

DESIGN FOR FLEXURAL RESISTANCETypes of Flexural Failure:-

It depends upon

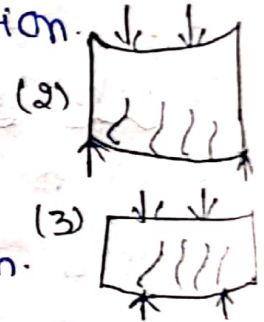
1. % of reinforcement in the section.
2. Bond b/w concrete and steel/tension.
3. Compressive strength of concrete.
4. Ultimate tensile strength of tendon.

Types :-

1. Fracture of Steel in tendon
2. Failure of under reinforcement section.
3. Failure of over reinforced section.
4. Other modes of failure

Pretensioned $\rightarrow$ Inadequate transmission length.

Post tensioned $\rightarrow$ Anchorage failure

IS code provisions :-

IS:1343-1980 : Appendix-B
Pg:59,60.

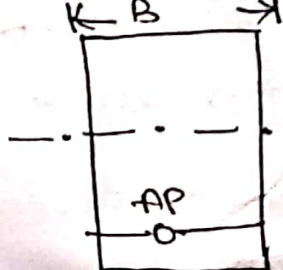
Rectangular Section:-

The Indian Standard code method for computing the flexural strength of rectangular sections (or) T-sections in which the neutral axis lies within the flange, is based on the rectangle and

Parabolic stress

blocks

parameters.



A.P.fpu

$0.446 \sigma_{st}$
 $0.419 \sigma_{st}$
 $0.26 f_{ck} b x_u$

$$M = f_{pu} A_p [d - 0.42 x_u]$$

where,

m = moment of resistance of the section.

f_{pu} = ultimate tensile stress in the tendons.

A_p = Area of Prestressing tendon.

d = effective depth.

x_u = neutral axis depth.

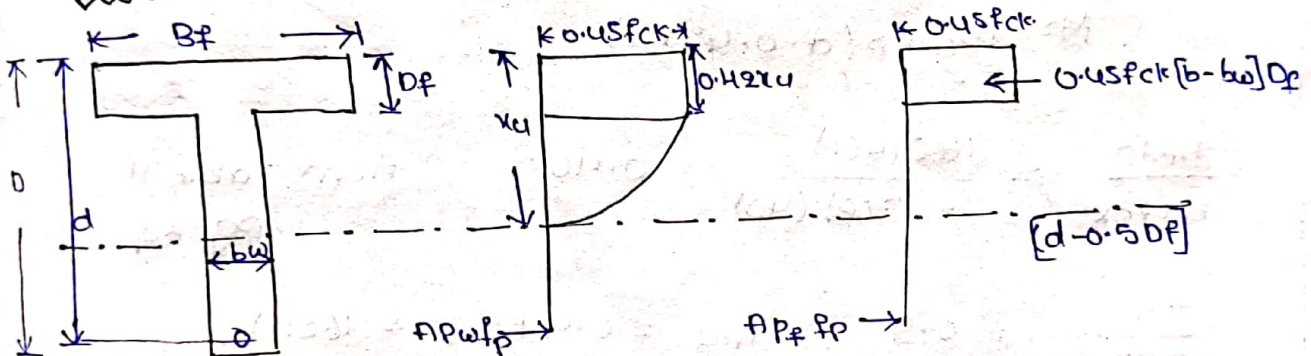
Based on the value of $\frac{A_p f_{pu}}{b d f_{ck}}$.

The value of $\frac{A_p f_{pu}}{b d f_{ck}}$ are obtained from Table: 11 of IS: 1343-1980 NXB Pg: 59 & 60.

11/2/2020

$$f_{pu} \geq 0.45 f_b$$

T-section: [moment of resistance of flanged section]



where $x_u > D_f$

$$M_u = f_{pu} A_{pw} [d - 0.42 x_u] + 0.45 f_{ck} [b - b_w] D_f [d - 0.5 D_f]$$

$$A_{pf} = 0.45 f_{ck} [b - b_w] \left[\frac{D_f}{f_b} \right]$$

$$\text{then } A_{pw} = A_p - A_{pf}$$

effective reinforcement ratio

$$\frac{A_{pw} f_p}{b d f_{ck}} \quad (IS: 1343-1980)$$

A_{pw} = Area of Prestressing steel for web.
 A_{pf} = Area of Prestressing steel for flange.

D_f = thickness of flange.

b_w = thickness of web.

The corresponding values of $\frac{f_{pu}}{0.87 f_b}$ & $\frac{x_u}{d}$ are obtained from table 11 pg: 59.

of 50mm. If $f_{ck} = 40 \text{ N/mm}^2$ and $f_p = 1600 \text{ N/mm}^2$ and the area of prestressing steel (A_p) = 461 mm^2 . Calculate the Ultimate flexural Strength of the Section using IS code method.

Sol: Given data,

Wide (b) = 150mm

Deep (D) = 350mm

effective cover = 50mm

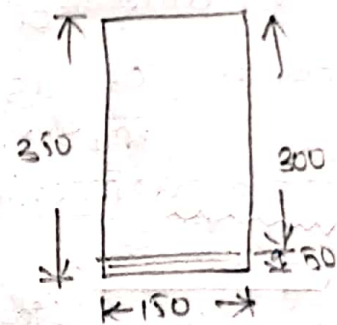
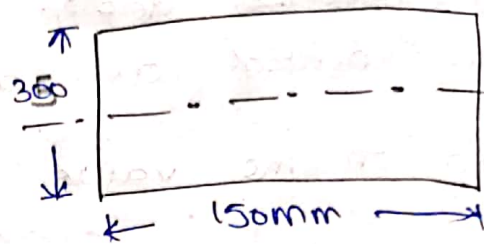
$f_{ck} = 40 \text{ N/mm}^2$

$f_p = 1600 \text{ N/mm}^2$

$A_p = 461 \text{ mm}^2$

effective depth (d) = $350 - 50 = 300 \text{ mm}$

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



$$\frac{A_p f_p}{b d f_{ck}} = \frac{1600 (461)}{150 (300) (40)} = 0.40$$

From Table 11
Pg: 59

$$\frac{f_{pu}}{0.87 f_p} = 0.9 \Rightarrow f_{pu} = 0.9 (0.87 * 1600)$$

$$f_{pu} = 1252.8 \text{ N/mm}^2$$

$$\frac{x_u}{d} = 0.783 \Rightarrow 0.783 (300) = 234.9 \text{ mm}$$

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

$$= 1252.8 (461) [300 - (0.42 * 234.9)]$$

$$= 577540.13$$

$$M = 116.46 \text{ kN-m}$$

12/10/20

2) A pretensioned T-section has a flange of 1200mm wide & 150mm thick. The width & depth of the Rib 300mm and 150mm resp. The high tensile steel has an area of 4700 mm^2 and is located at an effective depth of 1600mm. If the characteristic strength of the cube concrete & tensile strength of steel is

40 N/mm² & 1600 N/mm² resp. Calculate the flexural strength of T-Section.

Given data,

$$b_f = 1200 \text{ mm}$$

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

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

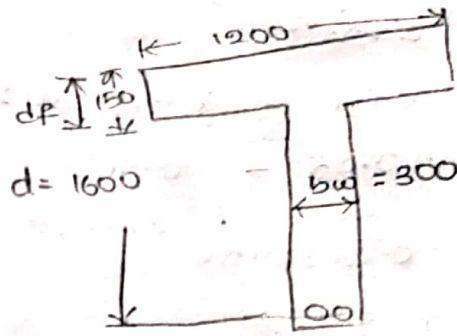
$$b_{cd} = 150 \text{ mm}$$

$$\text{Area } (A_p) = 4700 \text{ mm}^2$$

$$\text{effective depth } (d) = 1600 \text{ mm}$$

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

$$\text{Tensile strength } (f_p) = 1600 \text{ N/mm}^2$$



$$\text{Effective reinforcement ratio} = \frac{A_p w f_p}{b_w d f_{ck}}$$

$$= \frac{(4700)(300)(1600)}{(1200)(1600)(40)}$$

$$A_{pw} = A_p - A_{pf}$$

$$\text{then } A_{pf} = 0.45 f_{ck} [b_f - b_w] \left[\frac{d_f}{f_p} \right]$$

$$= 0.45(40) [1200 - 300] \left[\frac{150}{1600} \right]$$

$$= 1518.75 \text{ mm}^2$$

$$A_{pw} = 4700 - 1518.75$$

$$= 3181.25 \text{ mm}^2$$

$$\Rightarrow \frac{3181.25(1600)}{300(1600)(40)} = 0.26$$

$$\begin{array}{l} 0.25 - 1 \\ 0.30 - 1 \end{array} \quad \text{Interpolation} = 1.0$$

$$\frac{f_{pu}}{0.87 f_p} = 1$$

$$\Rightarrow f_{pu} = (1)(0.87)(1600) = 1392 \text{ N/mm}^2$$

$$\frac{x_u}{d} = 0.265$$

$$0.25 - 0.542 \rightarrow (1)$$

$$0.26 - \text{?} \rightarrow (2)$$

$$0.30 - 0.655 \rightarrow (3)$$

$$(2) - (1) \quad 0.01 = x - (0.542)$$

$$(3) - (1) \quad 0.05 = 0.113$$

$$x = \frac{1.1 \times 10^3 + 0.027}{0.05}$$

$$\boxed{x = 0.56}$$

$$0.26 = 0.56$$

$$\frac{x_u}{d} = 0.56$$

$$x_u = 0.56(1600) = 896 \text{ mm}$$

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

where

$$x_u > D_f$$

$$M_u = f_{pu} A_{pw} [d - 0.42 x_u] + 0.45 f_{ck} [b - b_w] D_f [d - 0.5 D_f]$$

$$= (1392)(3181.25) [1600 - 0.42(896)] + 0.45(40) [1200 - 300] 150 [1600 - 0.5(150)]$$

$$= 4428300 (122368) + 16200 (228750)$$

$$M_u = 9124.5 \text{ kN-m}$$

$$\boxed{M_u = 9124.5 \text{ kN-m}}$$

7/12/2020

3) A post tensioned prestressed concrete T-beam having a flange width of 1200mm and thickness of flange 200mm, thickness of web deep be 300mm is prestressed by 2000mm² of high tensile steel located at an effective depth of 1600mm. If $f_{ck} = 40 \text{ N/mm}^2$, $f_p = 1600 \text{ N/mm}^2$, estimate the ultimate flexural strength of the unbonded T-section - Assuming span to depth ratio as 20 and $f_{pe} = 1000 \text{ N/mm}^2$.

sol: Given data.

$$b_f = 1200 \text{ mm}$$

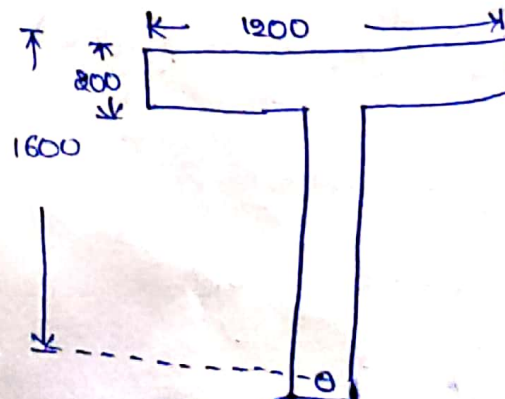
$$d_f = 800 \text{ mm}$$

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

$$A_p = 2000 \text{ mm}^2$$

$$\text{effective depth } (d) = 1600 \text{ mm}$$

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



$$f_{pe} = 1600 \text{ N/mm}^2$$

$$\frac{l}{d} = 20$$

Assuming the neutral axis to fall within the flange. We have the ratio of

$$= \frac{A_p f_{pe}}{b d f_{ck}}$$

$$= \frac{2000 (1000)}{1800 \times 1600 \times 40}$$

$$= 0.026$$

$$0.025 - 1.34$$

$$0.026 - x$$

$$0.05 - 1.32$$

consider 1.34.

$$\Rightarrow \frac{f_{pu}}{f_{pe}} = 1.34$$

$$f_{pu} = 1.34 [1000] = 1340 \text{ N/mm}^2$$

$$\Rightarrow \frac{x_u}{d} = 0.10$$

$$x_u = 0.10 (1600)$$

$$x_u = 160 \text{ mm}$$

$$x_u < d_f$$

consider

$$M = f_{pu} A_p [d - 0.42 x_u]$$

$$= 1340 (2000) [1600 - 0.42 (160)]$$

$$= 2680000 (1532.8)$$

$$M = 4107 \text{ kN-m}$$

4/A Post-tensioned bridge girder with unbonded tendon is of box section of overall dimension 1200mm wide & 1800mm deep with wall thickness of 150mm and high tensile steel has an area of 4000mm² and is located at an effective depth of 1600mm. The effective prestress in steel after all losses is 1000N/mm² (f_{pe}), effective span of girder is 84m. If $f_{ck} = 40 \text{ N/mm}^2$ and $f_{pu} = 1600 \text{ N/mm}^2$. Estimate the ultimate flexural strength of section. Assume $b_w = 300 \text{ mm}$.

Sol: Given data:

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

$$-A_p = 4000 \text{ mm}^2$$

$$\text{effective depth } (d) = 1600 \text{ mm}$$

$$f_{pe} = 1000 \text{ N/mm}^2$$

$$\text{span length} = 24 \text{ m}$$

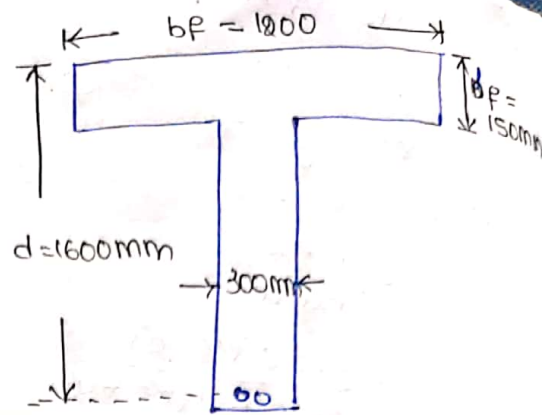
$$b_f = 1200 \text{ mm} ; D_f = 150 \text{ mm}$$

$$d_f = 1800 \text{ mm}$$

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

$$f_{pu} = 1600 \text{ N/mm}^2$$

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



$$\text{Effective reinforcement ratio} = \frac{A_{pw} f_p}{b d f_{ck}}$$

$$A_{pw} = A_p - A_{pf}$$

$$\begin{aligned} A_{pf} &= 0.45 f_{ck} [b_f - b_w] \left[\frac{D_f}{f_p} \right] \\ &= 0.45 (40) [1200 - 300] \left[\frac{150}{1600} \right] \\ &= 1518.75 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} A_{pw} &= A_p - A_{pf} \\ &= 4000 - 1518.75 \\ &= 2481.25 \text{ mm}^2 \end{aligned}$$

$$\frac{2481.25 (1600)}{300 (1600) (40)} = 0.129$$

$$\frac{l}{d} = \frac{24 \times 10^3}{1600} = 15$$

$$\text{Consider } \frac{l}{d} = 10$$

$$0.10 \rightarrow 1.45 \rightarrow (1)$$

$$0.129 \rightarrow x \rightarrow (2)$$

$$0.15 \rightarrow 1.36 \rightarrow (3)$$

$$(2) - (1) \Rightarrow 0.029 = x - 1.45$$

$$(3) - (1) \Rightarrow 0.05 = -0.09$$

$$\begin{aligned} -0.0018 &= 0.05x - 0.0725 \\ -2.61 \times 10^{-3} & \end{aligned}$$

$d_f = 150 \text{ mm}$

$$\gamma = 1.38$$

$$\frac{f_{pu}}{f_{pe}} = 1.38$$

$$f_{pu} = 1.38 (1000) = 1380 \text{ N/mm}^2$$

$$\frac{x_u}{d} = 0.129$$

$$0.10 - 0.36$$

$$0.129 - \gamma$$

$$0.15 - 0.52$$

$$0.52 + \frac{[0.52 - 0.36]}{0.15 - 0.10} \times (0.129 - 0.15) = 0.45$$

$$= 0.45$$

$$\frac{x_u}{d} = 0.45$$

$$x_u = 0.45 (1600)$$

$$= 720 \text{ mm}$$

$$x_u > D_f$$

$$M_u = f_{pu} P_w [d - 0.42 x_u] + 0.45 f_{ck} [b - b_w] D_f [d - 0.5 D_f]$$

$$= (1380) (2481.25) [1600 - 0.42 (720)] +$$

$$0.45 (40) [1200 - 300] 150 [1600 - 0.5 (150)]$$

$$= 4443.4 \times 10^6 + 3705.75 \times 10^6$$

$$M_u = 8148.4 \text{ kN-m}$$

10/2/2020

Main reasons for controlling deflection:

Large deflections:

→ Under dynamic effect and unkr

→ Under variable loading.

Excessive deflections:

→ Damage to finishers

→ Partitions

→ Associated structures.

Factors influencing deflection:

- 1) Imposed load and selfweight.
- 2) Magnitude of Prestressing force.
- 3) Span of member.
- 4) Cable profile.
- 5) Young's modulus of concrete.
- 6) Moment of Inertia (or) 2nd moment of area of cross-section.
- 7) Shrinkage, creep and relaxation of stress.
- 8) Fixidity conditions.

Calculation of deflections:

These are two types.

1. Post cracking

2. Pre cracking.

→ It is similar R.C.C.
R.C.C is " to P.S.C.

→ From whole section is considered moment using Mohr's 2nd theorem.

→ moment curvature

Deflection calculation: 2nd moment area of c/s section
(or) 2nd moment Inertia.

Mohr's first law:

$$\text{slope} = \frac{\text{Area of BMD}}{\text{Flexural rigidity.}}$$

$$\theta = \frac{A}{EI}$$

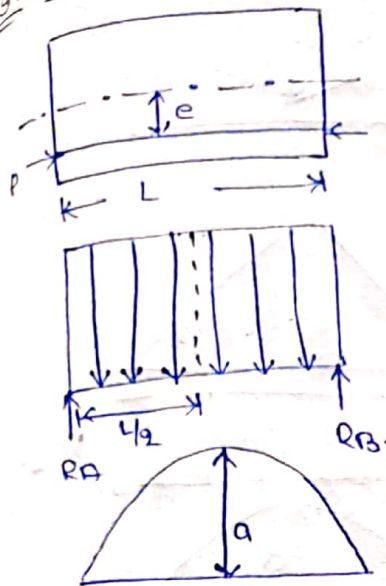
Mohr's second law:

$$\Delta = \frac{\text{moment of area of BMD}}{\text{flexural rigidity}}$$

$$= \frac{A \bar{x}}{EI}$$

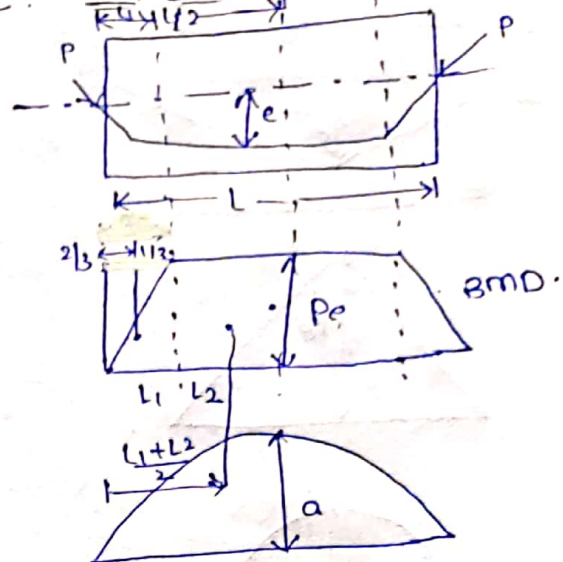
Effect of tendon profile on deflection:-

straight tendon:



$$a = \frac{PeL^2}{8EI} \text{ (upward).}$$

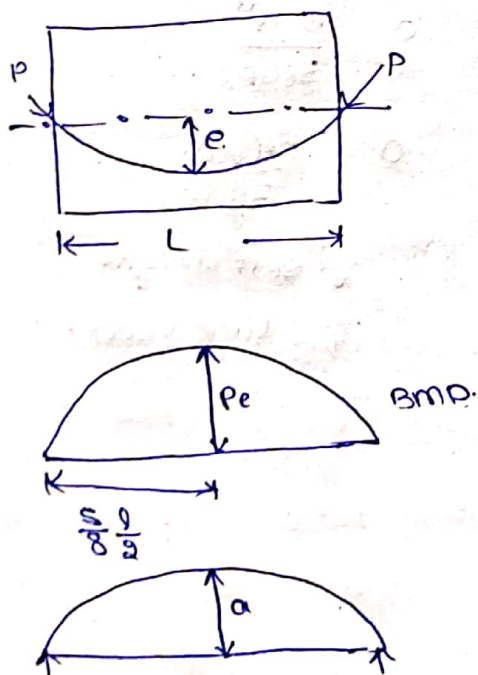
11/9/2020 Trapezoidal Tendon:



$$a = -\frac{Pe}{6EI} [2l_1^2 + 6l_1l_2 + 3l_2^2]$$

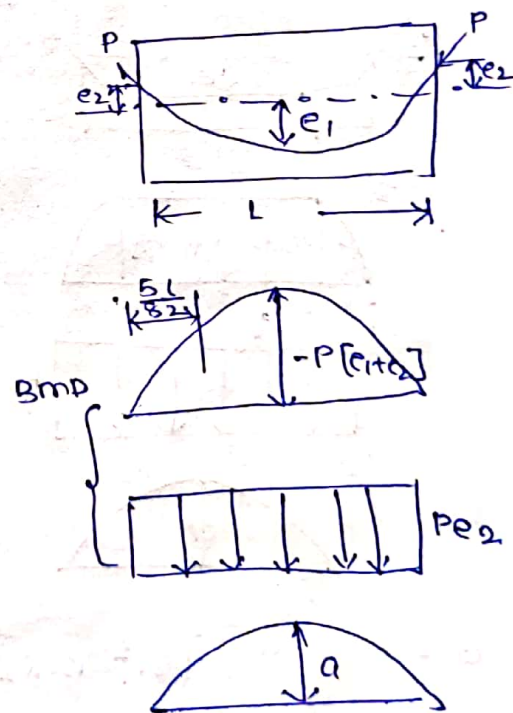
3. Parabolic tendon

1) center



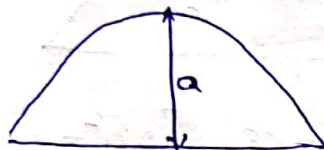
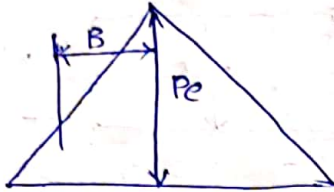
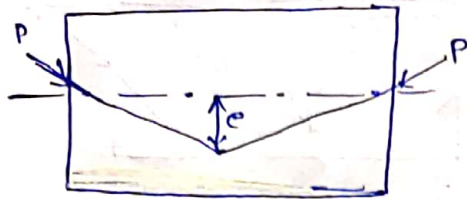
$$a = -\frac{5PeL^2}{48EI}$$

2) eccentric



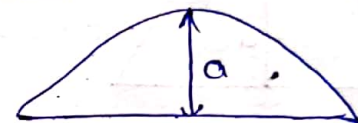
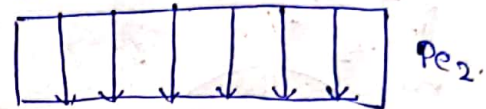
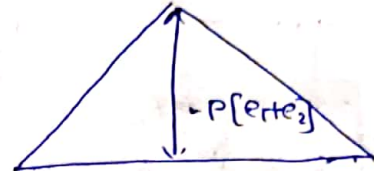
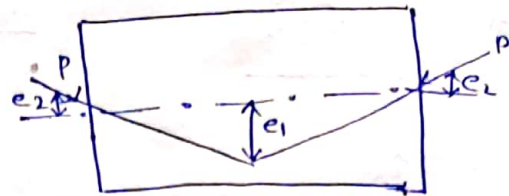
$$a = -\frac{Pl^2}{48EI} [-5e_1 + e_2]$$

Triangular Tendon:



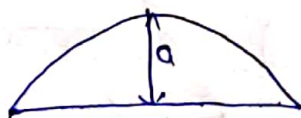
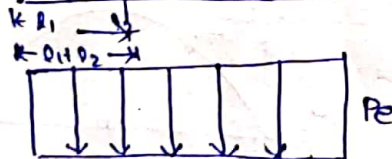
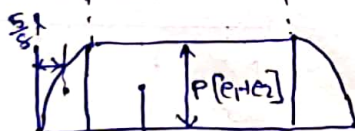
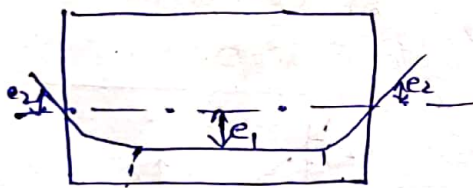
$$a = \frac{Pe l^2}{12EI}$$

slopping tendon



$$a = \frac{Pl^2}{24EI} [-2e_1 + e_2]$$

Parabolic Straight tendon:



$$a = \frac{-P[e_1 + e_2]}{12EI} [5l_1^2 + 120l_1l_2 + 6l_2^2] + \frac{Pe_2l^2}{8EI}$$

Deflection due to self weight & imposed load

$$a = \frac{5wl^4}{384EI}$$

$$a = \frac{5(q+q_l)l^4}{384EI}$$

q = self weight

q_l = live load.

The deck of a Prestressed concrete column is made up of a slab 500mm thick. The slab is spanning over 10.4m and supports a total UDL comprising the dead & live loads of 23.5 kN/m. The modulus of elasticity of concrete (E_c) = 38 kN/mm². The concrete slab is prestressed by straight cables each containing 12 high tensile wires of 7mm dia. is stressed to 1200 N/mm² at a constant eccentricity of 195mm. The cables are spaced at 328mm intervals in the transverse direction. Estimate the instantaneous deflections of the slab at the centre of the span under Prestressed & imposed loads.

12/3/2020

Sol: Given data,

Thickness of slab = 500 mm

stress

$$= 1200 \text{ N/mm}^2$$

Span length (l) = 10.4m.

Load (dead + live) ($q + q_l$) = 23.5 kN/m.

$$E_c = 38 \text{ kN/mm}^2$$

No. of wires = 12 wires - 7mm ϕ .

eccentricity (e) = 195mm.

Spacing of cable in transverse direction = 328mm.

Assume width (b) = 1000mm.

Force in each cable (F) = stress $\times$ Area

$$\text{Area (A)} = n \frac{\pi}{4} (d)^2$$

$$= 12 \frac{\pi}{4} (7)^2 = 461.81$$

$$F = 38 \times 461.81 \times 12$$

$$P = 554.7 \text{ kN}$$

Hence the prestressing force per meter width of slab is compound as

$$P = \frac{1000 \times 554.17}{388}$$

$$P = 1689.54 \text{ kN}$$

Deflection due to prestressing force (straight cable)

$$a = -\frac{Pe l^2}{8EI}$$

$$I = \frac{bd^3}{12} = \frac{(1000)(500)^3}{12} = 10416.66 \times 10^6 \text{ mm}^4$$

$$a = -\frac{(1689.54)(195)(10.4 \times 10^3)^2}{8 \times 38 \times 10416.66 \times 10^6}$$

$$= -11.25 \text{ mm}$$

$$a = -11.25 \text{ mm} \uparrow (\text{Ans})$$

Deflection due to self weight and dead load

$$a = \frac{5(q+g)l^4}{384EI}$$

$$= \frac{5(23.5 \times 10^3)(10.4 \times 10^3)^4}{384 \times 38 \times 10416.66 \times 10^6}$$

$$a = 9.043 \text{ mm} \text{ (due to loads +ve)}$$

$$\text{Resultant deflection} = 9.043 - 11.25$$

$$a = -2.20 \text{ mm} \uparrow$$

2) A PSC beam of rectangular beam 120mm wide & 300mm deep span over 6m the beam is prestressed by a straight cable. Carrying an effective force of 200kN at an eccentricity of 50mm. $E_c = 38 \text{ kN/mm}^2$ compute the deflection at centre of span for following cases.

i) Deflection under Prestress + Self weight.

ii) Find the magnitude of UDL live load which will nullify the deflection due to Prestress & self weight.

Given data,

$$b = 120 \text{ mm}$$

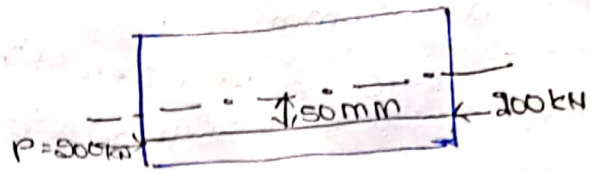
$$d = 300 \text{ mm}$$

$$\text{length } (l) = 6 \text{ m}$$

$$\text{force } (P) = 200 \text{ kN}$$

$$e = 50 \text{ mm}$$

$$F_c = 38 \text{ kN/mm}^2$$



$$\text{stress} = \frac{\text{force}}{\text{Area}} = \frac{200 \times 10^3}{120 \times 300}$$

$$\text{Stress } (F) = 5 \text{ N/mm}^2$$

Deflection due to Prestressing force [st. cabo]

$$a = \frac{-P e l^2}{8 E I}$$

$$I = \frac{b d^3}{12} = \frac{(120)(300)^3}{12} = 270 \times 10^6 \text{ mm}^4$$

$$a = \frac{-(200)(50)(6 \times 10^3)^2}{8 \times 38 \times 270 \times 10^6}$$

$$a = -4.38 \text{ mm}$$

Deflection due to selfweight.

$$a = \frac{5 W l^4}{384 E I}$$

$$W = 0.12 \times 0.3 \times 24 = 0.86 \text{ kN/m}$$

$$a = \frac{5(0.86 \times 10^3)(6 \times 10^3)^4}{384(38)(270 \times 10^6)}$$

$$a = 1.42 \text{ mm}$$

Resultant deflection = $1.42 - 4.38$

$$a = -2.96 \text{ mm}$$

13/10/20

ii) Deflection due to live load.

$$a = \frac{5 W l^4}{384 E I}$$

$$W = 9$$

$$a = \frac{5 \times 9 \times 6^4}{384 \times 38 \times 270 \times 10^6}$$

'a' can be taken as +ve; and resultant deflection

$$a = 296 \text{ mm}$$

$$2.96 = \frac{5(6)(6 \times 10^3)^4}{384(38)(270 \times 10^6)}$$

$$1 \text{ m} = 1000 \text{ mm}$$

$$\frac{1}{1000} \text{ m} = 1 \text{ mm}$$

$$\text{KN/m} \leftarrow \text{N/m}$$

$$q = 1.8 \text{ N/mm}$$

$$q = 1.8 \text{ kN/m}$$

3) A rectangular beam of c/s section 150mm & 300mm deep is simply supported over a span of 8m & is prestressed by means of a symmetric parabolic cable at a distance of 75mm from the bottom of the beam, at mid span & 125mm from the top of the beam at the support sections. If the force in the cable is 350kN and $E_c = 38 \text{ kN/mm}^2$ Calculate: (a) the deflection at mid span when the beam is supporting its own weight.

(b) the concentrated load which must be applied at midspan to restore it to the level of support ($e_2 = 150 - 125 = 25$)

sol: Given data,

$$E_c = 38 \text{ kN/mm}^2$$

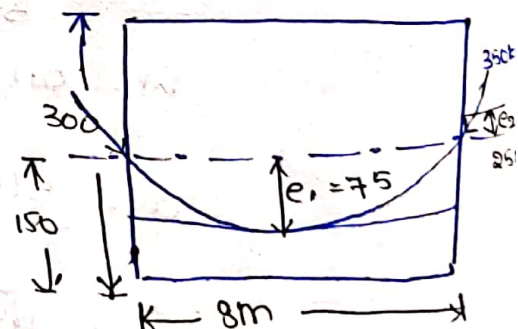
$$P = 350 \text{ kN}$$

$$\text{c/s} = 150 \times 300 \text{ mm}$$

$$L = 8 \text{ m}$$

$$e_1 = 75 \text{ mm}$$

$$e_2 = 25 \text{ mm}$$



a) deflection due to prestressing force [parabolic eccentricity term]

$$\Delta = \frac{P e l^2}{48 E I} [-5e_1 + e_2]$$

$$I = \frac{bd^3}{12} = \frac{150 \times 300^3}{12} \Rightarrow I = 337.5 \times 10^6 \text{ mm}^4$$

$$a = + \frac{350(8 \times 10^3)^2}{48(38)(337.5 \times 10^6)} [-5(75+) + 95]$$

$$= 0.036(-350)$$

$$\boxed{a = -12.6 \text{ mm}} \uparrow$$

deflection due to self weight

$$a = \frac{5ql^4}{384EI}$$

$$q = 24 \times 0.15 \times 0.3 = 1.08 \text{ kN/m}$$

$$q = 1.08 \text{ N/mm}$$

$$a = \frac{5(1.08)(8 \times 10^3)^4}{384(38)(337.5 \times 10^6)}$$

$$\boxed{a = 4.49 \text{ mm}} \downarrow (\text{downward})$$

prestress at self weight = $4.49 - 12.6$

$$\boxed{a = 8.11 \text{ mm}}$$

(b) concentrated load

$$a = \frac{wl^3}{48EI} \quad w = q$$

$$q = \frac{48EIa}{l^3}$$

$$= \frac{48 \times 38 \times 337.5 \times 10^6 \times 8.11}{(8 \times 10^3)^3}$$

$$\boxed{q = 9.74 \text{ kN}}$$

12/30/20

LONG TERM DEFLECTIONS:

- 4) A post tensioned roof girder spanning over 30m has an unsymmetrical I-section with a 2nd moment area of $72,440 \times 10^6 \text{ mm}^4 (I)$ & an overall depth of 1300mm. The effective eccentricity of the group of parabolic cables at a centre of span is 580mm towards the

Support and 170mm towards the top of the beam at supports. The cables carries an initial Prestressing force of 3200 kN. The Self weight of the girder 10.8 kN/m and the live load of the girder is 9 kN/m. The modulus of elasticity of concrete is 34 kN/mm². If the creep coefficient is 1.6 and the total loss of prestress is 15%. Estimate the deflections at the following stages & compare them with the permissible values according to IS code: 1343 limits.

- Instantaneous deflection due to prestress + self weight
- Resultant maximum long term deflection allowing for loss of prestress and creep of concrete.

Sol: Given data,

$$\text{span}(l) = 30\text{m}$$

$$\text{2nd moment area}(I) = 781490 \times 10^6 \text{ mm}^4$$

$$\text{overall depth}(d) = 1300\text{mm}$$

$$e_1 = 580\text{mm}$$

$$e_2 = 170\text{mm}$$

$$\text{Force}(P) = 3200\text{ kN}$$

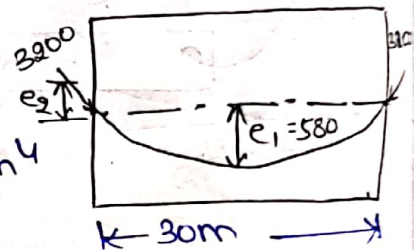
$$\text{Self weight of girder}(g) = 10.8\text{ kN/m}$$

$$\text{live load}(q) = 9\text{ kN/m}$$

$$E_c = 34\text{ kN/mm}^2$$

$$\text{creep coefficient}(\phi) = 1.6$$

$$\text{Total loss of prestress is } 15\%$$



a) Prestress + self weight.

$$a = \frac{Pl^2}{48EI} [-5e_1 + e_2]$$

$$= \frac{3200(30 \times 10^3)^2}{48(34)(781490 \times 10^6)} [-5(580) + 170]$$

$$a = -66.45 \text{ mm} \uparrow (\text{upward})$$

deflection due to self weight.

$$a = \frac{5gl^4}{384EI}$$

$$= \frac{5(9.81)(30 \times 10^3)^4}{384(38)(7249 \times 10^6)}$$

$$a = 46.21 \text{ mm} \downarrow$$

prestress at self weight. = $46.21 - 66.45$

$$a = -20.23 \text{ mm} \uparrow$$

deflection due to live load.

$$a = \frac{5gl^4}{384EI}$$

$$20.23 = \frac{5(9.81)(30 \times 10^3)^4}{384(38)(7249 \times 10^6)} \times$$

$$a = \frac{5gl^4}{384EI}$$

$$a = \frac{5(9.81)(30 \times 10^3)^4}{384(38)(7249 \times 10^6)}$$

long term deflection due to creep.

$$a = 38.51 \text{ mm}$$

$$a_f = [1 + \phi] a_i$$

$$a_f = [1 + 1.6] [46.21]$$

$$a_f = 120.14$$

15% loss of prestress

$$= 0.85 \times P$$

$$= 0.85 \times -66.45$$

$$= -56.48$$

$$100 - 15 = 85\%$$

$$\frac{85}{100} = 0.85$$

Total resultant long term deflection =

$$38.51 + 120.14 - 56.48$$

$$= 102.16 \text{ mm}$$

From IS 1343: Pg. 32 cl: 19.3.1 (b) 180.3.1 above

value should not be exceed $\frac{\text{span}}{250} = \frac{30 \times 10^3}{250}$

$$= 120 \text{ mm}$$

$$102.16 < 120 \text{ mm.}$$

Hence ok.