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CMR Institute of Technology, Bangalore
DEPARTMENT OF CIVIL ENGINEERING
II - INTERNAL ASSESSMENT

Semester: 8-CBCS 2017
Subject: DESIGN OF PRE STRESSED CONCRETE ELEMENTS (17CV82)
Faculty: Ms Sreelakshmi G

Date: 19 Jun 2021
Time: 09:00 AM - 10:30 AM
Max Marks: 50

Instructions to Students :

Answer all questions

Answer All Questions

Q.No		Marks	CO	PO	BT/CL
1	What are the factors influencing the deflection of prestressed concrete members?	6	CO1	PO1	L1
2	A prestressed concrete beam of rectangular section 120 mm wide and 300 mm wide, span over 6m. The beam is prestressed by a straight cable carrying an effective force of 200kN at an eccentricity of 50 mm . The modulus of elasticity of concrete is 34 kN/mm^2.compute the deflection at centre span for the following cases 1. Deflection under prestress + self-weight 2. Find the magnitude of uniformly distributed live load which can nullify the deflection due to prestress + self weight	14	CO3,CO4	PO3,PO4	L3
3	What are the different types of flexural failure observed in PSC beams?	4	CO3,CO4	PO11	L1
4	A prestressed pre-tensioned concrete beam of rectangular sections 150 mm wide and 350 mm deep is prestressed by cables of area = 461 mm^2. The effective cover is 50 mm , fck = 40 N/mm^2 , fp = 1600 N/mm^2. Calculate the ultimate strength of the section using IS 1343 1980 code provisions	10	CO2,CO4	PO2,PO4	L3
5	Design a post-tensioned concrete girder to suit the following data Span of the beam = 22 m Live load = 10kN/m fck = 50 N/mm^2 fct = 40 N/mm^2 loss ratio = 0.80 fp = 1500 N/mm^2 Cable contains 12 wires of 7 mm diameter Design the girder as aType 1member using IS1343 1980 code provisions	16	CO2,CO3	PO2,PO3	L3

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⇒ DESIGN OF PRESTRESSED STRUCTURES.

1A) Factors influencing the deflection of prestressed concrete members:-

In general practice, as per various codes of practice, the structural concrete members should be designed to have adequate stiffness to limit deflections that may adversely affect the strength & serviceability of the structures at working load. Thus, the deflection of flexural members is calculated to satisfy a limit state of serviceability. Various factors, that influence this are:

(i.) Imposed load & self weight.
(ii.) Magnitude of prestressing force.
(iii.) Cable profile.
(iv.) Second moment of area of cross-section.
(v.) Modulus of elasticity of concrete.
(vi.) Shrinkage, creep, relaxation of steel stress.
(vii.) Span of the member.
(viii.) Fixity conditions.

These are the various factors, that influence the deflection of prestressed concrete members.

2A) Given:

$b = 120mm = 0.12m$

$D = 300mm = 0.3m$ (∵ considering as depth)

$L = 6000mm = 6m$

$P = 200kN = 2\times10^3 N$

eccentricity, $e = 50mm$

$E_c = 34\ kN/mm^2 = 34\times10^3 N/mm^2$.

WKT,

Area, $A = b\times D =$ ~~1200×~~ 120×300

$= 36000\ mm^2 = 0.036\ m^2$.

Moment of Inertia, $I = \frac{bD^3}{12} = \frac{120\times(300)^3}{12} = 270\times10^6\ mm^4$

self weight of beam $= 0.12 \times 0.3 \times 1 \times 24$ [$\because$ Density of concrete $= 24\,kN/m^3$
1 = thickness]

$w_d = 0.864\,kN/m$

$w_d = 0.864\,N/mm.$

Upward deflection due to prestress, $\delta_p = -\dfrac{P e L^2}{8 E_c I}$

$= -\dfrac{(200 \times 10^3) \times 50 \times 6000^2}{8 \times 34 \times 10^3 \times 270 \times 10^6}$

$= -4.901\,mm.$

Downward deflection due to self weight, $\delta_d = \dfrac{5 w_d L^4}{384 E_c I}$

$= \dfrac{5 \times 0.864 \times (6000)^4}{384 \times 34 \times 10^3 \times 270 \times 10^6}$

$= 1.588\,mm.$

$\Rightarrow$ Deflection under prestress & self weight $= \delta_p + \delta_d$

$= -4.901 + 1.588$

$= -3.313\,mm.$ — (1)

Magnitude of UDL live load, that can nullify the deflection caused due to prestress & self weight = Q (let)

WKT,

Deflection due to live load $=$ ~~5QL4~~ $\dfrac{5 w_L L^4}{384 E_c I}$

Here, $w_L = Q.$

$\Rightarrow$ Deflection $= \dfrac{5 Q L^4}{384 E_c I}$ — (2)

Equating ① & ②

$$-3.313 = \frac{5 \times q \times 6000^4}{384 \times 34 \times 10^3 \times 270 \times 10^6}$$

$\Rightarrow q = 1.80$ kN/m.

∴ The uniformly distributed live load, that can nullify deflection of ~~3.313~~ '-3.313mm' is $q = 1.80$ kN/m.

3A.) # Types of flexural failure observed in PSC beams:

The various types of flexural failures that are observed in prestressed concrete members:-

(i) Fracture of steel in tension: This causes sudden failure of prestressed members without any warning.

(ii.) Failure of under reinforced structure: If cross-section is provided with steel with amount greater than the minimum prescribed in case '1' this failure occurs.

(iii.) Failure of over reinforced section: Occurs, when the compressive strength of concrete & the tensile strength of steel exceeds a range of effective reinforcement index values.

(iv.) Other modes of failure:

~~Flex~~ - Flexure shear failure:- Occurs when members are not designed adequately for shear.

- Shear-compression failure:- Occurs in beams/members, that contain adequate web reinforcement.

- Web shear cracks that may develop due to excessive principle stresses & if thin webs are used due to web crushing.

- Failure of bond between steel & surrounding concrete in prestressed members due to inadequate transmission lengths at the ends. etc..

44) Given:-

Prestressed pretensioned concrete beam, rectangular section.

$b = 150mm$ effective cover $= 50mm$.

~~d = 350mm~~ $D = 350mm$ $d = 350 - 50 = 300mm$

prestressed by cables of area, $A_{ps} = 461 \, mm^2$.

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

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

Ultimate flexural strength of section = ?

Step 1: effective reinforcement ratio:

$$\frac{A_{ps} f_{pu}}{b d f_{ck}}$$ [∵ IS 1343 - 2012, Pg 51]

$$= \frac{461 \times 1600}{150 \times 300 \times 40} = 0.40$$

Step 2:

$$\left(\frac{f_{pb}}{0.87 f_{pu}}\right) = 0.9 \qquad \left(\frac{x_u}{d}\right) = 0.783$$ [∵ Table, 11, pg 51, IS 1343-2012]

$$\Rightarrow f_{pb} = 0.9 \times 0.87 \times 1600$$
$$= 1252.8 \, N/mm^2.$$

$$\Rightarrow x_u = 0.783 \times 300$$
$$= 234.9 \, mm.$$

Step 3: Ultimate flexural strength of section:

$$M_u = f_{pb} A_{ps} (d - 0.42 x_u)$$

$$= 1253 \times 461 \times (300 - (0.42 \times 234.9))$$

$$= 116.301 \times 10^6 \, N/mm. = 116.301 \, kN\text{-}m.$$

5A) Given:

Span of beam, $L = 22m$

live load, $= 10 kN/m$

$f_{ck} = 50 N/mm^2$

$f_{ct} = 40 N/mm^2$

loss ratio $= 0.80$

$f_p = 1500 N/mm^2$

Cable contains 12 wires of $7mm\ \phi$

To design type 1 girder.

Area of one cables $= 12 \times \frac{\pi}{4} \times 7^2$

$A_p = 461.8 mm^2$

Area of concrete section, $A = b \times d$

step 1:

Let, $b = 250mm = 0.25m$ ($\because$ assumming)

depth of section $= d$

self weight of beam/gravity load $= 24 \times 0.25 \times d$

$= 6d$ [$\because$ density of concrete $= 24 kN/m^3$]

Moment due to self wt, $M_g = \frac{6d \times 22^2}{8}$ [$\because M_g = \frac{wL^2}{8}$]

$= 363 d$ kN-m.

Moment due to live load, $M_q = \frac{10 \times 22^2}{8}$ [$\because M_q = \frac{wL^2}{8}$]

$= 605$ kN-m.

Step 2:

w.k.t,

$$Z_b = \frac{M_q + (1-\eta) M_g}{(\eta f_{ct} - f_{tw})} \quad -①$$

& for rectangular section, $Z_b = \frac{bd^2}{6}$ $-②$

$$① \Rightarrow Z_b = \frac{605 + (1-0.80)\, 363\, d}{((0.80 \times 40000) - 0))}$$

[$\because \eta = 0.80$ (given)
$f_{tw} = 0$ $\because$ its Type-I member.
$f_{ct} = 40$ N/mm² (given)]

$$= \frac{605 + 363d - 290.4d}{32000}$$

$$= \frac{605 + 72.6d}{32000}.$$

$$② \Rightarrow Z_b = \frac{5d^2}{6} = \frac{0.25d^2}{6}$$

Equating ① & ②

$$\frac{605 + 72.6d}{32000} = \frac{0.25d^2}{6}.$$

$$3630 + 435.6d = 8000d^2.$$

$$\Rightarrow d = 0.701\,m \cong 0.705\,m$$

$\therefore$ Dimensions of section = 250 x 705 mm.

$$\Rightarrow M_g = 363\,d = 255.91\ kN\text{-}m.$$

Step 3: Z_t, Z_b.

$$Z_t = Z_b = \frac{bd^2}{6} \quad (\because \text{rectangular section})$$

$$= \frac{0.25 \times 0.705^2}{6} = 0.02\ m^3.$$

Step 4: stress at top fibre; f_t

$$f_t = f_{tt} - \frac{M_g}{Z_t} = 0 - \frac{255.91 \times 0.705}{0.02}$$

$$= -9020.82\ kN/m^2 = \cancel{-0.902\ N/mm^2}$$

$$= -9.02\ N/mm^2.$$

Step 5: stress at bottom fibre; f_b

$$f_b = \frac{1}{\eta}\left(f_{tw} + \frac{M_g}{Z_b} + \frac{M_q}{Z_b}\right)$$

$$= \frac{1}{0.60}\left(0 + \frac{255.91}{0.02} + \frac{605}{0.02}\right) = 53806.87\ kN/m^2$$

$$= 53.806\ N/mm^2.$$

Step 6: Prestressing force.

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

$$= \frac{(0.25 \times 0.705)\left((0.02 \times -9020.8) + (0.02 \times 53806.87)\right)}{(0.02 + 0.02)} = 3941.16\ kN$$

Step 7: Eccentricity

$$e = \frac{(f_b - f_t) Z_t Z_b}{A(Z_t f_t + Z_b f_b)} = \frac{(53806.87 - (-9020.42))(0.02)^2}{0.176 \times (0.02 \times (-9020.8 + 53806.8))}$$

$$= 0.156 \cong 0.16 m = 160 mm.$$

Step 8: No. of cables required.

Prestressing force, $P = 3941.16$ kN $= 3941160$ N.

characteristic strength of tendon, $f_p = 1500$ N/mm²

$$\therefore f_p = \frac{\text{Prestressing force}}{\text{Total area of cables}} = \frac{3941160}{\text{Total area of cables}} = 1500$$

$$\Rightarrow \text{Total area of cables} = 2627.44 \text{ mm}^2.$$

Area of one cable $= 12 \times \frac{\pi}{4} \times 7^2 = 461.81$ mm²

$$\text{No. of cables} = \frac{\text{Total area of cables}}{\text{Area of one cable}}$$

$$= \frac{2627.44}{461.81} = 5.68 \cong 6 \text{ cables.}$$

$\therefore$ 6 cables are needed for the concrete girder of A = 250×705 mm & span 22m.