III Yr BE (Civil) Home Assignment Sheet
DESIGN OF CONCRETE STRUCTURES-II ECEC-3160

Q1 (a) Why does the Code disallow moment distribution when bending moments in continuous beams are based on the Code moment coefficient.

(b) What do you mean by moment redistribution and what are its implications in design?

(c) A reinforced concrete beam is continuous over two equal spans of length 6m each. It is fixed at end supports and continuous over central support. It carries a uniformly distributed load of intensity 15 kN/m inclusive of its own weight over its entire length of 12m. Design for flexure and shear a suitable section for the beam. Adopt M-20 grade concrete and Fe-415 HYSD bars.

Q2 (a) What are the basic assumptions underlying the approximate method of frame analysis for multi-storeyed buildings.

(b) Is it desirable to have (i) high strength steel (ii) high strength concrete in earthquake-resistant design of reinforced concrete structures? Justify your answers

(c) Why are inclined stirrups and bent-up bars unsuitable as shear reinforcement in earthquake resistant design?

Q3 (a) What is the purpose of retaining wall? What are different types of concrete retaining wall? What is the purpose of shear key? Describe its action.

(b) Suggest suitable proportions for a counterfort retaining wall to support difference in ground elevation of 9m. The foundation depth may be taken as 1.5m below ground level, with a safe bearing capacity of 160 kN/m². Assume a level backfill with a unit weight of 16 kN/m³ and an angle of shearing resistance of 30°. Assume a coefficient of friction $\mu = 0.5$ between soil and concrete. Check the stability of the wall.

Q4 (a) Describe the different types of water tanks. Why Intz type tanks are mostly preferred in overhead tanks.

(b) A reinforced concrete Intz type water tank is required to

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Store 250,000 litres of water. Height of staging is 12m above ground level. The tank is supported on six columns. Safe bearing capacity of the soil = 150 KN/m³. Basic wind pressure = 1.5 KN/m². Adopting M-20 grade concrete and Fe-415 grade of steel, design the Intz type tank and sketch the details of reinforcement in the various structural components of the tank.

Q5(a) Describe the different type of losses in pre-stressed concrete.

(b) A pre-tensioned concrete beam 200 mm x 400 mm cross-section is stressed by 10-7 mm high tensile steel wires below the centre line of the section. Determine moment of resistance of the section. Assume f_{ck} = 35 MPa and σ_p = 1500 MPa.

(c) A post-tensioned bonded beam of rectangular section 150 mm wide and 300 mm deep has prestressing cables at an eccentricity of 60 mm. Determine the flexural strength of the section if it is prestressed with 15-4 mm high tensile steel wires. Assume f_{cu} = 45 MPa and σ_p = 1400 MPa.

(d) The roof of a room is 12 m wide and 25 m long. It consists of prestressed T-beams placed at 5 m centre to centre. The slab is 12 cm thick and the super-imposed load is 4 KN/m² including that of the slab. The live load is 2.5 KN/m². Design an intermediate T-beam in M40 concrete. Assume suitable data.

Note: (i) The Home assignment is to be submitted by 10.5.2020.
(ii) The sessional part is of 70 marks due to Pandemic
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Design Of Concrete Structures - II

Q. 1

(a) why does the code disallow moment distribution when bending moment in continuous beams are based on the code moment coefficient.

Ans. Moment distribution is structural analysis method for statically indeterminate beams and frames while moment redistribution refers to the behaviour of statically indeterminate structure that are not completely elastic, but have some reserve plastic capacity. Code disallow the moment distribution became that are close to the exact value of the max. load effect obtainable from rigours analysis, if some how it given minor errors are assumed to be accomodated tough the inherent capacity for moment redistribution in the structure.

(b). what do you mean by moment redistribution and what are its implication in design.

Ans. Moment redistribution means the transfer of additional moments to the stressed section. as the highly stressed section with the peak moments yields on reading their ultimate moment capacity to apply moment redistribution, the highly stressed sec. are design for lower moment and are less the values obtained from a elastic analysis.

Implication in design:

(A). Equilibrium b/w the inertial forces and the external load is maintained.

(B). The ultimate moment of resistance provided at any section of a member is not less than 70% of the moment at that section obtained from an elastic max. moment at any section. moment diagram covering all appropriate combination of loads.

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(c). The elastic moment at any section in a member due to a particular combinations of loads shall not be reduced by more than 30% of the numerical largest moment given any where by the elastic maximum moments diagram for the particular member covering all appropriate combination of loads

(D). At section where the moment capacity after redistribution in less than that from the elastic maximum moment diagram, the following relationship shall be satisfied.

$$\frac{x_u}{d} + \frac{\delta M}{100} \leq 0.6$$

where,

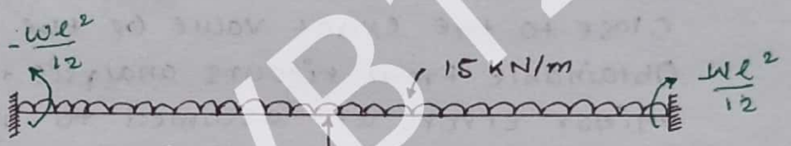
x_u = depth of N.A

d = Effective depth

δM = % reduction in moment

1.(c).

Sol^n:



Given

$W_d = 15\,kN/m$

Effective depth, $d = \frac{Span}{12}$

$= \frac{6000}{12} = 500\,mm$

Let as $D = 600\,mm$

$d = 550\,mm$

$L_{eff} = 6000 + 550 = 6.55\,m$

Let, Width of beam $= 0.3\,m$

By Codal Method,

$-ve$ BM at interior support $= -\left[\frac{1}{10} W_d L_{eff}^2\right]$

$= -\frac{1}{10} \times 15 \times (6.55)^2$

$= -64.35\,kNm$

$+ve$ BM in the Span $= +\frac{1}{12} W_d L_{eff}^2$

$= \frac{1}{12} \times 15 \times (6.55)^2$

$= 53.62\,kNm$

Shear at support $= 0.6\,[W_d \cdot L_{eff}]$

$= 0.6 \times 15 \times 6.55$

$= 58.95\,kN$

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Factored -ve BM = 1.5 (-64.35)
$= 96.52 kN/m$

Fact. +ve BM = 1.5×53.62
$= 80.43 kNm$

$M_u = 0.138 f_{ck} b d^2$

$10^6 \times 96.52 = 0.138 \times 20 \times 300 \times d^2$

$d = 342.42 mm$ OK

Section is under reinforced.

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} \times f_y}{b d f_{ck}}\right]$

$96.52 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{\cancel{550} \times 415 A_{st}}{300 \times 550 \times 20}\right]$

$A_{st} = 519.82 mm^2$

Provide 2-16mm Ø bar and 2-12mm Ø bars.

$A_{st pro.} = \frac{\pi}{4}\left[16^2 \times 2 + 12^2 \times 2\right]$

$= 628.31 mm^2$

Shear strain

$\tau_v = \frac{V_u}{bd} = \frac{56.95 \times 10^3}{300 \times 550}$

$= 0.35 N/m$

$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 628.31 \times}{300 \times 550}$

$= 0.38\%$

From Table

$\tau_c = 0.36 + \frac{0.48 - 0.36}{0.5 - 0.25}(0.38 - 0.25)$

$\tau_c = 0.4224 N/mm^2$

2 bars bent up at quarter, Shear Force taken by bars

$V_{vs} = 0.87 f_y A_s \sin\alpha$

$= 0.87 \times 415 \times \frac{\pi}{4} \times (12)^2 \times \sin 45°$

$= 55.75 kN$

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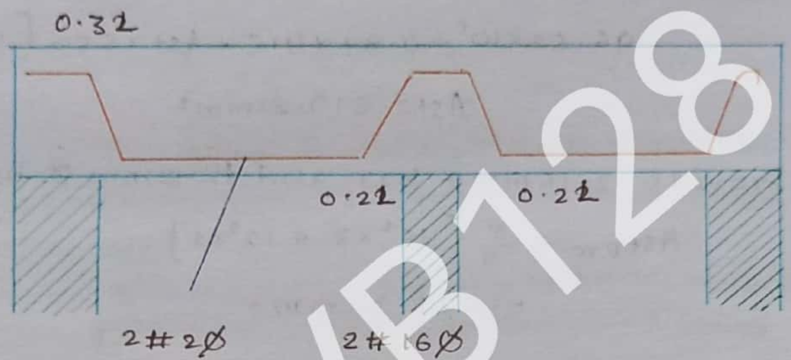
Shear taken by Concrete

$$\tau_{bd} = 0.4224 \times 300 \times 500$$
$$= 69.69\,KN$$

$$\text{Total} = 55.75 + 69.69$$
$$= 125.44\,KN > 58.95\,KN$$

$\therefore$ No shear reinforcement is required but as per codal provisions, minimum shear reinforcement is to be provided.



Provide 2 Legged 8mm Ø bars @ 300mm c/c

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Q2.

(a).

Ans. Approximate analysis of building frame can carried out either by Portal Method or by Cantilever Method. The Portal method is supposed to be satisfactory for most buildings upto about 25 storey, where as the Cantilever method is good enough for about 35 storeys.

PORTAL METHOD:

In this method the following Assumptions are made

★ There is a point of Inflection at the centre of each girder.

★ There is a point of inflection at the centre of each column.

★ The total horizontal shear on each storey is divided between the column of that story so that each interiors column carrier twice as much shear as each exterior column.

CONTINOUS METHOD:

In this method the following assumptions are made

★ There is a point of inflection at the centre of each girder.

★ The intensity of axial stress on each column of a storey is proportional to the horizontal distance of that column from the centre of gravity of all the columns of the storey under consideration.

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2(b).

Ans. ① Lower grade steel has clearly defined and longer yield plateau and hence the plastic hinges formed will have larger rotation capacities, leading to greater energy dissipation. The actual yield strength of the steel used should not be markedly higher than the yield strength, for in excess of that specified, may lead to excusive shear and bond stresses as the plastic moment is developed. The lower the grade, the higher is the ratio of ultimate tensile strength (fu) to the yield strength (fy). A high ratio of fu/fy is desirable as it results in an increased length of plastic hinge (along the member

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axis) and thereby an increased plastic rotation capacity. For these reasons Mild Steel (Fe250) is best suited for use as flexure reinforcement in the vertical earthquake resistant design. However its use will necusitate larger section of flexure members.

Hence the code permits the use of the higher grade Fe415 but prohibits the use of grade higher than Fe415.

(ii). With regard to the grade of concrete, the code limits the minimum grade of concrete to M20 (For all buildings which are more than 3 storey in height)

It may be noted that very high strength concrete is also undesirable because higher compressive strength is associated with lower ultimate comp strain (ϵ_c) which adversely affect ductility. Likewise low density concrete is undesirable because of its relatively poor performance under reversed cyclic loading.

2(C). Why are inclined stirrups and bent up bars are unsuitable as shear reinforcement in Earthquake resistance design?

Ans In Earthquake resistance structure, the design shear force will be the larger of

(i). Shear force as obtained from analysis for given load combination and

(ii) Actual shear force likely to develope in a member after flexural failure has taken place.

According to the code (a. 6. 3. 3 IS13920) the web reinforcement in the form of vertical stirrups shall be provided so as to develop the vertical shear due to formation of the plastic hinges at both ends of the beam plus the factored gravity load on the span.

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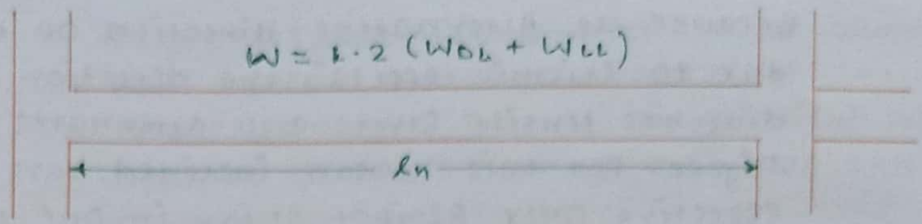

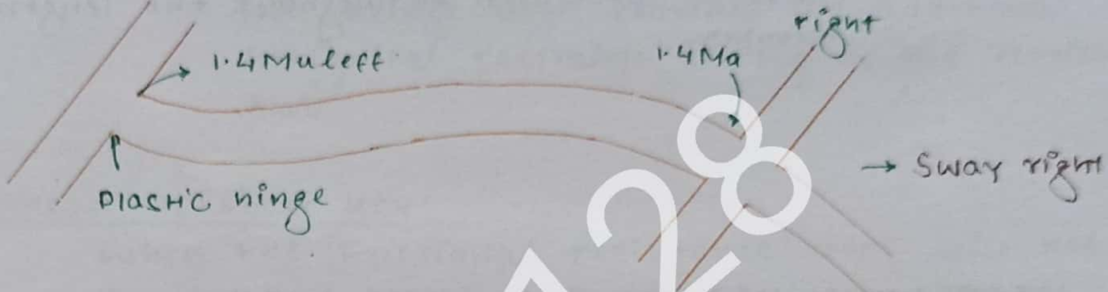

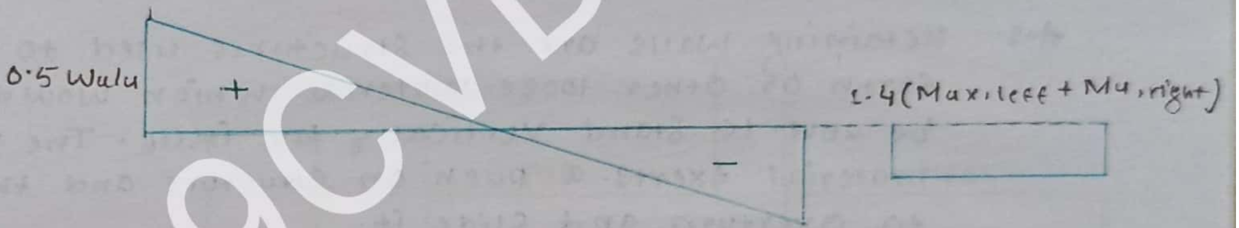

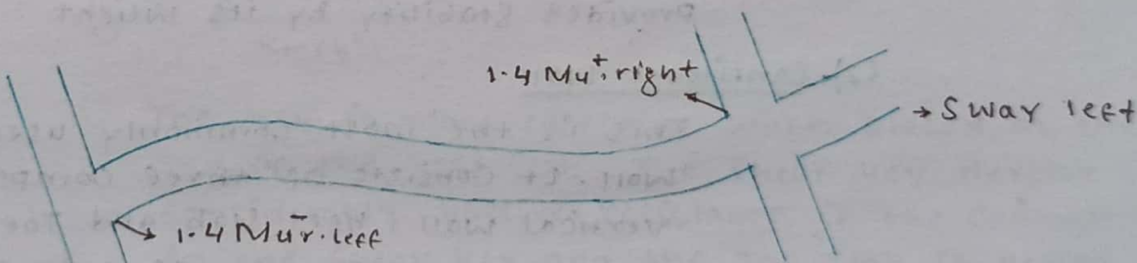

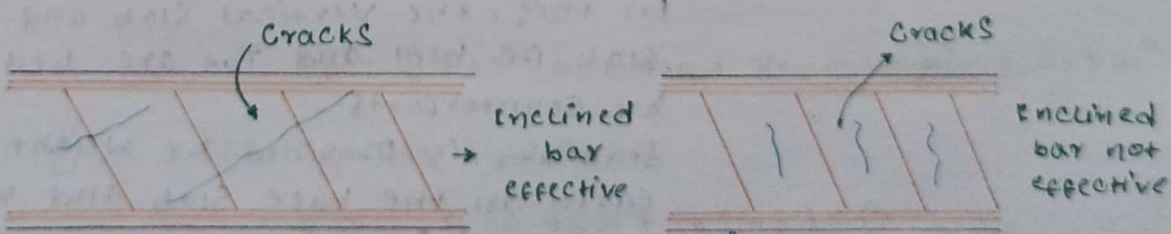

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Because of Alternating direction of the Shear Force due to Seismic effets, the direction of the associates diagonal tensile Stress also alternates as shown in Figure. For this reason inclined bars (which are effective only against shear in one direction) are not allowed.

Web reinforcement for seismic design must be in the Form of Closed Stirrups called hoops placed perpendicular to the longitudinal reinforcement and must be provided throughout the length of the member.

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3(a). What is the purpose of retaining wall? What are different types of conc. retaining wall? What is the purpose of Shear key? Describes its action.

Ans. Retaining walls are the structures used to retain earth or other loose material which would not be able to stand vertically by itself. The retained material exerts a push on structure and this tends to overturn and slide it.

Type of Conc. Retaining wall:

(1). Gravity wall:

A gravity wall is of plain concrete and provides stability by its weight

(2). Cantilever wall

This is the most commonly used retaining wall. It consists of three components: vertical wall, heel slab and Toe slab. Each of the three components acts as a cantilever beam.

(3). Counterfort wall:

In this, the vertical slab and the hori. slab i.e heel and Toe are tied together by counterforts.

Stability is provided by weight of the earth on the base slab and wt. of the retaining wall.

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④. Buttress wall

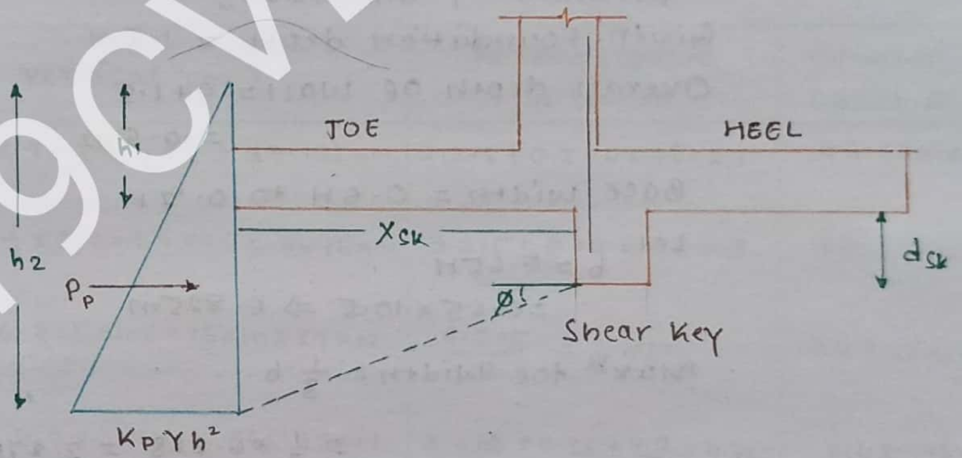
A buttress wall is similar to a counterfort wall except that the transverse walls are located on the side of the vertical wall opposite to the retained material and act as compression struts.

⑤. Bridge Abutment

A wall type bridge abutment acts similar to a cantilever retaining wall except that the bridge deck provides an additional horizontal restraint at top of the vertical slab.

<u>Purpose of Shear key:</u>

When the frictional resistance alone will not provide the required factor of safety against sliding, in such cases, it is advisable to provide a shear key.



The key is more effective when placed at the end of the heel slab. The shear key develop considerably passive resistance if the concrete in the shear key and the toe slab is placed against the undisturbed soil, without the use of vertical forms conservative estimate of passive pressure.

$$P_p = \frac{1}{2} K_p \gamma (h_1 + d_{sk} + X_{sk} \tan\phi)^2 - \frac{1}{2} K_p \gamma h_1^2$$

FOS against sliding:

$$FOS = \frac{\mu (0.9 \Sigma w + P_a \sin\delta) + P_p}{P\cos\delta} \geq 1.4$$

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3.
(b).

Solⁿ Given data:

Height of wall above G.L. = 9m
Safe bearing capacity = 160 kN/m²
$\mu = 0.5$
$\phi = 30°$
$\gamma = 16 kN/m^2$
Foundation depth below G.L. = 1.5m

Step I:

Coeff. of Earth Pressure:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$= \frac{1-\sin 30°}{1+\sin 30°} = \frac{1}{3}$$

$$K_p = \frac{1}{K_a} = 3$$

Step II:

Priliminary dimensions

Given Foundation depth = 1.5m

Overall depth of wall = 9+1.5
= 10.5m

Base width = 0.6H to 0.9H

Let, $b = 0.65H$
$= 0.65 \times 10.5 \Rightarrow 6.825m$

Max^m toe width = $\frac{1}{3}b$

$= \frac{1}{3} \times 6.825 = 2.275m$

Thickness of base slab $= \frac{H}{15}$ to $\frac{H}{20}$

$= \frac{10500}{15}$ to $\frac{10500}{20}$

$= 700$ to $525mm$

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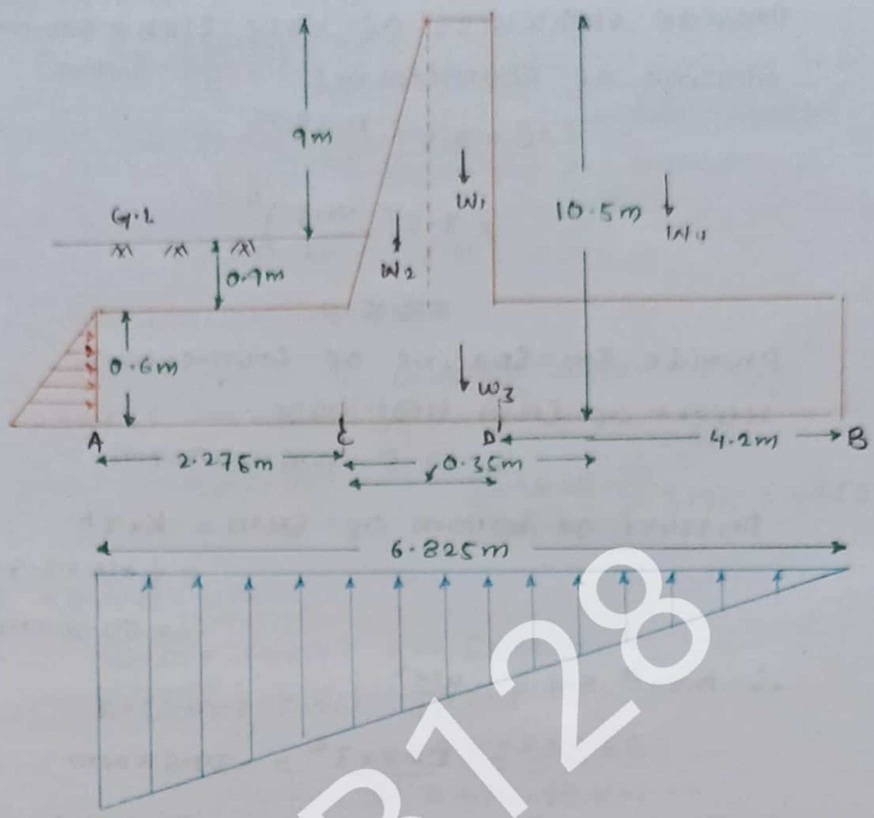

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Load	Vertical load	Perpendicular distance	Moment about A
W_1	$9.9 \times 0.15 \times 25 = 37.125$ KN	$2.275 + 0.2 + 0.075 = 2.25$	94.66 KNm
W_2	$\frac{1}{2} \times 0.2 \times 9.9 \times 25 \doteq 24.75$ KN	$2.275 + \frac{2}{3} \times 0.2 = 2.408$	59.59 KNm
W_3	$6.825 \times 0.6 \times 25 = 102.37$ KN	$\frac{6.825}{2} = 3.41$m	349.08 KNm
W_4	$4.2 \times 9.9 \times 6 = 665.28$ KN	$2.275 + 0.35 + \frac{4.2}{2} = 4.725$	3143.44 KNm
	$\Sigma W = 829.52$ KN		$\Sigma M_A = 3646.71$

Total downward load, $\Sigma W = 829.52$ KN ↓

$$\therefore \Sigma W \bar{x} = \Sigma M_A$$

$$x = \frac{3646.71 - 971.25}{829.52} = 3.22\text{m}$$

Eccentricity, $e = \frac{b}{2} - \bar{x}$

$$= \frac{6.825}{2} - 3.22$$

$$= 0.1925\text{m}$$

Provide thickness of base slab = 600mm

Spacing of counterforts

$$l = 3.5\left(\frac{H}{\gamma}\right)^{0.25}$$

$$= 3.5\left(\frac{10.5}{16}\right)^{0.25}$$

$$= 3.15m$$

Provide 3m spacing of counterforts

Height of stem near base

$$h = 10.5 - 0.6 \Rightarrow 9.9m$$

Pressure at bottom of stem = $K_a \gamma h$

$$= \frac{1}{3} \times 16 \times 9.9$$

$$= 52.8\ KN/m^2$$

$\therefore$ Max^m BM $= -\frac{wl^2}{12}$

$$= -\frac{52.8 \times 3^2}{12} = -39.6\ KNm$$

Factored BM = $1.5 \times 39.6 = -59.4$ KNm

$$M_u = 0.138 f_{ck} b d^2$$

$$59.4 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$d = 146.70mm$$

For shear criterion, Provide larger depth.

$$d = 300mm$$

$$D = 300 + 50 = 350mm$$

Provide width = 150mm

Heel width = $6.825 - 2.275 - 0.35$

Load	Horizontal Load	distance from A
P	$\frac{1}{2} \times 52.8 \times 10.5 = 277.5\ KN$	$\frac{1}{3} \times 10.5 = 3.5$

Overturning moment about A = 277.5×3.5

$$= 971.25\ KNm$$

Sliding Force = 277.5 KN ←

Max. Pressure at A (Toe):

$$P_{max} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{829.52}{6.825}\left(1 + \frac{6 \times 0.192}{6.825}\right)$$

$$= 142.1\ KN/m^2 < SBC\ (160)\ \text{OK}.$$

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Min. Pressure at B (heel)

$$P_{min} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$$

$$= \frac{829.52}{6.825}\left(1 - \frac{6 \times 0.192}{6.825}\right)$$

$$= 100.97 \ kN/m^2 > 0$$

∴ No tension at base

FOS against Overturning $= \frac{M_r}{M_o}$

$$\frac{3646.17}{971.25} = 3.75 > 1.155 \ \underline{OK}$$

FOS against sliding:

Sliding Force = 277.5 kN ←

Resisting Force = $\mu \Sigma W$

$= 0.5 \times 829.52$

$= 414.76$ kN →

Neglecting ~~+ve~~ Passive Pressure of soil at Toe

$$\therefore FOS = \frac{414.76}{277.5} = 1.49 < 1.55 \ \text{Not OK.}$$

∴ Therefore we need to provide shear key in design to anchor in the ground running along the length of retaining wall.

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4(a). Describe the different types of water tank? Why Intz tanks are mostly preferred in overhead tanks.

Ans: Different types of RCC water tanks are;

(A). Based on Location:

(i). Underground water tank.

(ii). Water tanks resting on ground.

(iii). Overhead water tanks.

(B). Based on shape:

(i). Rectangular water tank

(ii). Circular water tank

(iii). Spherical water tank

(iv). Intz water tank

(v). Circular water tank with conical bottom.

Why Intz tanks are mostly preferred;

In flat bottom, the thickness and reinforcement is found to be heavy. In the dommed bottom, through the thickness and reinforcement in the dome is normal and the rein. in the ring beam is excusive. In case of Larger diameter tanks, an economic alternate would be to reduce its diameter at bottom by conical dome, such a tank is called as Intz tank and is very commonly used.

Advantage:

The main advantage of such a tank is that the outward thrust from the top of the conical part is resisted by the ring beam B_3 while the difference b/w the inward thrust and the outward thrust from the bottom dome are resisted by ring beam B_2

The portion of the conical dome and bottom dome are so arranged that the outward thrust from bottom dome balances the inward thrust due to conical dome.

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4 (b).

Sol^n. Given data:

Water capacity = 250,000 l

Ht. of staging = 12 m from G.L.

Safe bearing cap. = 150 KN/m^2

Wind pressure = 1.5 KN/m^2

Live load = 1.5 KN/m^2 by code

σ_{st} = 150 N/mm^2

σ_{cb} = 1.7 N/mm^2

σ_{ct} = 1.2 N/mm^2

① Dimension:

Let dia. of tank, D = 12m

Let dia. of lower ring beam = 8 m

Rise of Top Dome h_1 = 2m

Rise of bottom dome h_2 = 15m

Ht. of conical dome h_0 = 2m

Ht. of cylindrical portion = h = ?

Spacing of bracing = 4m

Capacity of tank

$$250\,m^3 = \frac{\pi}{4} D^2 h + \frac{\pi}{12} \times h_0 (D^2 + D^2 - DD_2) - \frac{\pi}{3} h_2 (3R_2 - h_2)$$

where,

$$R_2 = \frac{(D_2/2)^2 + h_2^2}{2h_2} = \frac{(8/2)^2 + (1.5)^2}{2 \times 1.5} = 6.08$$

$$250 = \frac{\pi}{12} \times 12^2 \times h + \frac{\pi}{12} \times 2 (12^2 - 8^2 - 12 \times 8) - \frac{\pi}{3} \times 1.5^2 (3 \times 6.08 - 1.5)$$

h = 1.151m

Take, h = 2.5m

② Design of Top Dome:

(i). Meridrinal Force

(ii). Hoop Forces

(i). Meridinal Force (T_1)

Let thickness of Top dome = t = 100mm

Live load on dome = 1.5 KN/m^2

Self wt. of dome = 0.1×25

= 2.5 KN/m^2

Total load = 4 KN/m^2

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$$R_1 = \frac{(D/2)^2 + h_1^2}{2h_1} = \frac{6^2 \times 2^2}{2^2} = 10m$$

$$\sin\theta = \frac{D/2}{R} = \frac{6}{10} = 0.6$$

$$\theta = 36.86°$$

$$T_1 = \frac{WR}{1+\cos\theta}$$

$$T_1 = \frac{4\times10}{1+\cos 36.86°} = 22.22 \text{ KN/m}$$

$$\text{Meridimal stress} = \frac{22.22\times10^3}{1000\times100}$$

$$= 0.22 \text{ N/mm}^2 < 5 \text{ N/mm}^2 \text{ OK}$$

(ii). Hoop Force:

$$T_2 = WR_1\left[\cos\theta - \frac{1}{1+\cos\theta}\right]$$

$$= 4\times10\left[\cos 36.86 - \frac{1}{1+\cos 36.86}\right]$$

$$= 9.78 \text{ KN/m}$$

$$\text{Hoop Stress} = \frac{9.78\times10^3}{1000\times100} = 0.0978 \text{ N/mm}^2 < 5 \text{ N/mm}^2$$

∴ Provide Nominal reinforcement (0.24%)

$$A_{st} = \frac{0.24}{100}\times1000\times100 = 240 mm^2$$

Provide 8mm Ø @ 200 mm c/c both circumaxially and meridinally.

③ Design of Top Ring beam:

Hori. component of meridional Force (T_1)

$$W = T_1\cos\theta = 22.22\cos\theta = 17.77 \text{ KN/m}$$

$$\text{Total hoop tension in beam} = \frac{WD}{2}$$

$$= \frac{17.77\times12}{2} = 106.62 \text{ KN}$$

Ast For hoop tension

$$A_{st} = \frac{F}{\sigma_{st}} = \frac{106.62\times10^3}{150} = 710.8 mm^2$$

- Provide 7 # 12 Ø bar

Ast provided = 791.68 mm²

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Cl B.2.1.1 (IS 456-2000)

$$\text{Tensile Stress} = \frac{F \text{ or } T}{A_i + (m-1) A_{st}}$$

Let, $b = 300$

$$1.C \geq \frac{106.62 \times 10^3}{300 \times D + (13.33-1) 791.68}$$

$$D \geq 204.39\,mm$$

Provide 300mm by 300mm

Provide 8Ø - 2 Legged Stirrup.

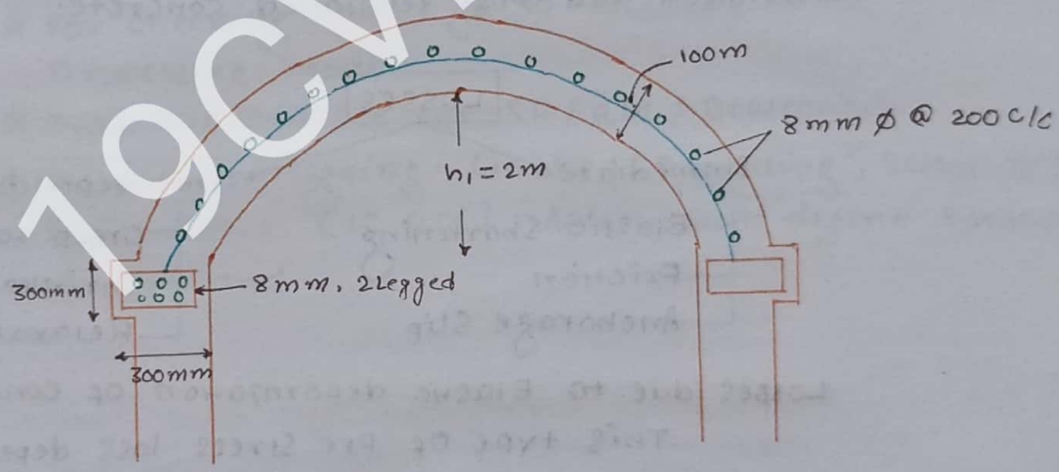
(a). $S_v = \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 \times 415 \times \frac{\pi}{4} \times 8^2 \times 2}{0.4 \times 300}$

$$S_v = 302.47\,mm$$

(b). $0.75 d = 0.75 \times 275 = 206\,mm$

(c). 300mm

Provide 8mm Ø 2-Leg vertical stirrups @ 200 mm c/c



④ Design of cylindrical wall:

$$T = \frac{1}{2} \gamma_w h D = \frac{1}{2} \times 100 \times 2.5 \times 12 = 150$$

$$A_{st} = \frac{150 \times 10^3}{150} = 1000\,mm^2$$

Provide 12mm Ø bars @ 110mm c/c throughout each face

$$A_{st}\ prov. = 1017.87\,mm^2$$

Thickness of wall,

$$\sigma_{ct} = \frac{T}{A_g + (m-1) A_{st}}$$

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$$\sigma_{ct} = \frac{T}{A_g + (m-1)A_{st}}$$

$$1.2 = \frac{150}{1000t' + (13.33-1)\,2035.74}$$

$$t' = 99.89\,mm$$

Take $t' = 150mm$ thick wall

$$\text{Distribution Steel}, = \frac{0.24 \times 150 \times 1000}{100}$$

$$= 360\,mm^2$$

Provide 10mm ∅ bar @ 250mm c/c on both face.

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5(a).

Ans

The different types of Pre-stress losses are executed in the two types of Pre stressed concrete viz. Pre tensioned and Post tensioned concrete.

LOSSES

- Immediate
  - Elastic Shortening
  - Friction
  - Anchorage Slip
- Time dependent
  - Creep Loss
  - Shrinkage loss
  - Relaxation loss

Losses due to Elastic deformation of Concrete:

This type of Pre Stress loss depends on the modular ratio and average stress in conc. at the level of centroid of steel tendons or wires.

Loss of Prestress due to Friction:

Friction Loss is of two types

(A). Loss of Stress due to the effect of curvature which is depends on the alignment of tendon.

(B). Loss of Stress due to wobble effect or wave effect.

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Losses due to Anchorage Slip:

It is Observed mostly in Post tensioning system where in the cable is tensioned & the jack is released to transfer the pre-stress to concrete, the Friction wedges that are used to grip the wires, get slipped for a small distance befores the wire are anchored Firmly b/w the edges.

Losses due to creep of concrete:

The sustained pre stress in a pre-stressed concrete member results in a creep of concrete.

Losses due to stress relaxation of steel:

In order to account for loss due to relaxation of steel, most of the codes all over the worlds describe the loss of stress due to stress relaxation as a %age of initial stress in steel.

5(b).

Sol^n:

Given Data,

$D = 400mm$

$B = 200mm$

$F_{ck} = 35 MPa$

$\sigma_p = 1500 MPa$

Let us suppose that the beam is stressed 100mm below the centre line of the section.

$\therefore d = 200+100 = 300mm$

$A_p = 10\left(\frac{\pi}{4}\times 7^2\right)$

$= 384.64 mm^2$

$\omega_p = \left(\frac{A_p}{Bd}\right)\left(\frac{\sigma_p}{f_{ck}}\right)$

$= \frac{384.64}{200\times 300} \times \frac{1500}{35}$

$= 0.274$

From table,

$\frac{\sigma_{pu}}{0.87\sigma_p} = 1$

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$\sigma_{pu} = 0.87 \times 1500$

$= 1305\ N/mm^2$

$\frac{x}{d} = \frac{\sigma_{pu}}{0.36\,\sigma_p} \times \omega_p = \frac{1305}{0.36 \times 1500} \times 0.274 = 0.66$

$x = 0.66d = 0.66 \times 300$

$x = 198mm$

$M_u = \sigma_{pu} \times A_p (d - 0.42x)$

$= 1305 \times 384.84 (300 - 0.42 \times 198)$

$= 108.90\ KN/m$

5(C).

Sol^n.

Given data,

$B = 150mm$

$D = 300mm$

$d = 300 - 60 = 240mm$

$\sigma_{ck} = 35 MPa$

$\sigma_p = 1500 MPa$

Area of prestressing steel.

$A_p = 15 \times \frac{\pi}{4} \times (4)^2$

$= 188.496 mm$

Prestressing Steel Index,

$\omega_p = \frac{A_p \sigma_p}{B d \sigma_{ck}}$

$= \frac{188.496 \times 1500}{150 \times 240 \times 35} = 0.22$

From table. For post tensioning beams $\omega_p = 0.22$

$\frac{\sigma_{pu}}{0.87\sigma_p} = 0.93$

$\sigma_{pu} = 0.93 \times 0.87 \times 1500 \Rightarrow 1213.65$

And $\frac{x}{d} = 0.46$

$x = 0.46 \times 240 = 110.4mm$

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

$= 1213.65 \times 188.496 (240 - 0.42 \times 110.4)$

$= 44.3\ KNm$

working moment $= \frac{44.3}{1.5} = 29.53\ KNm$

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5(d).

Sol^n. Let, $L_{eff} = 12 + 0.3 = 12.3m$

Super-imposed D.L. = $4 KN/m^2$

Live load = $2.5 KN/m^2$

Total Super imposed load = $6.5 KN/m^2$

Let Comp. Strength at transfer = 35 MPa

Characteristic Strength of tendons = 1600 MPa

Priliminary Discussions:

(A). $\frac{L}{24} \leq D \leq \frac{L}{16}$

$\frac{12.3}{24} \leq D \leq \frac{12.3}{16}$

$0.5125 \leq D \leq 0.7687m$

Let, $D = 700mm$

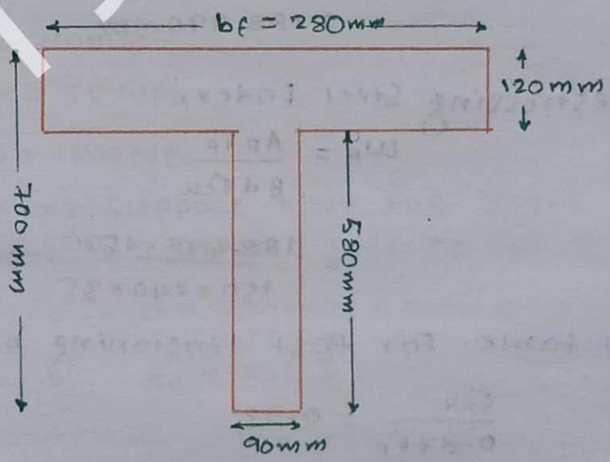
(B). $t_f = 120mm$ (depth of slab)

(C). Let. $\frac{b_f}{D} = 0.4$

$b_f = 280mm$

(D). Let $\frac{b_w}{b_f} = 0.3$

$b_w = 90mm$



Gross Area of Section

$A = (280 \times 120) + (580 \times 90)$

$A = 85800 mm^2$

$$\bar{x} = \frac{280 \times 120 \times 60 + 580 \times 90 \times (120 + \frac{580}{2})}{85800}$$

$\bar{x} = 272.93mm$

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MI of given section:

$$M.I = \left[\frac{280\times120^3}{12} + 280\times120\,(272.93-6)^2\right] + \left[90\times\frac{520^3}{12} + 520\times90\,(120+290-272.93)^2\right]$$

$$= 40.08\times10^8\ mm^4$$

$$r^2 = \frac{I}{A} = \frac{40.08\times10^8}{85800}$$

$$r^2 = 46.71\times10^3\ mm^2$$

Let, cover = 70mm

$e = 360mm$

$y_t = \bar{x} = 272.93 \simeq 293mm$

$y_b = 700 - 272.93 = 427.07\ mm$

$$\frac{e y_t}{r^2} = \frac{360\times273}{46.71\times10^3} = 2.1$$

$$\frac{e y_b}{r^2} = \frac{360\times427}{46710} = 3.29$$

$$Z_t = \frac{I}{y_t} = \frac{40.08\times10^8}{273} = 14.68\times10^6\ mm^3$$

$$Z_b = \frac{I}{y_b} = 9.36\times10^6\ mm^3$$

Limit of serviceability in compression:

First trial:

Self wt. of section $= \frac{85800}{10^6}\times25 = 2.125\ KN/m$

B.M due to D.L. $= M_d = 2.125\times\frac{12.3^2}{8} = 40.186\ KNm$

Super imposed load/m $= 6.5\ \frac{KN}{m}\times0.280 = 1.82\ KN/m$

BM due to super imposed load $= 1.82\times\frac{12.3^2}{8} = 2.79\ KNm$

$$\sigma_{tc} = \frac{M_d}{Z_t} = \frac{40.186\times10^6}{14.68\times10^6} = 2.737\ N/mm^2$$

$$\sigma_{bt} = \frac{M_d}{Z_b} = \frac{40.186\times10^6}{9.38\times10^6} = 4.284\ N/mm^2$$

$$\sigma_{pc} = \frac{M_l}{Z_t} = \frac{2.798\times10^6}{14.68\times10^6} = 0.19\ N/mm^2$$

$$\sigma_{bt} = \frac{M_l}{Z_b} = \frac{2.798\times10^6}{9.36\times10^6} = 0.299\ N/mm^2$$

Since $F_{ck} = 40MPa$

∴ Permissible comp. stress at transfer of stress

$$\sigma_{bc} = 0.51 f_{cc} = 0.51\times35 = 17.85\ N/mm^2$$

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Permissible tensile stress at transfer of PS

$\sigma_{tt} = 1 MPa$

Permissible comp. stress in concrete at service loads.

$$\sigma_{tc} = 0.41\, \sigma_{ck} - 0.06\, \frac{10}{30}\, \sigma_{ck}$$
$$= 0.39 \times 40$$
$$= 15.6 MPa$$

Permissible tensile stress in conc. at serviceable load $\sigma_{bt}' = 0$

Let determine the magnitude of prestressing force

$$P \leq \frac{A(\sigma_{tc} + \sigma_{tt}')}{\left(\frac{e y_t}{r^2} - 1\right)}$$

$$\leq \frac{85800(2.737+1)}{2.1-1}$$

$$\leq 291486 N \text{ ----- (i)}$$

$$P \leq \frac{A(\sigma_{bt} + \sigma_{bc}')}{\left(\frac{e y_b}{r^2} + 1\right)}$$

$$\leq 442680 N \text{ ------ (ii).}$$

$$P \geq \frac{A(\sigma_{tc} + 2\sigma_{tc} - \sigma_{tc})}{n\left(\frac{e y_t}{r^2} + 1\right)} \qquad n = \frac{P_o}{P} = 0.85$$

$$P \geq -1162934 N \text{ ---- (iii).}$$

$$P \geq \frac{85800(4.284 + 0.293 - 0)}{0.85(3.29+1)}$$

$$P \geq 107811.7 N \text{ ---- (iv).}$$

Considering eq^n (i), (ii), (iii) & (iv).

Take $P_{min} = 108000 N$

If initial prestress in tendon is 1200 MPa, area of PS steel,

$$A_p = \frac{P}{\sigma_{pu}} = \frac{108000}{1200} = 90 mm^2$$

use 5# 5mm ø high tension prestressing steel wire

$A_p = 98.125 mm^2$ OK.

Lets check eccentricity at support sect

$$e \leq k_t \left[\frac{A}{P}(2\sigma_{tc} + \sigma_{tt}') + 1\right]$$

$$e \leq k_b \left[\frac{A}{P}(2\sigma_{bt} + \sigma_{bc}') - 1\right]$$

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$$e \geq k_t\left[\frac{A}{P}(2\sigma_{tc} + e\sigma_{tc} - \sigma_{tc'}) + 1\right]$$

$$e \geq k_b\left[\frac{A}{P}(d\sigma_{bt} + e\sigma_{bt} - \sigma_{bc'}) - 1\right]$$

At Support Section.

$$d\sigma_{tc} = 0 = d\sigma_{bt} \text{ and } \frac{A}{P} = \frac{85800}{108000} = 0.794$$

$$P = 108000\,N, \quad A = 85800\,mm^2$$

$$K_t = \frac{r^2}{y_t} = \frac{46.91\times10^3}{273} = 171.09$$

$$K_b = \frac{r^2}{y_b} = 109.33$$

$$e \leq [0.044(0.8+1)+1] \times 171.09$$

$$e \leq 307\,mm$$

$$e \leq [0.794(0.0+17.85)-1]\,109.39$$

$$e \leq 1441\,mm$$

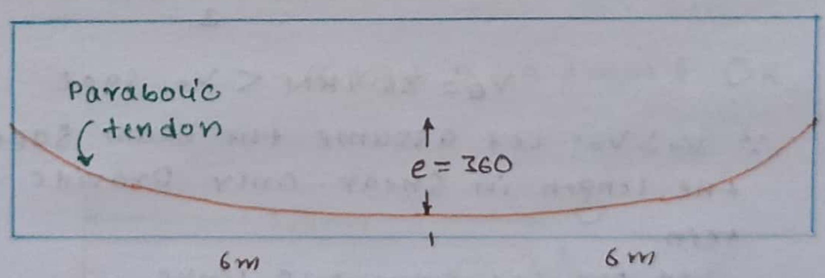
$$e \geq [0.794(0+75.6)+1]\,171.09$$

$$e \geq -1947$$

$$e \geq 109.39\,[0.794(0)-1]$$

$$e \geq -109.39$$

Now Providing PS wire in a parabolic curve shape with zero eccentricity at the support sect. and a max^m eccentricity of 360mm at mid span



Limit State of Collapse in Flexure:

Case-I N.A lies in Flange

Prestressing steel Index $w_p = \frac{A_p\,\sigma_p}{b_f\,d\,\sigma_{ck}}$

$$= \frac{98\times1600}{220\times630\times4}$$

$$= 0.022$$

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From Table $\frac{x}{d} = 0.054$

$x = 630 \times 0.054 = 34.02 < 120mm$

Now,

$M_u = \sigma_{pu} A_{st} (d - 0.42 x_u)$

$= 1600 \times 93 (630 - 0.42 \times 34)$

$= 96.5 \, KNm$

Total moment acting on T-beam

$M = 40.186 + 1.82 \Rightarrow 42 KNm$

Fact. moment, $M_u = 1.5 \times 42 \Rightarrow 63 KN < M_u$ Safe

Limit State of Collapse in Shear

Shear resistance is given by

$$V_o = 0.67 b_w D \sqrt{\sigma_t^2 + 0.8 \sigma_{cp} \sigma_t} + V_p$$

$$\sigma_{cp} = \frac{\eta P}{A} = \frac{0.85 \times 108000}{85000} = 1.07$$

$$\sigma_t = 0.24 \sqrt{24} = 1.52$$

$$\tan\theta = \frac{4e}{L} = \frac{4 \times 360}{12.3 \times 10^3} = 0.117 \simeq \sin\theta$$

$$V_p = P_o \sin\theta = 0.85 \times 108000 \times 0.117$$

$$= 10740.6 N$$

$$V_o = 0.67 \times 90 \times 700 \sqrt{1.52^2 + 0.6 \times 107 \times 1.52} + 10740.6$$

$$V_o = 90.96 \, KN$$

$$\text{Fact. Shear} = \frac{1.5 (2.125 \times 12.3 + 1.82 \times 12.3)}{2}$$

$$V_o' = 36.4 KN < V_o \text{ Safe}$$

$\because V_o > V_o'$ Let assume the beam safe throughout the length in shear. Only Provide minimum shear rein.

Let provide 6mm MS Links.

$$A_{sv} = \frac{\pi}{4} \times 6^2 = 28.3 \, mm^2$$

$$\text{Spacing, } S_v = \frac{0.87 \sigma_y A_{sv}}{0.4 b_w}$$

$$= \frac{0.87 \times 250 \times 28.3}{0.4 \times 90} = 171.14 mm$$

$$0.75d = 0.75 \times 630 = 472 mm$$

Provide 6mm Steel Links @ 170mm c/c

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Limit State of Serviceability for deflection:

Deflection due to PS force,

$$\Delta_P = \frac{5 P e L^2}{48 E I}$$

$$E_c = 4500\sqrt{40} = 28460.5 \text{ MPa}$$

$$\Delta_P = \frac{5 \times 108000 \times 630 \times 12300^2}{48 \times 28460.5 \times 48.02 \times 10^8} = 9.4 \text{mm} \uparrow$$

Deflection due to D.L. $= \frac{5}{384} \frac{W_d L^4}{E_c I}$

$$= 4.5 \times 10^{-4} \text{mm} \downarrow$$

Deflection due to Live/Superimposed load

$$\Delta_L = \frac{5}{384} \frac{W_L L^4}{E_c I}$$

$$= \frac{5}{384} \times \frac{1.82 \times 123200^2}{28460.5 \times 40.02 \times 10^8}$$

$$= 3.85 \times 10^{-4} \text{mm} \downarrow$$

Long term Modulus of Elasticity

$$E_{cL} = \frac{E_c}{1+\theta}$$

$$E_{cL} = 0.38 E_c$$

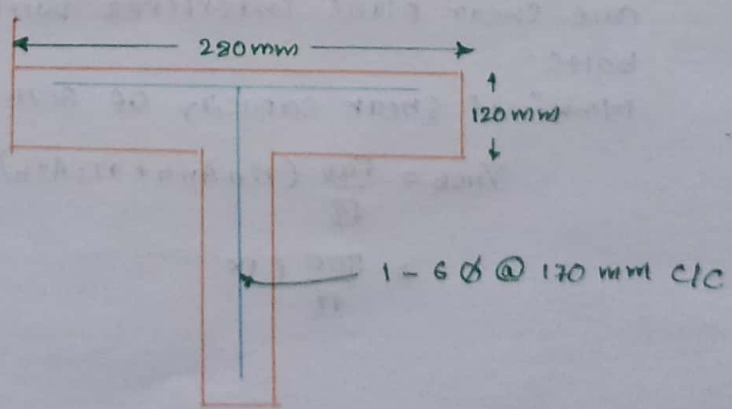
Long term net Deflection

$$\frac{4.5 \times 10^{-4}}{0.38} + 3.85 \times 10^{-4} - 0.26 \times 9.$$

$$= 7.99 \text{mm} \uparrow$$

Permissible deflection $= \frac{L}{250} \downarrow = \frac{12300}{250} = 49.2 \text{mm}$

$7.99 \uparrow > 49.2 \text{mm} \downarrow$ OK



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