = 0$$

maximum principal stress

$$f_{max} = \left(\frac{f_x + f_y}{2} \right) + \sqrt{\left(\frac{f_x - f_y}{2} \right)^2 + \tau_v^2}$$

$$= \left(\frac{5.79 + 0}{2} \right) + \sqrt{\left(\frac{5.79 - 0}{2} \right)^2 + (1.11)^2}$$

$$f_{max} = 5.99 \text{ N/mm}^2 \approx 6 \text{ N/mm}^2 \text{ (compression)}$$

minimum principal stress

$$f_{min} = \left(\frac{f_x + f_y}{2} \right) - \sqrt{\left(\frac{f_x - f_y}{2} \right)^2 + \tau_v^2}$$

$$= \left(\frac{5.79 + 0}{2} \right) - \sqrt{\left(\frac{5.79 - 0}{2} \right)^2 + (1.11)^2}$$

$$f_{min} = -0.205 \text{ N/mm}^2 \text{ (tension)}$$

SHEAR

1. A prestress member of $\square$ lar section girder of 150mm wide & 300mm deep to be designed to support and ultimate shear force of 130kN. The uniform prestress across the section is 5N/mm^2 given the characteristic cube strength of concrete as 40N/mm^2 and FeU15 HYSD bar of 8mm diameter. Design suitable spacing for stirrup confirming to the I.S code recommendations. Assume cover to the reinforcement as 50mm & the section uncracked in flexure.

Sol :

$$B = 150\text{mm}$$

$$D = 300\text{mm}$$

$$\text{Shear force (V)} = 130\text{kN}$$

ultimate shear

$$\text{uniform prestress } (f_p) = 5\text{N/mm}^2$$

$$f_{ck} = 40\text{N/mm}^2$$

FeU15 HYSD bar of 8mm ϕ

effective. cover = 50mm.

uncracked section in flexure:

$$V_c = V_{co}$$

$\therefore$ Pg No. 32

23.4.1

$$V_{co} = 0.67 b D \sqrt{f_t^2 + 0.8 f_{cp} f_t}$$

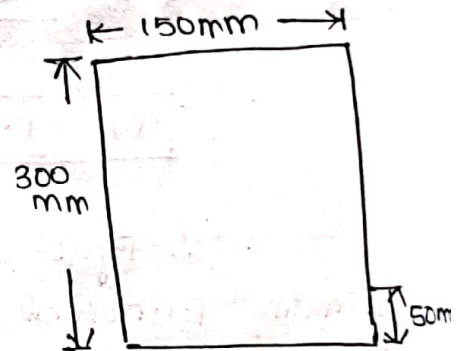
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$$f_t = 0.24 \sqrt{f_{ck}}$$

$$= 0.24 \sqrt{40}$$

$$f_t = 1.51\text{N/mm}^2$$

$$f_{cp} = 5\text{N/mm}^2$$



$$V_{c6} = 0.67 \times 150 \times 300 \sqrt{(1.51)^2 + (0.8)(5)(1.51)}$$

$$V_{c6} = 86.96 \text{ kN}$$

$$V > V_{c6} \text{ (or) } V/c$$

shear force (130) > 86.96

∴ provide shear reinforcement as per cl No-23.4

3.2
Pg: 33

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 f_{yd} t}$$

Assume 2 legged 8mm ϕ vertical stirrups.

$$A_{sv} = 2 \cdot \frac{\pi}{4} (8)^2 = 100.53 \text{ mm}^2$$

$$\frac{100.53}{S_v} = \frac{130 - 86.96}{0.87 \times 415 \times 250} \quad dt = d - \text{cover}$$

$$= 300 - 50$$

$$= 250 \text{ mm}$$

$$S_v = \frac{100.53 \times 0.87 \times 415 \times 250}{130 \times 10^3 - 86.96 \times 10^3}$$

$$S_v = 210.80 \text{ mm}$$

$$S_v \nless 0.75 dt$$

$$= 0.75 (250)$$

$$= 187.5 \text{ mm}$$

$$S_v \nless 4bw = 4(150) = 600$$

consider less value

∴ provide 2 legged 8mm ϕ at 187.5 mm c/c

6/3/2020.

3) A Pretensioned beam of rectangular c/s section 250mm x 550mm has an effective prestressing force of 900kN at an constant eccentricity of 800mm. It carries a total service load of 25.8 kN/m over an effective span of 11m. Design a shear reinforcement for the beam. The grade of concrete is M40. Design a shear reinforcement at support section & at 1/4th of the span.

Sol:

Cracking Flexural:

Given data;

$$B = 250 \text{ mm}$$

$$D = 550 \text{ mm}$$

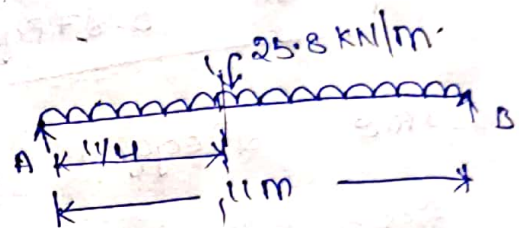
$$\text{force} = 900 \text{ kN}$$

$$e = 800 \text{ mm}$$

$$\text{load}(P) = 25.8 \text{ kN/m}$$

$$\text{span}(L) = 11 \text{ m}$$

$$\text{grade of concrete} = \text{M40}$$



At 1/4 span:

$$\begin{aligned} \text{shear force} &= \frac{wL}{2} - wx \\ &= \frac{25.8 \times 11}{2} - \left(\frac{11}{4}\right) 25.8 \\ &= \frac{25.8 \times 11}{2} - \left(\frac{11}{4}\right) 25.8 \end{aligned}$$

$$V = 70.95 \text{ kN}$$

$$R_A = \frac{25.8(11)}{2}$$

$$\begin{aligned} \text{moment (m)} &= R_A x - w \cdot x \cdot \frac{x}{2} \quad (\text{or}) \quad \frac{3wL^2}{32} \\ &= 11 \times (12.9) \times \frac{11}{4} - (25.8) \frac{11}{4} \times \left(\frac{11}{2}\right) \\ &= 390.225 - 97.56 \\ &= 292.66 \text{ kN-m} \end{aligned}$$

assume pre stressing $(f_p) = 1600 \text{ N/mm}^2$ in steel.

cracking flexural

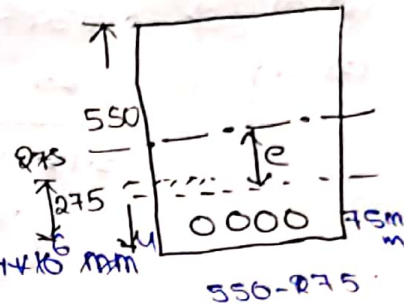
$$\lambda_{cr} = \left(1 - 0.55 \frac{f_{pe}}{f_p}\right) \gamma_o b d + m_o \frac{y}{m}$$

pg: 47

$$\begin{aligned} f_{pe} &= 0.6 f_p \\ &= 0.6 (1600) \\ &= 960 \text{ N/mm}^2 \end{aligned}$$

$$\Rightarrow \boxed{m_o = 0.8 f_{pt} \cdot \frac{I}{y}}$$

$$I = \frac{b d^3}{12} = \frac{250 (550)^3}{12} = 346.614 \times 10^6 \text{ mm}^4$$



$$I = 2232.74 \times 10^6 \text{ mm}^4$$

$$\begin{aligned} f_{pt} &= \frac{P}{A} + \frac{P e y}{I} \\ &= \frac{900 \times 10^3}{250 \times 550} + \frac{900 \times 10^3 \times 200 \times 275}{2232.74 \times 10^6} \end{aligned}$$

$$y = 275 - 75 = 200 \text{ mm}$$

$$f_{pt} = 6.54 + 16.12$$

$$\boxed{f_{pt} = 22.66 \text{ N/mm}^2}$$

$$\begin{aligned} m_o &= 0.8 f_{pt} \cdot \frac{I}{y} \\ &= 0.8 (22.66) \frac{(2232.74 \times 10^6)}{200} \end{aligned}$$

$$\boxed{m_o = 202.37 \text{ kN-m}}$$

→ for γ :

$$\frac{100 \frac{A_p}{b d}}{b d} = A_p e$$

∴ from table 6 : pg. 47.

$$A_p = \frac{P}{f_{pe}}$$

$$= \frac{900 \times 10^3}{960}$$

$$A_p = 937.5 \text{ mm}^2$$

$$100 \left(\frac{937.5}{250 \times 475} \right) = 0.789$$

for m40:

$$0.75 - 0.6 \rightarrow (1)$$

$$0.789 - \gamma \rightarrow (2)$$

$$1.00 - 0.68 \rightarrow (3)$$

$$(2)-(1) \Rightarrow 0.039 \times (\gamma - 0.6)$$

$$(3)-(1) \Rightarrow 0.25 - 0.08$$

$$3.12 \times 10^{-3} = 0.25\gamma - 0.15$$

$$\boxed{\gamma_c = 0.61 \text{ N/mm}^2}$$

$$V_{cr} = \left[1 - 0.55 \frac{f_{pe}}{f_p} \right] \gamma_{cbd} + m_0 \cdot \frac{V}{m}$$

$$= \left[1 - 0.55 \frac{960}{1600} \right] (0.61) (256) (475) + 208.3 \times 10^6$$

$$\frac{70.95 \times 10^3}{208.66 \times 10^6}$$

$$= 97.58 \text{ kN}$$

9/3/2020

At center Support Section:

$$\text{Shear force (V)} = 11$$

we have to provide shear reinforcement.

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_v}$$

Assume A_{sv} = 2 legged 8mm ϕ

$$A_{sv} = 2 \left(\frac{\pi}{4} \right) (8^2) = 100.53 \text{ mm}^2$$

$$\frac{100.53}{250 (sv)} = \frac{0.4}{0.87 (415)}$$

$$\boxed{sv = 362.96 \text{ mm}}$$

at support section:

$$\text{shear force } (V) = \frac{wl}{2}$$

$$= \frac{25.8(11)}{2}$$

$$V_e = 141.9 \text{ kN}$$

(Pg: 47)

$$V_{cr} = 0.16bd\sqrt{f_{ck}}$$

$$= (0.1)(250)(475)\sqrt{40}$$

$$V_{cr} = 75.10 \text{ kN}$$

$$V > V_{cr} \text{ (or) } V_c$$

we have to provide shear reinforcement

22.4.3.2

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 f_{yd} t}$$

Assume A_{sv} is 2-legged 8-mm ϕ .

$$A_{sv} = 2 \cdot \frac{\pi}{4} (8)^2 = 100.53 \text{ mm}^2$$

$$\frac{100.53}{S_v} = \frac{(141.9 \times 10^3) - (75.10 \times 10^3)}{0.87 (415) (475)}$$

$$\frac{100.53}{S_v} = 0.38$$

$$S_v = 264.55 \text{ mm}$$

check:

$$S_v \neq 0.75d$$

$$0.75(475)$$

$$356.25$$

$$SV \neq 4bw \Rightarrow 4(250) \\ = 1000$$

consider less value :

2 legged $8\text{mm}\phi$ at 264.5mm cross-section.