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2 Marks Questions

1.

Sol. Structural design is the process of planning and analyzing a structure to ensure it is safe and stable under various loads, while structural detailing is the subsequent step of creating specific, detailed drawings and specifications for construction.

2.

Sol. PCC (Plain Cement Concrete) is a mix of cement, sand, aggregate and water, used for non-structural applications like flooring and pavements, while RCC (Reinforced cement concrete) adds steel reinforcement to handle both compressive and tensile forces, making it suitable for load-bearing structural elements like beams, columns and bridges.

3.

Sol. The modular ratio (m) in Reinforced cement concrete (RCC) is the ratio of the modulus of elasticity of steel to that of concrete, expressed as $m = E_s / E_c$.

4.

Sol. In reinforced concrete (RC) structures, the terms UR, OR and BAL refer to under-reinforced, over-reinforced, and balanced sections, respectively.

5.

Sol. The main methods for designing concrete structures are the Working Stress Method, the Ultimate Load Method, and the Limit State Method.

6.

Sol. Concrete and steel are graded by their strength, with concrete graded by its characteristic compressive strength (e.g., M20, M30) and steel graded by its yield strength (e.g., Fe415, Fe500).

7.

Sol. The lever arm is the perpendicular distance between the resultant compressive force in the concrete and the resultant tensile force in the steel reinforcement.

8.

Sol. M15 grade concrete is a mix of cement, sand, and coarse aggregate with a nominal ratio of 1:2:4 (cement:sand:aggregate) that has a characteristic compressive strength of 15 N/mm² (Newton per square millimeter) after 28 days of curing.

9.

Sol. The formula used is $k = \frac{m}{m + \frac{\sigma_{st}}{\sigma_{cbc}}}$, which gives the result $k \approx 0.475$.

10.

Sol. Permissible stress, also known as allowable stress, is the maximum stress a material can handle in a structure without permanent deformation or failure, calculated by dividing the material's failure stress (or ultimate stress) by a factor of safety.

11.

Sol. The formula used is $k = \frac{m}{m + \frac{\sigma_{st}}{\sigma_{cbc}}}$, which gives the result $k \approx 0.475$.

12.

Sol. The limit state of serviceability is a set of criteria in structural engineering to ensure that a structure performs its intended function without undue discomfort to users, damage to non-structural components, or loss of functionality under normal operating conditions and working loads.

13.

Sol. A limit state is a condition in which a structure becomes unfit for its intended use or collapses, exceeding acceptable limits for safety or serviceability.

14.

Sol. A partial safety factor in RCC is a multiplier used to account for uncertainties in materials and loads, ensuring a structure's strength and stability under the limit state of collapse. For example, the factor for concrete is 1.5 and for steel is 1.15, as specified by codes like IS 456.

15.

Sol. The partial safety factor for concrete is 1.5, and for steel it is 1.15 when assessing the strength of a structure for the limit state of collapse. This is a standard value often used in structural design codes like IS 456.

16.

Sol. Lapping is the process of overlapping two reinforcing steel bars side by side to transfer load from one bar to another.

17.

Sol. Effective depth is the distance from the top of a concrete section to the center of the tension reinforcement while overall depth is the total vertical distance from the top to the bottom of the section.

18.

Sol. Development length is the minimum length of reinforcing steel (rebar) required to be embedded in concrete to transfer stress from the steel to the concrete, ensuring a strong bond and preventing separation or failure.

19.

Sol. "Bond" is the adhesive and frictional force between steel reinforcement and the surrounding concrete that allows them to act as a single composite unit.

20.

Sol.

$$b_f = \frac{l_0}{\frac{l_0}{b} + 4} + b_w$$

21.

Sol. The minimum percentage of distribution steel for a slab is 0.15% of the gross cross-sectional area for mild steel and 0.12% of the gross cross-sectional area for HYSD (high yield strength deformed) steel.

22.

Sol. It is an architectural feature designed for ascending and descending between floors and includes the horizontal steps (treads), the vertical front (risers), and often a handrail or balustrade for support and safety.

23.

Sol. A one-way slab distributes load in a single direction and is supported by beams or walls on two opposite sides, while a two-way slab distributes load in two directions and is supported on all four sides.

24.

Sol. A dog-legged staircase is a space-efficient design featuring two parallel flights of stairs connected by a shared landing, which allows for a 180-degree turn.

25.

Sol.

(a) **Flight:-** A Flight refers to a continuous series of steps in a staircase between floors or landings without any interruption. A single staircase may consist of one or multiple flights. In buildings, a landing is often used to break up long flights, allowing a person to rest or for the stairs to change direction.

(b) **Riser:-** The riser is the vertical or near vertical board that connects one stair tread to the next.

(c) **Tread of a staircase:-** The tread is the horizontal part of a staircase that a person steps on to ascend or descend.

26.

Sol. The minimum cover requirement for footings is 50 mm (2 inches).

27.

Sol. The criterion for minimum eccentricity in column design is a mandatory design consideration,

calculated as the greater of (unsupported length/500) + (least lateral dimension/30) or 20 mm.

28.

Sol. The main difference is how they fail under a compressive load : short columns fail by crushing due to material strength, while long columns fail by buckling (bending out of shape) due to instability.

29.

Sol. The slenderness ratio is the ratio of a column's effective length to its least radius of gyration, and it is a crucial measure of a structural member's susceptibility to buckling under axial load. A higher ratio indicates a greater risk of buckling, which is when the column bends sideways and fails at a lower load than if simply crushed. Its importance lies in helping engineers distinguish between short (crushing) and slender (buckling) columns to determine the appropriate design method, ensuring structural stability and safety.

5 Marks Questions

30.

Sol.

- **Simplicity and ease of use:-** The method is straightforward to understand and apply, leading to faster design calculations.
- **Better serviceability:-** WSM tends to produce larger structural members, which results in better performance regarding deflection and cracking under normal working loads.
- **Essential for serviceability checks:-** It is the only method to investigate the serviceability states of deflection and cracking, making knowledge of WSM crucial, even when using the Limit State Method (LSM) for overall design.
- **Conservative design:-** The method is conservative because it restricts stresses to safe, allowable limits, which provides a good degree of safety and reliability.
- **Familiarity for experienced engineers:-** Because it was the traditional method, many experienced engineers are familiar with WSM and continue to use it for certain applications where it is still appropriate.

31.

Sol. The main disadvantages of the Working Stress Method (WSM) are that it results in uneconomical, over-designed structures because it doesn't account for the material's full strength beyond the elastic limit. It also fails to accurately model material behaviour under complex conditions like creep, shrinkage, and simultaneous loads, and does not consider the ultimate strength of the structure or its actual failure modes.

32.

Sol.

- **Linear Elastic Behaviour:-** Both concrete and steel are assumed to be elastic and their stress-strain relationship is linear, following Hooke's law.
- **Plane Section Remains Plane:-** A cross-section that is plane before bending remains plane after bending. This implies that the strain distribution across the section is linear.
- **Neglect of Tensile Strength of Concrete:-** The concrete's ability to resist tension is ignored. All tensile stresses are assumed to be carried by the steel reinforcement.
- **Perfect Bond:-** There is a perfect bond between the concrete and the steel reinforcement, meaning they deform together without slipping.
- **Working Loads:-** The method ensures that stress and strain are compatible across the section to maintain equilibrium. The design is based on the service or working loads, not factored or ultimate loads.

33.

Sol. **Step 1: Calculate the critical neutral axis depth (x_c)**

First, the critical neutral axis depth, x_c , is determined using the given values for the modular ratio (m) and the permissible stresses in concrete (σ_{cbc}) and steel (σ_{st}).



$$x_c = \frac{d}{1 + \frac{\sigma_{st}}{m\sigma_{cbc}}}$$

Given: $d = 715$ mm, $\sigma_{st} = 230$ N/mm², $\sigma_{cbc} = 7$ N/mm², and $m = 13.33$

$$x_c = \frac{715}{1 + \frac{230}{13.33 \times 7}} = \frac{715}{1 + \frac{230}{93.31}} = \frac{715}{1 + 2.4649} = \frac{715}{3.4649} \approx 206.36 \text{ mm}$$

Step 2: Calculate the moment of resistance ($M_{r,bal}$)

For a balanced section, the moment of resistance can be calculated with respect to either concrete or steel. Using the concrete stress, the formula is:

$$M_{r,bal} = \frac{1}{2} \sigma_{cbc} b x_c \left(d - \frac{x_c}{3} \right)$$

Given: $\sigma_{cbc} = 7$ N/mm², $b = 450$ mm, $x_c = 206.36$ mm, and $d = 715$ mm

$$M_{r,bal} = \frac{1}{2} \times 7 \times 450 \times 206.36 \times \left(715 - \frac{206.36}{3} \right)$$

$$M_{r,bal} = 1575 \times 206.36 \times (715 - 68.78)$$

$$M_{r,bal} = 325017 \times 646.22 \approx 210032485.7 \text{ N-mm}$$

To convert this to kN-m, divide by 10^6 :

$$M_{r,bal} = \frac{210032485.7}{10^6} \approx 210.03 \text{ kNm}$$

34.

Sol. The Limit State Method (LSM) is the advanced and preferred design philosophy over the older Working Stress Method (WSM) because it provides a more realistic, safe, and economical design. While WSM is based on conservative elastic behaviour, LSM accounts for the actual, non-linear behaviour of materials and the probabilistic variation of loads and material strength.

35.

Sol.

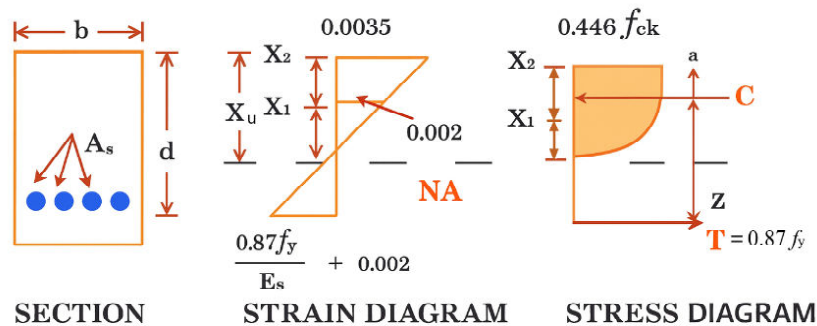


Fig.

From similar triangle of strain diagram,

$$\frac{0.0035}{x_u} = \frac{0.002}{x_1}$$

$$\therefore x_1 = 0.57x_u$$

Area of stress block = Area of rectangle + Area of parabola

$$= 0.446f_{ck} (x_u - 0.57x_u) + \frac{2}{3} \times 0.446f_{ck} \times (0.57x_u)$$

$$= 0.36 f_{ck} x_u$$

Compressive force = $C = 0.36 f_{ck} x_u b$

Tensile force $T = (\text{Area of steel}) \times (\text{stress})$

$$= A_{st} \times (0.87 f_y)$$

$$= 0.87 f_y A_{st}$$

36.

Sol. For a member under pure axial compression :

This applies to columns and short struts where the load is assumed to be applied concentrically.

Maximum concrete strain: The maximum compressive strain in the concrete is limited to 0.002.

Linear strain distribution:- For pure axial compression, the strain is assumed to be uniformly distributed across the entire cross-section.

Design stress:- The ultimate stress in the concrete is assumed based on the characteristic compressive strength (f_{ck}) and the partial safety factor for the material (1.5). This results in a design compressive stress of $0.446 f_{ck}$.

Load sharing:- Both the concrete and the steel reinforcement are assumed to share the axial compressive load.

37.

Sol. Under-reinforced sections are preferred over over-reinforced sections because they provide ductile failure, which gives warning signs before collapse, making them safer. In contrast, over-reinforced sections experience sudden, brittle failure without warning, which can lead to catastrophic collapse and loss of life. Under-reinforced designs ensure the steel reinforcement yields and deforms before the concrete crushes, providing time for evacuation and reducing hazards.

38.

Sol. Step 1: Calculate the geometrical properties of the column

First, we need to calculate the gross area of the concrete section (A_g) and the area of the steel reinforcement (A_{sc}).

- **Gross area of column (A_g):**

The column is square with dimensions $450 \text{ mm} \times 450 \text{ mm}$.

$$A_g = 450 \times 450 = 202500 \text{ mm}^2$$

- **Area of steel reinforcement (A_{sc}):**

The column has 8 bars of 16 mm diameter.

The area of a single 16 mm bar is given by $\pi r^2 = \pi (8)^2 \approx 201.06 \text{ mm}^2$

$$A_{sc} = 8 \times \left(\frac{\pi}{4} \times 16^2 \right) = 8 \times 201.06 = 1608.5 \text{ mm}^2$$

- **Net area of concrete (A_c):**

This is the gross area minus the area of the steel.

$$A_c = A_g - A_{sc} = 202500 - 1608.5 = 200891.5 \text{ mm}^2$$

Step 2: Determine the ultimate load capacity

The ultimate load (P_u) for a short column is calculated using the formula as per IS 456:2000.

The formula for the ultimate load capacity of a short column under axial compression is:

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

where:

f_{ck} = Characteristic compressive strength of concrete = 20 N/mm^2 (M 20 concrete).

f_y = Characteristic yield strength of steel = 415 N/mm^2 (Fe 415 steel).

A_c = Area of concrete = 200891.5 mm^2 .

A_{sc} = Area of steel = 1608.5 mm^2 .

Substituting the values into the formula:

$$P_u = 0.4 \times 20 \times 200891.5 + 0.67 \times 415 \times 1608.5$$

$$P_u = 8 \times 200891.5 + 278.05 \times 1608.5$$

$$P_u = 1607132 + 447243.425$$

$$P_u = 2054375.425 \text{ N}$$

To express the load in kilonewtons (kN), divide by 1000:

$$P_u = \frac{2054375.425}{1000} = 2054.37 \text{ kN}$$

39.

Sol. General formula for development length

The formula for development length (L_d) is:

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}}$$

(i) Required development length in tension

1. Calculate the bond stress for deformed bars in tension:

a. τ_{bd} = (Basic bond stress for M25) $\times$ (Increase for deformed bars)

b. $\tau_{bd} = 1.4 \text{ N/mm}^2 \times 1.60 = 2.24 \text{ N/mm}^2$

2. Substitute the values into the development length formula:

a. $L_d = \frac{\phi \times (0.87 \times 415)}{4 \times 2.24}$

b. $L_d = \frac{\phi \times 361.05}{8.96}$

c. $L_d \approx 40.3\phi$

Result for tension: The development length required for a bar in tension is approximately 40.3ϕ .

(ii) Required development length in compression

1. Calculate the bond stress for bars in compression:

a. The bond stress for deformed bars in compression is increased by 25% over the tension value.

b. τ_{bd} (in compression) = (τ_{bd} in tension) $\times$ 1.25

c. τ_{bd} (in compression) = $2.24 \text{ N/mm}^2 \times 1.25 = 2.8 \text{ N/mm}^2$.

2. Substitute the values into the development length formula:

a. $L_d = \frac{\phi \times (0.87 \times 415)}{4 \times 2.8}$

b. $L_d = \frac{\phi \times 361.05}{11.2}$

c. $L_d = 32.24\phi$

Result for compression: The development length required for a bar in compression is approximately 32.24ϕ .

40.

Sol. Step 1: Find the design bond stress (τ_{bd})

As per IS 456:2000, Table 26.2.1.1, the design bond stress for M20 grade concrete in plain bars is 1.2 N/mm^2 . Since the bar is of Fe415 grade, it is a deformed bar. For deformed bars, the bond stress is increased by 60%.

$$\tau_{bd} = 1.2 \times 1.6 = 1.92 \text{ N/mm}^2$$

Step 2: Calculate the development length in tension

The formula for development length (L_d) is given by:

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}}$$

where:

ϕ = diameter of the bar = 12 mm

f_y = characteristic strength of steel = 415 N/mm^2 (For Fe415 grade)

τ_{bd} = design bond stress = 1.92 N/mm^2

Substituting the values:

$$L_d = \frac{0.87 \times 415 \times 12}{4 \times 1.92} = 564.14 \text{ mm}$$

Step 3: Calculate the development length in compression

For development length in compression, the bond stress is further increased by 25%.

$$\tau_{bd, \text{compression}} = 1.92 \times 1.25 = 2.4 \text{ N/mm}^2$$

The formula for development length in compression is:

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd, \text{compression}}}$$

Substituting the values:

$$L_d = \frac{0.87 \times 415 \times 12}{4 \times 2.4} = 451.31 \text{ mm}$$

Step 4: Calculate the anchorage value for a 90° bend

The anchorage value of a standard hook (which is a 90° bend) is given by the formula:

$$8\phi$$

where, ϕ is the diameter of the bar.

$$\text{Anchorage value} = 8 \times 12 = 96 \text{ mm}$$

41.

Sol. **Advantages of T-beams:-**

- **Higher flexural strength:-** The wide flange acts as a large compression zone, increasing the moment of inertia and making the beam much more effective at resisting bending forces.
- **Increased stiffness and reduced deflection:-** The T-beam's shape increases its stiffness, which helps to reduce deflection under heavy loads, preventing cracks and damage over time.
- **Material efficiency:-** By using the slab as the flange, less concrete and steel are needed for a T-beam compared to a rectangular beam with the same strength.
- **Longer spans:-** T-beams can span longer distances without increasing their depth compared to rectangular beams, making them ideal for large commercial buildings and bridges.
- **Reduced floor depths:-** The flange is part of the slab, so the total floor depth can be reduced, which saves materials and allows for more headroom in multi-story buildings.

42.

Sol.

1. Loads and load factors

- Live load (L_L): 3 kN/m^2
- Floor finish load: Assume 1 kN/m^2 (typical value)
- Self-weight (SW): 25 kN/m^3 (for M20) $\times$ Slab thickness (D)
- Ultimate load (w_u): Factor of safety applied to total load.
 - Assume a slab thickness, e.g., 125 mm (or 0.125 m)
 - $\text{SW} = 25 \times 0.125 = 3.125 \text{ kN/m}^2$
 - Total working load $= \text{SW} + L_L + \text{Floor finish} = 3.125 + 3 + 1 = 7.125 \text{ kN/m}^2$
 - $w_u = 1.5 \times 7.125 = 10.6875 \text{ kN/m}^2$

2. Design for bending moment

- Maximum bending moment (M_u):- The formula for a cantilever is $M_u = \frac{w_u \times L^2}{2}$

$$M_u = \frac{10.6875 \times (1.25)^2}{2} = 8.35 \text{ kN.m}$$

- Maximum shear force (V_u):- The formula is $V_u = w_u \times L$
 - $V_u = 10.6875 \times 1.25 = 13.36 \text{ kN}$

3. Determine effective depth (d) and thickness (D)

- Assume effective depth (d): A deflection check is critical for cantilever slabs. A reasonable starting point for the depth ratio (L/d) is 9.7 (from IS 456).
 - $L/d = 9.7 \Rightarrow 1.25/d = 9.7 \Rightarrow d = 1.25/9.7 \approx 0.129 \text{ m}$ or 129 mm



- Adopt effective depth (d) and thickness (D): Use a practical value, such as $d = 130$ mm.
- Total thickness (D): Consider clear cover (25 mm) and half the bar diameter (assume 12 mm bar).
 - $D = d + \text{cover} + (\text{bar diameter}/2) = 130 + 25 + 6 = 161$ mm

43.

Sol.

Step 1: Assumptions and parameters

- Design axial load (P_u): 1800 kN
- Effective length (L_{eff}): 2.25 m = 2250 mm
- Grade of concrete (f_{ck}): M25 (25 N/mm²)
- Grade of steel (f_y): Fe 415 (415 N/mm²)
- Assume a square column: Size $b \times b$

Step 2: Check for slenderness (Short vs Long column)

- **Calculate minimum lateral dimension:** $12 \times L_{eff}$ (as per IS 456:2000)
 - $12 \times 2.25 = 27$ m (This is incorrect, slenderness ratio calculation depends on the least lateral dimension. Re-calculation needed).
- **Assuming a 400 mm $\times$ 400 mm column**, the least lateral dimension is 400 mm.
- **Calculate slenderness ratio:** $\frac{L_{eff}}{b} = \frac{2250}{400} = 5.625$
- **For a short column**, the slenderness ratio must be less than $12 \times 1 = 12$ (in the direction of the least lateral dimension). Since $5.625 < 12$, the column is

Step 3: Calculate required steel area (A_{sc})

- Use the design equation for columns: $P_u = 0.4f_{ck}A_g + 0.67f_yA_{sc}$
- Gross area (A_g): $b^2 = 400 \text{ mm} \times 400 \text{ mm} = 160,000 \text{ mm}^2$
- Plug in the values: $1800 \text{ kN} = 1800 \times 10^3 \text{ N}$
 - $1800 \times 10^3 = 0.4 \times 25 \times 160,000 + 0.67 \times 415 \times A_{sc}$
 - $1800 \times 10^3 = 1,600,000 + 278.05 \times A_{sc}$
 - $1800 \times 10^3 - 1,600,000 = 278.05 \times A_{sc}$
 - $200,000 = 278.05 \times A_{sc}$
 - $A_{sc} = \frac{200,000}{278.05} \approx 719.29 \text{ mm}^2$
- **Calculate the steel percentage:**
 - $p = \frac{A_{sc}}{A_g} \times 100 = \frac{719.29}{160,000} \times 100 \approx 0.45\%$

44.

Sol.

Step 1: Calculate the geometrical and material properties

First, we will calculate the gross area of the column, the area of the steel reinforcement, and the area of the concrete.

- **Gross area of column (A_g):**
 $A_g = 450 \text{ mm} \times 450 \text{ mm} = 202500 \text{ mm}^2$
- **Area of steel reinforcement (A_{sc}):**
 The column has 8 bars of 16 mm diameter.

$$A_{sc} = 8 \times \frac{\pi}{4} \times (16 \text{ mm})^2 = 1608.49 \text{ mm}^2$$

- **Area of concrete (A_c):**
 $A_c = A_g - A_{sc} = 202500 \text{ mm}^2 - 1608.49 \text{ mm}^2 = 200891.51 \text{ mm}^2$
- **Material strengths:**
 Characteristic compressive strength of concrete (f_{ck}): 20 N/mm² (from M20)
- **Material strengths:**
 Characteristic compressive strength of concrete (f_{ck}): 20 N/mm² (from M20 grade)

Characteristic yield strength of steel (f_y): 415 N/mm² (from Fe 415 grade)

Step 2: Check for slenderness ratio

The slenderness ratio is calculated to confirm that the column is a short column, as stated in the problem. The slenderness ratio is the ratio of the effective length to the least lateral dimension.

- **Slenderness ratio (λ):**

$$\lambda = \frac{L_{\text{eff}}}{D} = \frac{2500 \text{ mm}}{450 \text{ mm}} = 5.56$$

Since the slenderness ratio is less than 12, the column is considered a short column.

Step 3: Calculate the ultimate load

For a short column, the ultimate load (P_u) is calculated using the formula from the Limit State Method (LSM) as per IS 456:2000.

Ultimate load formula:

$$P_u = 0.4 \times f_{ck} \times A_c + 0.67 \times f_y \times A_{sc}$$

Calculation:-

$$P_u = (0.4 \times 20 \times 200891.51) + (0.67 \times 415 \times 1608.49)$$

$$P_u = 1607132.08 + 447240.64 = 2054372.72 \text{ N}$$

To express the load in kilonewtons (kN), divide by 1000 :

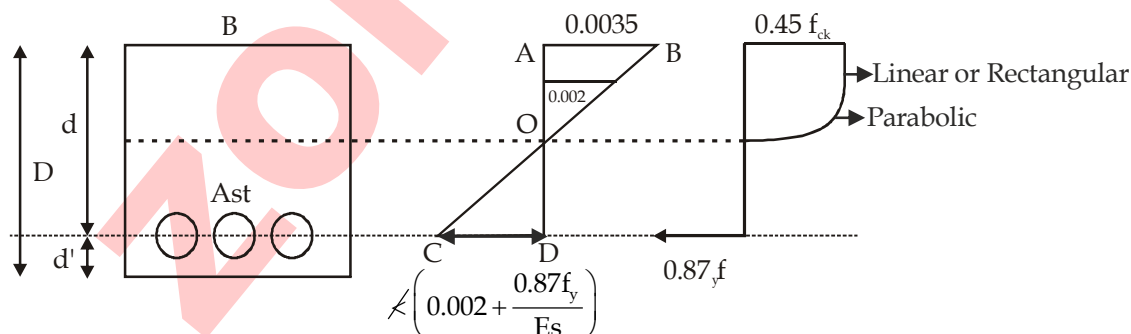
$$P_u = \frac{2054372.72}{1000} = 2054.37 \text{ kN}$$

45.

Sol.

Assumption of limit state method

1. The plane s/c remain plane before and after bending it means the strain variation is linear from top to bottom and changes its sign from compression to tension.
2. The maximum compressive strain taken in concrete is 0.0035.
3. The stress diagram is parabolic from 0 to 0.002 strain and straight from 0.002 to 0.0035.
4. The tensile strength of concrete is ignored because concrete is weak in tension area below the N.A. is ignored so we can say the tensile stress is taken by steel only.
5. The partial F.O.S for concrete and steel is taken as 1.5 and 1.15 respectively.
6. The higher F.O.S is taken for concrete as compare to steel because the quality of concrete change more as compare to steel in laboratory or site or we can say the quality control of steel is better as compare to quality control of concrete.



46.

Sol.

Note:- In question 20 days is mention so please change 28 days in place of 20 days

Given,

Grade of concrete M20

Age of concrete 28 days

As we know that

$$\text{long term elastic modulus of concrete} = E_{CR} = E_c / 1 + \theta$$

where,

$$E_c = 5000 \sqrt{f_{ck}} = 5000 \times \sqrt{20} = 22360.67 \text{ mPa}$$

θ = Creep coefficient for 28 days = 1.6

$$E_{CR} = 22360.67 / 2.6 = 8600.26 \text{ mPa}$$

47.

Sol. Development length (L_d) $\Rightarrow$

$\Rightarrow$ Development length is the minimum length of R/F is to be embedded (inserted) in the concrete to balance the additional force developed in the tension R/F and for development length the bond stress is taken maximum so it is permissible bond stress.

$\Rightarrow$ Maximum force developed in R/F

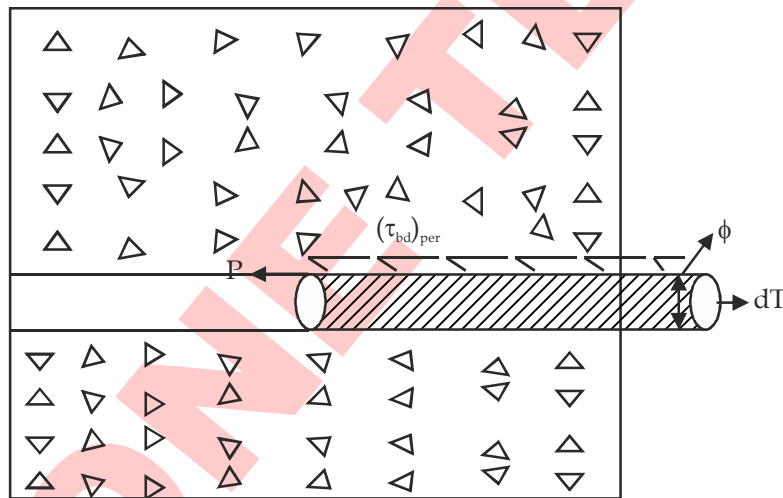
$$dT = \sigma_{st} \times \frac{\pi}{4} \times (\phi)^2 \times n \quad \text{.....(1)}$$

Force resisted by $(\tau_{bd})_{per}$.

$$P = (\tau_{bd})_{per} \times n\pi\phi \times L_d \quad \text{.....(2)}$$

Significance:

Without sufficient development length bars can slip, compromising the structures ability to carry loads and leading to premature failure.



$$\sigma_{st} \times \frac{\pi}{4} \times (\phi)^2 \times n = (\tau_{bd})_{per} \times n\pi\phi \times L_d$$

$$L_d = \frac{\sigma_{st}\phi}{4(\tau_{bd})_{per}} \Rightarrow \text{Acc to WSM}$$

$$L_d = \frac{0.87f_y\phi}{4(\tau_{bd})_{per}} \Rightarrow \text{Acc to LSM}$$

20 Marks Questions

48.

Sol.

Step 1: Calculate the maximum bending moment

The maximum bending moment for a simply supported beam with a uniformly distributed load

(UDL) is calculated using the formula $M = \frac{wL^2}{8}$

Given:

- Uniformly distributed load, $w = 25000 \text{ N/m}$
- Span, $L = 5 \text{ m}$

$$M = \frac{25000 \times 5^2}{8} = \frac{25000 \times 25}{8} = 78125 \text{ Nm}$$

$$M = 78125 \text{ Nm} = 78.125 \times 10^6 \text{ Nmm}$$

Step 2: Determine Permissible stresses and Modular ratio

Based on the given material grades and the working stress method (WSM):

- For M20 concrete, the permissible compressive stress in bending is $\sigma_{cbc} = 7 \text{ N/mm}^2$
- For Fe415 steel, the permissible tensile stress is $\sigma_{st} = 230 \text{ N/mm}^2$

- The modular ratio for M20 concrete is calculated as $m = \frac{280}{3\sigma_{cbc}}$

$$m = \frac{280}{3 \times 7} = \frac{280}{21} \approx 13.33$$

Step 3: Calculate design constants

The design constants for a balanced section are calculated using the permissible stresses and modular ratio.

- Neutral axis depth factor, k :

$$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{13.33 \times 7}{13.33 \times 7 + 230} = \frac{93.31}{93.31 + 230} = \frac{93.31}{323.31} \approx 0.288$$

Lever arm factor, j :

$$j = 1 - \frac{k}{3} = 1 - \frac{0.288}{3} = 1 - 0.096 = 0.904$$

Moment of resistance factor, R :

$$R = \frac{1}{2} \sigma_{cbc} k j = \frac{1}{2} \times 7 \times 0.288 \times 0.904 \approx 0.910$$

Step 4: Check if the beam is singly or doubly reinforced

First, determine the effective depth (d) of the beam. Assuming a clear cover of 40 mm and a 20 mm diameter bar, the effective cover is approximately.

$$40 + \frac{20}{2} = 50 \text{ mm}$$

- Overall depth, $D = 500 \text{ mm}$
- Effective depth, $d = D - \text{effective cover} = 500 - 50 = 450 \text{ mm}$
- Width, $b = 300 \text{ mm}$

Now, calculate the limiting moment of resistance (M_{lim}) of the concrete section.

$$M_{lim} = Rbd^2 = 0.910 \times 300 \times (450)^2 = 55.28 \times 10^6 \text{ Nmm}$$

Comparing the applied bending moment (M) with the limiting moment (M_{lim}):

- $M = 78.125 \times 10^6 \text{ Nmm}$
- $M_{lim} = 55.28 \times 10^6 \text{ Nmm}$

Since $M > M_{lim}$, the beam is over-reinforced. Compression steel is required, and the beam needs to be designed as a doubly reinforced section.

Step 5: Calculate the area of steel

The total moment of resistance is the sum of the moment resisted by the concrete tension steel couple and the moment resisted by the compression steel tension steel couple.

$$M = M_{lim} + M_2$$

$$M_2 = M - M_{lim} = (78.125 - 55.28) \times 10^6 = 22.845 \times 10^6 \text{ Nmm}$$

The area of tension steel is composed of two parts: A_{st1} to resist M_{lim} and A_{st2} to resist M_2 .

$$\bullet \quad A_{st1} = \frac{M_{lim}}{\sigma_{st} j d} = \frac{55.28 \times 10^6}{230 \times 0.904 \times 450} \approx 587.72 \text{ mm}^2$$

$$\bullet \quad A_{st2} = \frac{M_2}{\sigma_{st} (d - d')} = \frac{22.845 \times 10^6}{230 \times (450 - 50)} \approx 248.31 \text{ mm}^2$$

$$\bullet \quad \text{Total tension steel, } A_{st} = A_{st1} + A_{st2} = 587.72 + 248.31 = 836.06 \text{ mm}^2$$

The area of compression steel (A_{sc}) is determined by the moment M_2 .

$$\text{The stress in compression steel is } \sigma_{sc} = \frac{m \sigma_{cbc} (kd - d')}{kd}$$

$$\sigma_{sc} = \frac{13.33 \times 7 (0.288 \times 450 - 50)}{0.288 \times 450} = \frac{93.31 (129.6 - 50)}{129.6} = \frac{93.31 \times 79.6}{129.6} \approx 57.3 \text{ N/mm}^2$$

$$A_{sc} = \frac{M_2}{\sigma_{sc} (d - d')} = \frac{23.135 \times 10^6}{57.3 \times (450 - 50)} = \frac{23.135 \times 10^6}{22920} \approx 1009.47 \text{ mm}^2$$

49.

Sol. Step 1: Calculate loads and maximum bending moment

The self-weight of the beam is calculated using a concrete density of 25 kN/m^3 .

$$w_{\text{self-weight}} = \text{cross-section area} \times \text{density} = (0.3 \text{ m} \times 0.6 \text{ m}) \times 25 \text{ kN/m}^3 = 4.5 \text{ kN/m}$$

The total uniformly distributed load (UDL) is the sum of the superimposed load and the self weight.

$$w_{\text{total}} = 60 \text{ kN/m} + 4.5 \text{ kN/m} = 64.5 \text{ kN/m}$$

For a simply supported beam, the maximum bending moment ($M_{\max}$) is given by:

$$M_{\max} = \frac{w_{\text{total}} L^2}{8} = \frac{64.5 \times (6)^2}{8} = 290.25 \text{ kNm}$$

Step 2: Determine permissible stresses and section properties

For M20 concrete and Fe415 steel, the permissible stresses according to the working stress method (WSM) are:

- Permissible compressive stress in concrete, $\sigma_{cbc} = 7 \text{ N/mm}^2$
- Permissible tensile stress in steel, $\sigma_{st} = 230 \text{ N/mm}^2$

The modular ratio (m) is:

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

Assuming an effective cover of 50 mm, the effective depth (d) is:

$$d = \text{Overall depth} - \text{cover} = 600 \text{ mm} - 50 \text{ mm} = 550 \text{ mm}$$

The breadth of the beam is $b = 300 \text{ mm}$.

Step 3: Check beam section for bending resistance

First, calculate the moment of resistance (M_R) of a balanced section to check if the given dimensions are adequate.

The critical neutral axis depth factor (k) is:

$$k = \frac{m \sigma_{cbc}}{m \sigma_{cbc} + \sigma_{st}} = \frac{13.33 \times 7}{(13.33 \times 7) + 230} = 0.2886$$

The lever arm factor (j) is:

$$j = 1 - \frac{k}{3} = 1 - \frac{0.2886}{3} = 0.9038$$

The moment of resistance of the balanced section is:

$$M_R = \frac{1}{2} \sigma_{cbc} k_j b d^2 = \frac{1}{2} \times 7 \times 0.2886 \times 0.9038 \times 300 \times (550)^2 = 82.85 \times 10^6 \text{ N-m}$$

Since M_R (82.85 kNm) is less than the maximum bending moment ($M_{\max}$) (290.25 kNm), the section is adequate as a doubly reinforced beam. (but in WSM over rainforce section is allow)

Step 4: Calculate required area of steel

The required area of steel (A_{st}) is calculated based on the maximum bending moment.

$$A_{st} = \frac{M_{\max}}{\sigma_{st} j d} = \frac{290.25 \times 10^6}{230 \times 0.9038 \times 550} = 2538.22 \text{ mm}^2$$

We will use 25 mm diameter bars. The area of one 25 mm bar is $\frac{\pi}{4} (25)^2 = 490.87 \text{ mm}^2$

Number of bars required: $\frac{2538.22}{490.87} \approx 5.17$. We will provide 6 bars.

Area of steel provided, $A_{st, \text{prov}} = 6 \times 490.87 = 2945.22 \text{ mm}^2$.

50.
Sol.

Step 1: Convert to ultimate moment

- Ultimate moment (M_u): $75 \text{ kN-m} \times 1.5 = 112.5 \text{ kN-m}$

Step 2: Determine Limiting moment of resistance ($M_{u, \text{lim}}$)

- The formula for $M_{u, \text{lim}}$ for a rectangular beam with Fe415 steel is $0.138 f_{ck} b d^2$
- **Note:-** Since the beam's width (b) and effective depth (d) are not given, we can use a reasonable depth-to-width ratio ($d/b \approx 2$) to find them.
- Let's assume a common width, say $b = 230 \text{ mm}$.
- Then, $M_{u, \text{lim}} = 0.138 \times 25 \times 230 \times d^2 = 112.5 \times 10^6 \text{ N-mm}$
- $d^2 = 141776.938$
- $d = 376.53$
- The overall depth (D) would be $d + \text{cover} \approx 376.53 + 33 = 409.53 \approx 410 \text{ mm}$.

Step 3: Determine steel reinforcement (A_{st})

- The area of steel is given by the formula:

$$A_{st} = \frac{0.5 f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}} \right] b d$$

$$A_{st} = \frac{0.5 \times 25 \times 230 \times 376.53}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 112.5 \times 10^6}{25 \times 230 \times 376.53^2}} \right]$$

- $A_{st} \approx 1656 \text{ mm}^2$
- This can be achieved with 9 bars of 16 mm diameter.

Final design

- **Beam size:** 230 mm $\times$ 410 mm
- **Reinforcement:** 9 bars of 16 mm diameter (at the bottom)
- **Concrete:** M25 grade
- **Steel:** Fe415 grade

51.
Sol.

1. Design parameters and calculations

- Working load (P_{uw}) = 900 kN
- Factored load (P_u) = $1.5 \times 900 \text{ kN} = 1350 \text{ kN}$
- Concrete grade = M20
- Steel grade = Fe415



- Unsupported length (L_u) = 4.8 m
 - End conditions: Held in position but not restrained in direction. This means the effective length factor (K_L) is 1.0, as per IS 456:2000, clause 25.1.4.1.
 - Effective length (L_e): $L_e = 1.0 \times 4.8 \text{ m} = 4.8 \text{ m}$
 - Assume column as short column
 - Minimum eccentricity ($e_{\min}$): $e_{\min} = \max\left(\frac{L_e}{500} + \frac{D}{30}, 20 \text{ mm}\right)$
 - We need to assume a size first to calculate the minimum eccentricity.
 - Assume a size of 350 mm × 350 mm
 - $e_{\min} = \max\left(\frac{4800}{500} + \frac{350}{30}, 20\right) = \max(9.6 + 11.67, 20) = \max(21.27, 20) = 21.27$
 - **Slenderness ratio:** $e_{\min}$ does not exceed $0.05 \times D$
 - $0.05 \times 350 \text{ mm} = 17.5 \text{ mm}$
 - Since $e_{\min} > 17.5 \text{ mm}$, the column is a short column for design purposes.
- 2. Trial size calculation**
- $P_u = 0.45 f_{ck} (A_g - A_{sc}) + 0.75 f_y A_{sc}$
 - Assume $A_{sc} = 1\%$ of A_g
 - $1350 \text{ kN} = 0.45(20)(A_g - 0.01A_g) + 0.75(415)(0.01A_g)$
 - $1350 = 9A_g - 0.09A_g + 3.1125 A_g$
 - $A_g = \frac{1350}{9 + 3.1125 - 0.09} = \frac{1350}{12.0225} \approx 104.15 \text{ m}^2$
 - $A_g = 112.289 \times 10^6 \text{ mm}^2$
 - Side of the column: $D = \sqrt{112.289 \times 10^6} = 10.59 \text{ m}$. This is the required size for a short column with minimum eccentricity.
 - For a long column, the reduction factor δ needs to be calculated, and the area of steel A_{sc} must be considered as 1.05 times the area for a short column.

52.

Sol.

Step 1: Given data and material properties

The given data for the rectangular beam is as follows:

- Beam section dimensions: 230 mm × 440 mm effective depth
- Reinforcement: 6 × 16 mm diameter bars at the bottom
- Factored shear force (V_u): 200 kN
- Concrete grade: M20
- Steel grade: Fe 415

The material properties are:

- For M20 concrete, the characteristic compressive strength is $f_{ck} = 20 \text{ N/mm}^2$
- For Fe 415 steel, the characteristic yield strength is $f_y = 415 \text{ N/mm}^2$.
- The effective depth (d) is 440 mm
- The width of the beam (b) is 230 mm

- The area of tension steel (A_s) is $6 \times \frac{\pi}{4} \times (16)^2 = 1206.37 \text{ mm}^2$

Step 2: Calculate the nominal shear stress (τ_v)

The nominal shear stress is calculated using the formula:

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

where $V_u = 200 \text{ kN} = 200 \times 10^3 \text{ N}$

$$\tau_v = \frac{200 \times 10^3}{230 \times 440} = 1.97 \text{ N/mm}^2$$

Step 3: Calculate the design shear strength of concrete (τ_c)

First calculate the percentage of tension steel (p_t):

$$p_t = \frac{100A_s}{bd} = \frac{100 \times 1206.37}{230 \times 440} = 1.19\%$$

Using the IS 456:2000 table for M20 concrete and $p_t = 1.19\%$ the value of τ_c can be found by interpolation:

For $p_t = 1.00$, $\tau_c = 0.62 \text{ N/mm}^2$

For $p_t = 1.25$, $\tau_c = 0.67 \text{ N/mm}^2$

$$\tau_c = 0.62 + (0.67 - 0.62) \times \frac{1.19 - 1.00}{1.25 - 1.00} = 0.658 \text{ N/mm}^2$$

Step 4: Check if shear reinforcement is required

We must compare the nominal shear (τ_v) with the design shear strength of concrete (τ_c).

Since $\tau_v (1.97 \text{ N/mm}^2) > \tau_c (0.658 \text{ N/mm}^2)$ shear reinforcement is required.

We also need to check against the maximum shear stress allowed ($\tau_{c, \max}$) for M20 concrete, which is 2.8 N/mm^2

Since $\tau_v (1.97 \text{ N/mm}^2) < \tau_{c, \max} (2.8 \text{ N/mm}^2)$, the section is safe in shear and can be designed.

Step 5: Calculate the shear to be resisted by reinforcement (V_{us})

The shear force to be resisted by the stirrups is:

$$V_{us} = V_u - V_{uc} = V_u - (\tau_c \times b \times d)$$

$$V_{us} = 200 \times 10^3 - (0.658 \times 230 \times 440) = 200000 - 66589.6 = 133410.4 \text{ N}$$

Step 6: Design the shear reinforcement

Using 8 mm diameter two-legged stirrups, the area of shear reinforcement (A_{sv}) is:

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

The spacing (s_v) for these stirrups is calculated using the formula:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

$$s_v = \frac{0.87 \times 415 \times 100.53 \times 440}{133410.4} = 119.70 \text{ mm}$$

Provide stirrups at a spacing of 110 mm center-to-center.

53.

Sol.

- **Effective depth:** A common ratio for effective depth (d) to span (L) is $1/10$ to $1/12$. We will assume an effective depth of 400 mm. This assumption needs to be verified after the design.
- **Cover:** A clear cover of 25 mm is assumed.

1. Calculations

- **Effective span:**

$$L_{\text{eff}} = \text{clear span} + \text{effective depth} = 6000 \text{ mm} + 400 \text{ mm} = 6400 \text{ mm}$$

$$\text{Factored load: } w_u = 30 \text{ kN/m}$$

$$\text{Maximum bending moment: } M_u = \frac{W_u L_{\text{eff}}^2}{8} = \frac{30 \times (6.4)^2}{8} = 153.6 \text{ kNm}$$

$$\text{Maximum shear force: } V_u = \frac{W_u L_{\text{eff}}}{2} = \frac{30 \times 6.4}{2} = 96 \text{ kN}$$

2. Flexural design

- **Check for singly reinforced beam:** Calculate the limiting moment of resistance ($M_{u, \text{lim}}$) for Fe415 steel, which is $0.138 f_{ck} b d^2$
 $M_{u, \text{lim}} = 0.138 \times 25 \times 230 \times (400)^2 \approx 126.96 \text{ kNm}$
- **Compare moment capacities:** Since $M_u (153.6 \text{ kNm}) > M_{u, \text{lim}} (126.96 \text{ kNm})$, the section can be designed as a doubly reinforced section.
- **Calculate steel area:**



$$A_{st1} = \frac{(m_{u\lim})}{0.87 \times f_y \times (d - 0.42 \times x_{u\lim})} = \frac{126.96 \times 10^6}{0.87 \times 415 \times (400 - 0.42 \times 192)}$$

$$= \frac{126.96 \times 10^6}{0.87 \times 415 \times (400 - 80.64)} = \frac{126.96 \times 10^6}{115304.928}$$

$$A_{st1} = 1101.08 \text{ mm}^2$$

$$A_{st2} = \frac{Bm_u - m_{u\lim1}}{0.87 \times f_y \times (d - d_c)} = \frac{153.6 - 126.96}{0.87 \times 415 \times (400 - 35)}$$

$$d_c = \text{clear cover} + \frac{\phi}{2} (\because \phi = 20 \text{ mm})$$

$$A_{st2} = 202.15 \text{ mm}^2$$

$$A_{sc} = \frac{Bm_u - m_{u\lim}}{(f_{sc} - 0.45f_{ck}) \times (d - d_c)} = \frac{153.6 - 126.96}{(350 - 0.45 \times 25) \times (400 - 35)}$$

$$A_{sc} = 215.45 \text{ mm}^2$$

4. Check for Shear

- **Nominal shear stress:** Calculate the nominal shear stress (τ_v) using the formula

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

$$\tau_v = \frac{96 \times 10^3}{230 \times 400} = 1.04 \text{ N/mm}^2$$

$$\tau_{\max} = 0.62\sqrt{f_{ck}} = 0.62 \times \sqrt{25} = 3.1 \text{ N/mm}^2$$

So, beam is safe in shear

54.

Sol.

- Effective depth, $d = 500 \text{ mm}$
- Breadth, $b = 350 \text{ mm}$
- Tension reinforcement, 4 - 25 mm diameter bars
- Factored shear force, $V_u = 350 \text{ kN} = 350 \times 10^3 \text{ N}$

Step 1: Calculate the area of steel (A_{st}) and material properties

The area of steel for 4 bars of 25 mm diameter is calculated.

$$A_{st} = 4 \times \frac{\pi}{4} \times (25)^2$$

$$A_{st} = 4 \times 490.87 = 1963.49 \text{ mm}^2$$

The material properties for M20 concrete and Fe415 steel are:

- Characteristic strength of concrete, $f_{ck} = 20 \text{ N/mm}^2$
- Characteristic strength of steel, $f_y = 415 \text{ N/mm}^2$

Step 2: Calculate nominal shear stress (τ_v)

The nominal shear stress is calculated using the formula:

$$\tau_v = \frac{V_u}{b.d} = \frac{350 \times 10^3}{350 \times 500} = 2.0 \text{ N/mm}^2$$

For M20 concrete, the maximum permissible shear stress ($\tau_{c, \max}$) is 2.8 N/mm^2 . Since $\tau_v (2.0) < \tau_{c, \max} (2.8)$, shear reinforcement can be designed.

Step 3: Determine shear strength of concrete (V_c)

First, calculate the percentage of tension steel (p_t):

$$p_t = \frac{100.A_s}{b.d} = \frac{100 \times 1963.5}{350 \times 500} = 1.122\%$$

From IS 456:2000 Table 19, the design shear strength of concrete (τ_c) for M20 grade concrete at $p_t = 1.122\%$ needs to be interpolated.

- For $p_t = 1.0\%$, $\tau_c = 0.62 \text{ N/mm}^2$
- For $p_t = 1.25\%$, $\tau_c = 0.68 \text{ N/mm}^2$

By linear interpolation:

$$\tau_c = 0.62 + \frac{1.122 - 1.0}{1.25 - 1.0} \times (0.68 - 0.62) = 0.62 + \frac{0.122}{0.25} \times 0.06 = 0.649 \text{ N/mm}^2$$

Now, calculate the shear force resisted by the concrete (V_c):

$$V_c = \tau_c \cdot b \cdot d = 0.649 \times 350 \times 500 = 113,575 \text{ N} \approx 113.58 \text{ kN}$$

Step 4: Calculate shear force for stirrups (V_{us})

The shear force to be resisted by the stirrups is:

$$V_{us} = V_u - V_c = 350 - 113.58 = 236.42 \text{ kN} = 236.42 \times 10^3 \text{ N}$$

Step 5: Design shear reinforcement

Assume 2-legged vertical stirrups of 10 mm diameter. The area of stirrup reinforcement (A_{sv}) is:

$$A_{sv} = 2 \times \frac{\pi}{4} \times (10)^2 = 157.08 \text{ mm}^2$$

The spacing of the stirrups (s_v) is calculated using the formula:

$$V_{us} = \frac{0.87.f_y.A_{sv}.d}{s_v}$$

$$s_v = \frac{0.87.f_y.A_{sv}.d}{V_{us}} = \frac{0.87 \times 250 \times 157.08 \times 500}{236.42 \times 10^3} = 72.3 \text{ mm}$$

Step 6: Check spacing against code requirements

As per IS 456:2000, the maximum spacing of vertical stirrups should not exceed:

- $0.75d = 0.75 \times 500 = 375 \text{ mm}$
- 300 mm
- The minimum shear reinforcement requirement: $A_{sv}/(b.s_v) \geq 0.4/(0.87.f_y)$. This is checked to ensure minimum spacing is 72.3 mm , which is less than both 375 mm and 300 mm .

The calculated spacing of 72.3 mm is practical. We can round it down to a standard, convenient value.

55.

Sol.

Step 1: Calculate the area of steel (A_{st}) and material properties

The area of steel for 4 bars of 25 mm diameter is calculated.

$$A_{st} = 4 \times \frac{\pi}{4} \times (25)^2$$

$$A_{st} = 4 \times 490.87 = 1963.49 \text{ mm}^2$$

The material properties for M20 concrete and Fe415 steel are:

- Characteristic strength of concrete, $f_{ck} = 20 \text{ N/mm}^2$
- Characteristic strength of steel, $f_y = 415 \text{ N/mm}^2$

Step 2: Determine the position of the neutral axis (x_u)

To find the depth of the neutral axis (x_u), we equate the total compressive force and the total tensile force.

Assuming the neutral axis lies within the flange ($x_u \leq D_f$), the T-beam can be treated as a rectangular beam with a width equal to the flange width (b_f).

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$0.36 \times 20 \times 1200 \times x_u = 0.87 \times 415 \times 1963.49$$



$$8640x_u = 708918.065$$

$$x_u = \frac{708918.065}{8640} = 82.05 \text{ mm}$$

The assumed flange depth is 100 mm. Since $x_u = 82.05 \text{ mm} < D_f = 100 \text{ mm}$, the neutral axis lies within the flange.

Step 3: Calculate the limiting depth of the neutral axis ($x_{u,lim}$)

For Fe415 steel, the limiting depth of the neutral axis is given by:

$$x_{u,lim} = 0.48d$$

$$x_{u,lim} = 0.48d$$

$$x_{u,lim} = 0.48 \times 500 = 240 \text{ mm}$$

Since $x_u = 82.04 \text{ mm} < x_{u,lim} = 240 \text{ mm}$, the beam is under reinforced.

Step 4: Calculate the moment of resistance (M_u)

The moment of resistance is calculated using the tensile force:

$$M_u = 0.87 f_y A_{st} (d - 0.42x_u)$$

$$M_u = 0.87 \times 415 \times 1963.49 \times (500 - 0.42 \times 82.04)$$

$$M_u = 708918.065 \times (500 - 34.46)$$

$$M_u = 708918.065 \times 465.54 = 330029713.7 \text{ N-mm}$$

Converting to kN-m:

$$M_u = \frac{330029713.7}{10^6} = 330.29 \text{ kNm}$$

56.

Sol.

Step 1: Calculate the area of steel and material properties

The given values are:

- Effective flange width, $b_f = 1800 \text{ mm}$
- Thickness of slab, $D_f = 100 \text{ mm}$
- Width of rib, $b_w = 230 \text{ mm}$
- Effective depth, $d = 500 \text{ mm}$
- Number of reinforcing bars = 4
- Diameter of bars = 25 mm
- Concrete grade = M20, so $f_{ck} = 20 \text{ N/mm}^2$
- Steel grade = Fe415, so $f_y = 415 \text{ N/mm}^2$

First calculate the area of steel (A_{st}):

$$A_{st} = 4 \times \frac{\pi}{4} \times (25)^2 = 1963.5 \text{ mm}^2$$

Step 2: Determine the location of the neutral axis

To check if the neutral axis lies within the flange or the web, assume the depth of the neutral axis (x_u) is less than or equal to the flange thickness (D_f). The compressive force (C) and tensile force (T) are equated:

$$C = 0.36.f_{ck}.b_f.x_u$$

$$T = 0.87.f_y.A_{st}$$

Equating the forces, $C = T$

$$0.36.f_{ck}.b_f.x_u = 0.87.f_y.A_{st}$$

$$0.36.20.1800.x_u = 0.87.415.1963.5$$

$$12960.x_u = 708921.675$$

$$x_u = \frac{708921.675}{12960} = 54.7 \text{ mm}$$

Since $x_u = 54.7 \text{ mm} < D_f = 100 \text{ mm}$, the neutral axis lies within the flange. The beam can be treated as a rectangular section with a width equal to the flange width, $b_f = 1800 \text{ mm}$.

Step 3: Calculate the limiting depth of the neutral axis

For Fe415 steel, the limiting depth of the neutral axis ($x_{u,lim}$) is given by:

$$x_{u,lim} = 0.48.d = 0.48.500 = 240 \text{ mm}$$

Since $x_u = 54.7 \text{ mm} < x_{u,lim} = 240 \text{ mm}$, the section is under-reinforced.

Step 4: Calculate the moment of resistance

The factored moment of resistance (M_u) can be calculated using the following formula:

$$M_u = 0.87.f_y.A_{st} \cdot (d - 0.42.x_u)$$

Substitute the values:

$$M_u = 0.87 \times 415 \times 1963.5 \times (500 - 0.42.54.7)$$

$$M_u = 708891.9 \times (500 - 22.974)$$

$$M_u = 708891.9 \times 477.026$$

$$M_u = 338174071 \text{ N-mm}$$

57.

Sol.

(a) $x_u = ?$, (b) $moR = ?$ (c) $w_u = ?$ S.S.B lift = 4.5 m

(i) Assuming N.A lies within flange portion : ($x_u \leq D_f$)

$$C = T$$

$$0.36 \times f_{ck} \times B_f \times x_u = 0.87 \times f_y \times A_{st}$$

$$0.36 \times 20 \times 1800 \times x_u = 0.87 \times 415 \times 7 \times \frac{\pi}{4} \times (28)^2$$

$$x_u = 120.07 \text{ mm} \Rightarrow \text{Our Assumption is wrong.}$$

(ii) Assuming N.A lies within web portion : ($x_u > D_f$) + $\left(\frac{3}{7} \times x_u \geq D_f\right)$

$$C_1 + C_2 = T_1 + T_2$$

$$0.36 \times f_{ck} \times B_w \times x_u + 0.45 \times f_{ck} \times (B_f - b_w) \times D_f = 0.87 \times f_y \times A_{st}$$

$$0.36 \times 20 \times 230 \times x_u + 0.45 \times 20 \times (1800 - 230) \times 100 = 0.87 \times 415 \times 7 \times \frac{\pi}{4} \times (28)^2$$

$$x_u = 86.48 \text{ mm} \Rightarrow \text{Wrong assumption}$$

By putting $y_f = (0.15x_u + 0.65 \times D_f)$ in place of D_f in above eq.

$$y_f = 0.15 \times x_u + 0.65 \times 100 = (65 + 0.15 \times x_u)$$

$$0.36 \times 20 \times 230 \times x_u + 0.45 \times 20 \times (1800 - 230) \times (65 + 0.15 \times x_u)$$

$$= 0.87 \times 415 \times 7 \times \frac{\pi}{4} \times (28)^2$$

$$1656x_u + 2119.5 \times x_u = 637771.22$$

$$x_u = 168.92 \text{ mm}$$

$$\frac{3}{7} \times x_u < D_f \Rightarrow \frac{3}{7} \times 168.92 < 100$$

$$72.39 < 100$$

$$x_{u,lim} = 0.48 \times d = 0.48 \times 500 = 240 \text{ mm}$$

So, $x_u < x_{u,lim} \Rightarrow$ Under Reinforcement Section

$$(MOR)_c = 0.36 \times f_{ck} \times b_w \times x_u \times (d - 0.42 \times x_u) + 0.45 \times f_{ck} \times (B_f - b_w) \times y_f \times \left(d - \frac{y_f}{2}\right)$$

$$= 0.36 \times 20 \times 230 \times 168.92 \times (500 - 0.42 \times 168.92) +$$



$$0.45 \times 20 \times (1800 - 230) \times (0.15 \times 168.92 + 0.65 \times 100) \times \left(500 - \frac{90.338}{2} \right)$$

$$= 700600644 \text{ N-mm} = 700.6 \text{ kN-m}$$

58.

Sol.

Step 1: Preliminary calculations and slab thickness

First, we determine the type of slab and a suitable thickness. The ratio of the longer span (L_y) to the shorter span (L_x) is $5.3/4.2 = 1.26$, which is less than 2. Therefore, this is a two-way slab. For a two-way slab with all edges continuous, a preliminary effective depth (d) can be estimated. We will assume a total depth of 170 mm and an effective cover of 25 mm.

- **Effective depth, d :** $170 - 25 = 145 \text{ mm}$
- **Effective shorter span, L_{ex} :** $L_x + d = 4200 + 145 = 4345 \text{ mm}$
- **Effective longer span, L_{ey} :** $L_y + d = 5300 + 145 = 5445 \text{ mm}$

Step 2: Load calculation

Calculate the dead load (DL) and factored load (W_u)

- **Slab self-weight:** $0.170 \text{ m} \times 1 \text{ m} \times 25 \text{ kN/m}^3 = 4.25 \text{ kN/m}^2$
- **Dead Load (DL):** Slab self-weight + Floor finish (FF) $= 4.25 + 0.8 = 5.05 \text{ kN/m}^2$
- **Factored Load (W_u):** $1.5 \times (\text{DL} + \text{LL}) = 1.5 \times (5.05 + 2) = 10.575 \text{ kN/m}^2$

Step 3: Bending Moment and shear force calculation

The bending moments are calculated using coefficient from IS 456:2000, Table 26, for a panel with four continuous edges and $L_y/L_x = 1.25$. The coefficient are interpolated.

- **Bending moment coefficient, α_x (negative):** 0.065
- **Bending moment coefficient, α_x (positive):** 0.049
- **Bending moment coefficient, α_y (negative):** 0.052
- **Bending moment coefficient, α_y (positive):** 0.037

The bending moments are:

- **Negative Bending moment (mid span of support) in x-direction, M_{ux} :**
 $\alpha_x \cdot W_u \cdot L_x^2 = 0.065 \times 10.575 \times 4.2^2 = 12.13 \text{ kNm}$
- **Positive Bending moment (mid span) in x-direction, M_{ux} :**
 $\alpha_x \cdot W_u \cdot L_x^2 = 0.049 \times 10.575 \times 4.2^2 = 9.14 \text{ kNm}$
- **Negative Bending moment (mid span of support) in y-direction, M_{uy} :**
 $\alpha_y \cdot W_u \cdot L_x^2 = 0.052 \times 10.575 \times 4.2^2 = 9.70 \text{ kNm}$
- **Positive Bending moment (mid span) in y-direction, M_{uy} :**
 $\alpha_y \cdot W_u \cdot L_x^2 = 0.037 \times 10.575 \times 4.2^2 = 6.90 \text{ kNm}$

The maximum shear force at the edge of the shorter span is assumed to be half the total load on the span;

- **Maximum shear force, $V_u = W_u \cdot L_x / 2 = 10.575 \times 4.2 / 2 = 22.21 \text{ kN}$**

Step 4: Reinforcement calculation

The required area of steel (A_{st}) is calculated for the positive and negative moments in both directions using the equation for singly reinforced sections. The minimum required steel area is $0.12\% \times b \times D = 0.0012 \times 1000 \times 170 = 204 \text{ mm}^2/\text{m}$. We will use 10 mm diameter bars ($A_{bar} = 78.54 \text{ mm}^2$)

- **Negative Bending moment (Mid-span-of support) x-direction:**

$$Bm_u = Q \times B \times d_{requ.}^2$$

$$= 12.13 \times 10^6 = 0.133 \times F_{ck} \times 1000 \times d_{requ.}^2$$

$$= 12.13 \times 10^6 = 0.133 \times 25 \times 1000 \times d_{requ.}^2$$

$$= d_{requ.} = 60.39 \text{ mm}$$

So, $0.36 \times F_{ck} \times B \times x_{ulimit} = 0.87 f_y \times A_{st}$

$$0.36 \times 25 \times 1000 \times 0.46 \times 60.39 = 0.87 \times 500 \times A_{st}$$

$$A_{st} = 574.74 \text{ mm}^2$$

Assuming 10 mm dia. of bar so no. of bar $n = \frac{A_{st}}{\frac{\pi}{4} \times (10)^2} = \frac{574.74}{\frac{\pi}{4} \times (10)^2} = 7.31$ or 8

$A_{st} \text{ Provided} = 8 \times \frac{\pi}{4} \times (10)^2 = 628.32 \text{ mm}^2$

- **Positive Bending moment (Mid-span-of support) x-direction:**

$$\begin{aligned} Bm_u &= Q \times B \times d_{\text{requ.}}^2 \\ &= 9.14 \times 10^6 = 0.133 \times F_{ck} \times 1000 \times d_{\text{requ.}}^2 \\ &= 9.14 \times 10^6 = 0.133 \times 25 \times 1000 \times d_{\text{requ.}}^2 \\ &= d_{\text{requ.}} = 52.42 \text{ mm} \end{aligned}$$

So, $0.36 \times F_{ck} \times B \times x_{\text{ulimit}} = 0.87 f_y \times A_{st}$
 $0.36 \times 25 \times 1000 \times 0.46 \times 52.42 = 0.87 \times 500 \times A_{st}$
 $A_{st} = 498.89 \text{ mm}^2$

Assuming 10 mm dia. of bar so no. of bar $n = \frac{A_{st}}{\frac{\pi}{4} \times (10)^2} = \frac{498.89}{\frac{\pi}{4} \times (10)^2} = 6.35$ or 7

$A_{st} \text{ Provided} = 7 \times \frac{\pi}{4} \times (10)^2 = 549.77 \text{ mm}^2$

- **Negative Bending moment (Mid-span-of support) y-direction:**

$$\begin{aligned} Bm_u &= Q \times B \times d_{\text{requ.}}^2 \\ &= 9.70 \times 10^6 = 0.133 \times F_{ck} \times 1000 \times d_{\text{requ.}}^2 \\ &= 9.70 \times 10^6 = 0.133 \times 25 \times 1000 \times d_{\text{requ.}}^2 \\ &= d_{\text{requ.}} = 54.011 \text{ mm} \end{aligned}$$

So, $0.36 \times F_{ck} \times B \times x_{\text{ulimit}} = 0.87 f_y \times A_{st}$
 $0.36 \times 25 \times 1000 \times 0.46 \times 54.011 = 0.87 \times 500 \times A_{st}$
 $A_{st} = 514.035 \text{ mm}^2$

Assuming 10 mm dia. of bar so no. of bar $n = \frac{A_{st}}{\frac{\pi}{4} \times (10)^2} = \frac{514.035}{\frac{\pi}{4} \times (10)^2} = 6.54$ or 7

$A_{st} \text{ Provided} = 7 \times \frac{\pi}{4} \times (10)^2 = 549.77 \text{ mm}^2$

- **Positive Bending moment (Mid-span-of support) y-direction:**

$$\begin{aligned} Bm_u &= Q \times B \times d_{\text{requ.}}^2 \\ &= 6.90 \times 10^6 = 0.133 \times F_{ck} \times 1000 \times d_{\text{requ.}}^2 \\ &= 6.90 \times 10^6 = 0.133 \times 25 \times 1000 \times d_{\text{requ.}}^2 \\ &= d_{\text{requ.}} = 45.55 \text{ mm} \end{aligned}$$

So, $0.36 \times F_{ck} \times B \times x_{\text{ulimit}} = 0.87 f_y \times A_{st}$
 $0.36 \times 25 \times 1000 \times 0.46 \times 45.55 = 0.87 \times 500 \times A_{st}$
 $A_{st} = 433.51 \text{ mm}^2$

Assuming 10 mm dia. of bar so no. of bar $n = \frac{A_{st}}{\frac{\pi}{4} \times (10)^2} = \frac{433.51}{\frac{\pi}{4} \times (10)^2} = 5.51$ or 6

$$A_{st} \text{ Provided} = 6 \times \frac{\pi}{4} \times (10)^2 = 471.23 \text{ mm}^2$$

All provided steel areas are greater than the minimum required A_{st} .

59.

Sol.

1. Design assumptions

- **Material:** M20 concrete ($f_{ck} = 20 \text{ N/mm}^2$) and Fe 415 steel ($f_y = 415 \text{ N/mm}^2$)
- **Span:** Clear span ($L_{clear} = 3 \text{ m}$)
- **Support width:** 300 mm
- **Live load:** $LL = 3 \text{ kN/m}^2$.
- **Floor finish:** Assume floor finish load (FF) = 1 kN/m^2

2. Calculations

Step 1: Calculate loads

- **Assume slab thickness:** Let's assume a slab thickness of 120 mm ($D = 120 \text{ mm}$).
- **Self-weight of slab:** $SW = \text{thickness} \times \text{density} = 0.12 \text{ m} \times 25 \text{ kN/m}^3 = 3 \text{ kN/m}^2$.
- **Total load:** $w = SW + FF + LL = 3 + 1 + 3 = 7 \text{ kN/m}^2$
- **Factored load:** $w_u = 1.5 \times w = 1.5 \times 7 = 10.5 \text{ kN/m}^2$ (using partial safety factor)

Step 2: Determine effective span

- Effective span (L_{eff}):

$$L_{eff} = L_{clear} + \text{average of support widths} = 3 + \frac{0.3}{2} + \frac{0.3}{2} = 3.3 \text{ m}$$

Step 3: Calculate maximum bending moment

- Maximum bending moment (M_u): For a simply supported one-way slab,

$$M_u = \frac{W_u L_{eff}^2}{8}$$

$$M_u = \frac{10.5 \times (3.3)^2}{8} = 14.29 \text{ kNm}$$

Step 4: Determine effective depth and moment of resistance

- Effective depth (d): $d = D - \left(\text{cover} + \frac{\phi}{2} \right)$, assuming $\phi = 10 \text{ mm}$ bar,
 $d = 120 - (20 + 5) = 95 \text{ mm}$
- Moment of resistance ($M_{u,R}$): For M20 and Fe 415, let's find the moment of resistance to ensure it is greater than the applied moment. From standard design tables or calculations, $M_{u,R} \approx 0.138 f_{ck} b d^2$
- $M_{u,R} = 0.138 \times 20 \times (1000) \times (95)^2 = 24.909 \text{ kNm}$
- Since $M_u < M_{u,R}$, the assumed section is adequate, and the slab can be designed as an under-reinforced section.

Step 5: Calculate the area of steel reinforcement (A_{st})

- **Formula:** $A_{st} = \frac{0.5 f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}} \right] b d$

$$A_{st} = \frac{0.5 \times 20}{415} \left[1 - \sqrt{1 - \frac{4.6 \times 14.29 \times 10^6}{20 \times 1000 \times 95^2}} \right] \times 1000 \times 95$$

$$A_{st} = 466 \text{ mm}^2$$

60.

Sol. **Given data**

- **Building type:** Public building
- **Ceiling height:** 3.5 m

- **Width of each flight:** 1.5 m
- **Concrete grade:** M20 (characteristic compressive strength, $f_{ck} = 20 \text{ N/mm}^2$)
- **Steel grade:** Fe 415 (characteristic yield strength, $f_y = 415 \text{ N/mm}^2$)

Step 1: Assume riser and tread dimensions

For public buildings, standard dimensions for steps are recommended by the code for safe and comfortable use.

- **Riser (R):** Assume a vertical height of 150 mm.
- **Tread (T):** Assume a horizontal depth of 250 mm.
- **Note:** The relationship between riser and tread should be comfortable. A good rule of thumb is $R + 2T = 500$ to 650 mm.
- Let's reconsider. An alternative is $R = 150 \text{ mm}$ and $T = 250 \text{ mm}$.
- $150 + 2(250) = 650 \text{ mm}$ (This is an excellent proportion).
- So, we will proceed with $R = 150 \text{ mm}$ and $T = 250 \text{ mm}$.

Step 2: Determine geometry of the staircase

The total vertical height is the ceiling height plus the slab thickness. Assuming a waist slab thickness of 200 mm for now.

- **Floor to floor height:** $3.5 \text{ m} + 0.2 \text{ m} = 3.7 \text{ m}$ or 3700 mm.
- **Height of each flight:** For a dog-legged staircase, the total height is divided into two flights separated by a landing.
Height of each flight $= 3700/2 = 1850 \text{ mm}$.
- **Number of risers:** Number of risers in each flight $= \text{Height of each flight} / \text{Riser height}$
Number of risers $= 1850/150 = 12.33$.

Since the number of risers must be a whole number, let's adjust the riser height.

Try 12 risers: $R = 1850/12 = 154.17 \text{ mm}$. This is within the recommended range of 150-200 mm.

Number of risers per flight $= 12$.

Number of treads: Number of treads $= \text{Number of risers} - 1$.

Number of treads per flight $= 12 - 1 = 11$.

Going (horizontal length): Total going per flight $= \text{Number of treads} \times \text{Tread depth}$

Total going $= 11 \times 250 = 2750 \text{ mm} = 2.75 \text{ m}$.

Landing width: The landing width should not be less than the flight width.

Landing width $= \text{Flight width} = 1.5 \text{ m}$.

Stair hall dimensions: The required length of the stair hall is the sum of the going and the landing width.

Stair hall length $= 2.75 \text{ m} + 1.5 \text{ m} = 4.25 \text{ m}$.

Stair hall width $= \text{Flight width} + \text{Landing width} = 1.5 \text{ m} + 1.5 \text{ m} = 3.0 \text{ m}$.

Step 3: Calculate effective span and waist slab thickness

The staircase is assumed to span longitudinally between the beams at the ends of the landings. The effective span is the horizontal distance between the centerlines of the supporting beams or walls.

- **Effective span (L_{eff}):** Assume supporting walls are 300 mm thick.
 $L_{eff} = 2.75 \text{ m (going)} + 1.5 \text{ m (landing)} + (0.3 \text{ m}/2) + (0.3 \text{ m}/2) = 4.55 \text{ m}$.
- **Thickness of waist slab:** The overall depth (D) of the slab can be estimated using the deflection criteria as per IS 456. For a simply supported slab with Fe 415, the basic (span/depth) ratio is 20, and the modification factor can be taken as 1.3 to 1.4 for a staircase.
 $D_{req} = L_{eff} / (20 \times 1.3) = 4550 / (20 \times 1.3) = 175 \text{ mm}$
Let's provide an overall thickness $D = 200 \text{ mm}$.
- **Effective depth (d):** Assuming a clear cover of 25 mm and a 12 mm main reinforcement bar.
 $d = D - \text{clear cover} - (\text{bar diameter}/2) = 200 - 25 - (12/2) = 169 \text{ mm}$.
Use $d = 169 \text{ mm}$.

Step 4: Calculate design loads

- **Loads on the going (inclined part):**
 - **Dead load of waist slab (DL_{waist}):** The weight on a horizontal area is

$$D \times \left(\sqrt{R^2 + T^2} / T \right) \times \rho_{\text{concrete}}$$



$$DL_{\text{waist}} = 0.200 \times \left(\sqrt{0.15417^2 + 0.25^2} / 0.25 \right) \times 25 \text{ kN/m}^3 = 5.87 \text{ kN/m}^2$$

- **Dead load of steps (DL_{steps}):** $0.5 \times R \times \rho_{\text{concrete}}$
 $DL_{\text{steps}} = 0.5 \times 0.15417 \times 25 \text{ kN/m}^3 = 1.93 \text{ kN/m}^2$
- **Assumed floor finish:** 1 kN/m^2
- **Live load(public building):** 5 kN/m^2 as per IS 875.
- **Total load on going (W_g):** $5.87 + 1.93 + 1 + 5 = 13.8 \text{ kN/m}^2$
- **Loads on the landing:**
 - **Dead load of slab (DL_{slab}):** $D \times \rho_{\text{concrete}}$
 $DL_{\text{slab}} = 0.200 \times 25 \text{ kN/m}^3 = 5.0 \text{ kN/m}^2$
 - **Assumed floor finish:** 1 kN/m^2
 - **Live load:** 5 kN/m^2
 - **Total load on landing (w_l):** $5 + 1 + 5 = 11.0 \text{ kN/m}^2$
- **Factored loads:** The higher load (from the going) will govern the design.
 Factored ultimate load (W_u) = $1.5 \times 13.80 = 20.7 \text{ kN/m}^2$

Step 5: Calculate maximum bending moment

The staircase can be analyzed as a simply supported slab of width 1 m.

- **Maximum bending moment (M_u):** $W_u \times L_{\text{eff}}^2 / 8$

$$M_u = (20.7 \times 4.55^2) / 8 = 53.56 \text{ kNm}$$

Step 6: Design main reinforcement (A_{st})

Using the Limit State Method for a singly reinforced beam (since $M_u < M_{u,\text{lim}}$ for M20 and Fe 415), the required area of steel (A_{st}) can be calculated.

- **Maximum moment of resistance ($M_{u,\text{lim}}$):** $0.138 \times f_{ck} \times b \times d^2$

$$M_{u,\text{lim}} = 0.138 \times 20 \times 1000 \times 169^2 \times 10^{-6} = 78.8 \text{ kNm}$$

Since $53.56 < 78.8$, the section is under reinforced.

- **Area of steel (A_{st}):** From IS 456, Table G for
 $M_u / bd^2 = (53.56 \times 10^6) / (1000 \times 169^2) = 1.95 \text{ N/mm}^2$
 Using the formula

$$A_{st} = (0.5 \times f_{ck} / f_y) \times \left[1 - \sqrt{1 - (4.6 \times M_u / (f_{ck} \times b \times d^2))} \right] \times b \times d$$

$$A_{st} = 1001.32 \text{ mm}^2$$

- **Bar selection:** Using 12 mm diameter bars. Area of one bar = $\pi \times 12^2 / 4 = 113 \text{ mm}^2$
 Spacing = $(1000 \times 113) / 1001.32 = 112 \text{ mm}$
 Provide 12 mm bars at 120 mm center-to-center.

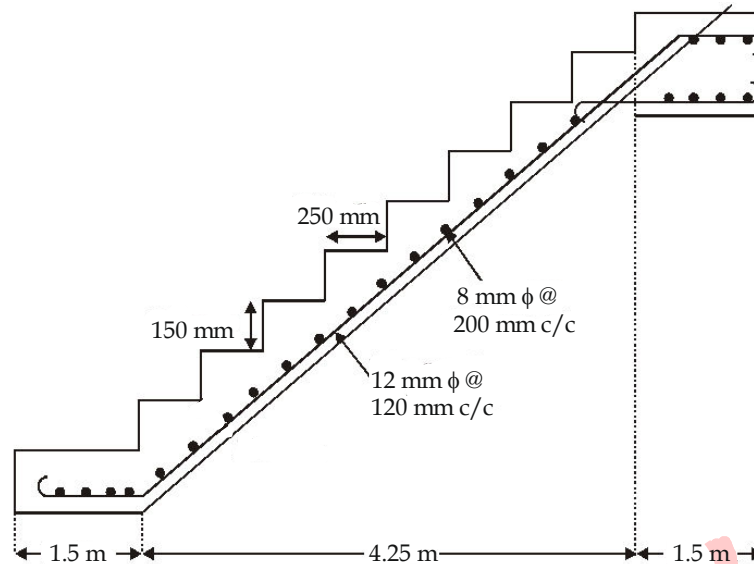
Step 7: Provide distribution reinforcement

As per IS 456, the minimum area of steel for Fe 415 is 0.12% of the total cross-sectional area.

- **Area of distribution steel ($A_{st,\text{dist}}$):** $0.12\% \times b \times D$
 $A_{st,\text{dist}} = 0.0012 \times 1000 \times 200 = 240 \text{ mm}^2$
- **Bar selection:** Using 8 mm diameter bars. Area of one bar = $\pi \times 8^2 / 4 = 50.27 \text{ mm}^2$
 Spacing = $(1000 \times 50.27) / 240 = 209.4 \text{ mm}$
 Provide 8 mm bars at 200 mm center-to-center

Step 8: Check for shear and deflection

- **Shear check**
 - **Maximum shear force (V_u):** $W_u \times L_{\text{eff}} / 2 = 21.56 \times 4.55 / 2 = 49.06 \text{ kN}$
 - **Nominal shear stress (τ_v):**
 $V_u / (b \times d) = (49.06 \times 10^3) / (1000 \times 169) = 0.29 \text{ N/mm}^2$
 - **Percentage of steel:** $100 \times A_{st} / (b \times d) = 100 \times 918 / (1000 \times 169) = 0.54\%$.
 - From IS 456, Table 19, for M20 concrete and 0.54% steel, the permissible shear strength (τ_c) is greater than 0.29 N/mm^2 , so the design is safe in shear.



61.

Sol.

1. Load calculations

- **Live load (LL):** 3 kN/m²
- **Floor finish Load (FF):** Assume a conservative value of 1 kN/m² for floor finish and other dead loads.
- **Total Dead Load (DL):** 1 kN/m²
- **Factored Load (W_u):**
 $W_u = 1.5 \times (DL + LL)$
 $W_u = 1.5 \times (1 + 3) = 6 \text{ kN/m}^2$
- **Ultimate Load per meter run for a 1 m width:**
 $W_u = 6 \text{ kN/m}^2 \times 1 \text{ m} = 6 \text{ kN/m}$

2. Design for bending moment and shear**Trail depth calculation**

- For a cantilever, the maximum span is typically limited to prevent excessive deflection, but here we will assume a trial depth based on a span-to-depth ratio.
- **Span (L):** 1.25 m
- **Trial depth (d):** A reasonable depth to start with is 200 mm.
 Check the span/depth ratio: $L/d = 1.25 \text{ m} / 0.200 \text{ m} = 6.25$
 According to IS 456, for a cantilever, the span/depth ratio should be less than 7 for deflection checks. Therefore, a thicker slab is required.
- **Deflection check (d):** A more precise method is needed to check deflection properly we will use the span-to-depth ratio to check deflection.
- **Effective depth (d):** = 200 mm

Calculate the Bending moment

- Maximum Bending moment (M_u) at the support:
 $M_u = (W_u \times L^2) / 2$
 $M_u = (6 \times (1.25)^2) / 2 = 4.6875 \text{ kN/m}$

Shear force calculation

- Maximum shear force (V_u) at the support:
 $V_u = W_u \times L = 6 \times 1.25 = 7.5 \text{ kN/m}$
- Nominal shear stress (τ_v) for 1 m width:
 $\tau_v = V_u / (b \times d)$
 $\tau_v = 7.5 \times 10^3 / (1000 \times 200) = 0.0375 \text{ N/mm}^2$
 $\tau_{\max} = 0.625 \times \sqrt{f_{ck}} = 0.625 \times \sqrt{20} = 2.79$

So slab is safe-in-shear.

62.

Sol. Given data

- **Service load (P):** 1600 kN
- **Concrete grade:** M20 ($f_{ck} = 20 \text{ N/mm}^2$)
- **Steel grade:** Fe500 ($f_y = 500 \text{ N/mm}^2$)

Step 1: Calculate the factored load

The service load is multiplied by a partial safety factor of 1.5 to find the factored load (P_u).

$$P_u = 1.5 \times P$$

$$P_u = 1.5 \times 1600 = 2400 \text{ kN}$$

$$P_u = 2400 \times 10^3 \text{ N}$$

Step 2: Determine the column dimensions

The design of a helically reinforced column must account for a 5% increase in load-carrying capacity compared to a similar column with lateral ties.

The formula for the load-carrying capacity of a column with helical ties is:

$$P_u = 1.05 \times [0.4 \times f_{ck} \times A_c + 0.67 \times f_y \times A_{sc}]$$

where:

- A_g = Gross area of the column
- A_{sc} = Area of longitudinal steel reinforcement
- A_c = Area of concrete ($A_c = A_g - A_{sc}$)

Substituting A_c we get:

$$P_u = 1.05 \times [0.4 \times f_{ck} \times (A_g - A_{sc}) + 0.67 \times f_y \times A_{sc}]$$

Assume 1% steel reinforcement, which is within the 0.8% to 6% limits specified by IS 456:2000.

$$A_{sc} = 0.01 \times A_g$$

Substitute this assumption into the load capacity formula:

$$2400 \times 10^3 = 1.05 \times [0.4 \times 20 \times (A_g - 0.01 \times A_g) + 0.67 \times 500 \times (0.01 \times A_g)]$$

$$2400 \times 10^3 = 1.05 \times [8 \times (0.99 \times A_g) + 3.35 \times A_g]$$

$$2400 \times 10^3 = 1.05 \times [7.92 \times A_g + 3.35 \times A_g]$$

$$2400 \times 10^3 = 1.05 \times [11.27 \times A_g]$$

$$A_g = \frac{2400 \times 10^3}{1.05 \times 11.27} = 202814.045 \text{ mm}^2$$

Now, find the required diameter (D) of the column using the gross area:

$$A_g = \frac{\pi}{4} \times D^2$$

$$D = \sqrt{\frac{4 \times A_g}{\pi}} = \sqrt{\frac{4 \times 202814.045}{\pi}} = 508.16 \text{ mm}$$

Provide a column diameter of 550 mm for a safe design.

$$\text{Gross area provided, } A_g = \frac{\pi}{4} \times 550^2 = 237583 \text{ mm}^2$$

Step 3: Design the longitudinal reinforcement

The area of steel provided, A_{sc} , is calculated by rearranging the load capacity formula using the new gross area.

$$P_u = 1.05 \times [0.4 \times f_{ck} \times (A_g - A_{sc}) + 0.67 \times f_y \times A_{sc}]$$

$$2400 \times 10^3 = 1.05 \times [0.4 \times 20 \times (237583 - A_{sc}) + 0.67 \times 500 \times A_{sc}]$$

$$2400 \times 10^3 = 1.05 \times [8 \times 237583 - 8 \times A_{sc} + 335 \times A_{sc}]$$

$$\frac{2400 \times 10^3}{1.05} = 1900664 - 8 \times A_{sc} + 335 \times A_{sc}$$

$$2285714 = 1900664 + 327 \times A_{sc}$$

$$327 \times A_{sc} = 2285714 - 1900664 = 385050$$

$$A_{sc} = 1177.52 \text{ mm}^2$$

Minimum and maximum steel area check (IS 456:2000):

Minimum steel: 0.8% of $A_g = 0.008 \times 237583 = 1900.66 \text{ mm}^2$

Maximum steel: 6% of $A_g = 0.06 \times 237583 = 14254.98 \text{ mm}^2$

The calculated A_{sc} is less than the minimum. Therefore, provide the minimum area of steel

$$A_{sc, \text{prov}} = 1900.66 \text{ mm}^2$$

Provide 6 bars of 20 mm diameter.

$$\text{Area of one 20 mm bar} = \frac{\pi}{4} \times 20^2 = 314.16 \text{ mm}^2$$

$$A_{sc, \text{prov}} = 6 \times 314.16 = 1884.96 \text{ mm}^2$$

This is slightly less than the minimum area, so we should provide a greater number or diameter of bars.

Provide 8 bars of 20 mm diameter.

$$A_{sc, \text{prov}} = 8 \times 314.16 = 2513.28 \text{ mm}^2$$

Step 4: Design the helical reinforcement

According to IS 456:2000 (Clause 26.5.3.2), the diameter of helical reinforcement should not be less than either 6 mm or one-fourth of the largest longitudinal bar diameter.

- $\frac{1}{4} \times 20 \text{ mm} = 5 \text{ mm}$
- 6 mm

The larger value is 6 mm. Provide 8 mm diameter helical reinforcement.

Core diameter (D_c):

Assume a clear cover of 40 mm, as per IS 456 : 2000 $\therefore$ Assume $D = 550 \text{ mm}$

$$D_c = D - 2 \times \text{clear cover}$$

$$D_c = 550 - 2 \times 40 = 470 \text{ mm}$$

Pitch (p) of the helical reinforcement:

The pitch must satisfy IS 456:2000, Clause 39.4.1.

$$\frac{V_{sh}}{V_c} \geq 0.36 \times \left(\frac{A_g}{A_c} - 1 \right) \times \frac{f_{ck}}{f_y}$$

where:

$$V_{sh} = A_{sp} \times \pi \times (D_c - \phi_{sh})$$

$$A_{sp} = \text{area of helical bar} = \frac{\pi}{4} \times 8^2 = 50.26 \text{ mm}^2 \therefore A_c = \text{Core area}$$

$$V_{sh} = 50.26 \times \pi \times (470 - 8) = 72948.15 \text{ mm}^3$$

$$V_c = A_c \times p = \frac{\pi}{4} \times D_c^2 \times p = \frac{\pi}{4} \times 470^2 \times p = 173494.4 \times p$$

Substitute into the volume ratio equation:

$$\frac{72948.15}{173494.4 \times p} \geq 0.36 \times \left(\frac{237583}{173494.4} - 1 \right) \times \frac{20}{500}$$

$$\frac{72948.15}{173494.4 \times p} \geq 0.36 \times (1.369 - 1) \times 0.04$$

$$\frac{0.42}{p} \geq 0.36 \times 0.369 \times 0.04$$

$$\frac{0.42}{p} \geq 0.0053$$

$$p \leq \frac{0.42}{0.0053} = 79.2 \text{ mm}$$

Check maximum and minimum pitch limits (IS 456:2000, Clause 26.5.3.2):

- **Maximum pitch:** Not more than 75 mm
- **Maximum pitch:** Not more than one-sixth of the core diameter ($D_c/6 = 470/6 = 78.33$ mm)
- **Minimum pitch:** Not less than 25 mm
- **Minimum pitch:** Not less than 3 times the helical bar diameter (3×8 mm = 24 mm)

The selected pitch must be less than 75 mm and greater than 25 mm. Provide a pitch of 70 mm.

Step 5: Check for minimum eccentricity ($e_{\min}$)

As a short column, it must be designed for minimum eccentricity. Assuming no additional information, the unsupported length (l_{un}) can be assumed as 3500 mm.

$$e_{\min} = \frac{l_{un}}{500} + \frac{D}{30} \text{ or } 20 \text{ mm, whichever is greater.}$$

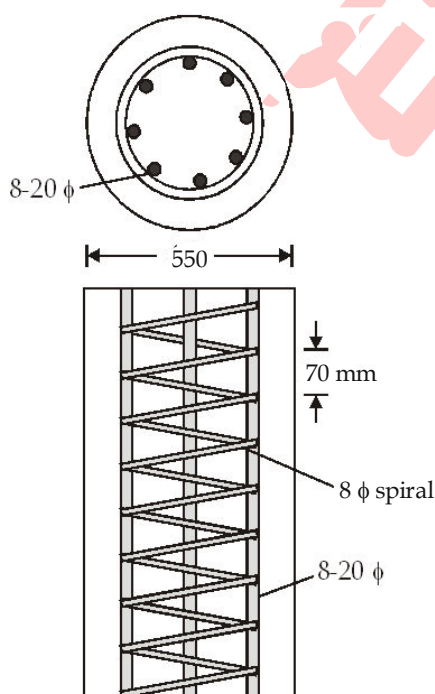
$$e_{\min} = \frac{3500}{500} + \frac{550}{30} = 7 + 18.33 = 25.33 \text{ mm}$$

The calculated minimum eccentricity is 25.33 mm, which is greater than the 20 mm minimum.

Check if $e_{\min} \leq 0.05 \times D$

$$0.05 \times D = 0.05 \times 550 = 27.5 \text{ mm}$$

Since $e_{\min}$ (25.33 mm) < $0.05 \times D$ (27.5 mm), the column can be designed as an axially loaded short column, and the 5% strength increase for helical reinforcement is valid.



63.

Sol.

Step 1: Determine the Footing dimensions

The footing dimensions are determined based on the unfactored load and the safe bearing capacity (SBC) of the soil.

- Factored load on the column, $P_u = 1.5 \times 250$ kN = 375 kN
- Unfactored load on the column, $P = 250$ kN
- Assume the self-weight of the footing is 10% of the column load.
Self-weight = 0.10×250 kN = 25 kN
- Total working load on the soil = $250 + 25 = 275$ kN

- Required footing area, $A_{\text{req}} = \frac{\text{Total working load}}{\text{SBC}} = \frac{275 \text{ kN}}{160 \text{ kN/m}^2} = 1.72 \text{ m}^2$

- For a square footing, the side length $B = \sqrt{1.72 \text{ m}^2} \approx 1.31 \text{ m}$



- Adopt a footing size of 1.4 m × 1.4 m.
- The provided area is $A_{\text{prov}} = 1.4 \times 1.4 = 1.96 \text{ m}^2$
- Net ultimate soil pressure for design,

$$\dot{P}_u = \frac{P_u}{\text{Provided Area}} = \frac{375 \text{ kN}}{1.96 \text{ m}^2} = 191.33 \text{ kN/m}^2$$

Step 2: Determine the footing depth

The depth of the footing is governed by one-way and two-way shear criteria. Let's assume a total depth $D = 450 \text{ mm}$ and an effective depth

$$d = 450 - 60 - \frac{12}{2} = 384 \text{ mm (assuming 60 mm cover and 12 mm bars).}$$

- **One way shear check:** The critical section is at a distance 'd' from the face of the column. Length of the cantilever,

$$L_c = \frac{1400 - 250}{2} - d = \frac{1150}{2} - 384 = 575 - 384 = 191 \text{ mm}$$

$$\text{Shear force, } V_u = \dot{P}_u \times L_c \times B = 191.33 \text{ kN/m}^2 \times 0.191 \text{ m} \times 1.4 \text{ m} = 51.16 \text{ kN}$$

Permissible shear stress for M25 concrete ($k_s \tau_c$) is 0.29 MPa for p_t from reinforcement calculations. This is typically satisfied. The depth is adequate for one-way shear.

- **Two-way shear (Punching shear) check:** The critical section is at a distance 'd/2' from the column face.

$$\text{Side of the critical section, } b_o = 250 + d = 250 + 384 = 634 \text{ mm}$$

$$\text{Punching shear perimeter, } P_c = 4 \times (b_o) = 4 \times 634 = 2536 \text{ mm}$$

Punching shear force,

$$V_{up} = P_u - \dot{P}_u \times (\text{Area of critical section}) = 375 - 191.33 \times (0.634)^2 = 375 - 76.9 = 298.1 \text{ kN}$$

$$\text{Nominal shear stress, } \tau_{vp} = \frac{V_{up}}{P_c \times d} = \frac{298.1 \times 10^3 \text{ N}}{2536 \text{ mm} \times 384 \text{ mm}} = 0.306 \text{ N/mm}^2$$

Permissible shear stress,

$$\tau'_c = k_s \times \tau_c = 1.0 \times 0.25 \sqrt{f_{ck}} = 1.0 \times 0.25 \sqrt{25} = 1.25 \text{ N/mm}^2$$

Since $\tau_{vp} < \tau'_c$, the depth is safe in punching shear.

Step 3: Design the reinforcement

The reinforcement is designed based on the bending moment at the face of the column.

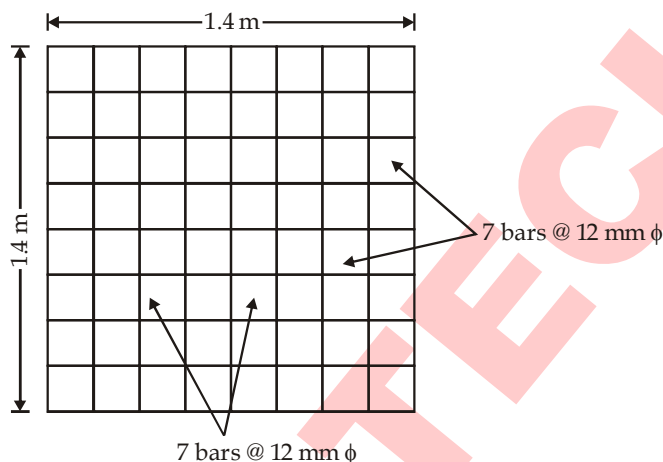
- Cantilever length from column face to edge, $L = \frac{1.4 - 0.25}{2} = 0.575 \text{ m}$
- Bending moment, $M_u = \frac{\dot{P}_u \times B \times L^2}{2} = \frac{191.33 \times 1.4 \times (0.575)^2}{2} = 44.28 \text{ kN-m}$
- To find the required area of steel (A_{st}), we use the formula for a singly reinforced section:

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{B d f_{ck}} \right)$$

- $44.28 \times 10^6 = 0.87 \times 415 \times A_{st} \times 384 \left(1 - \frac{A_{st} \times 415}{1400 \times 384 \times 25} \right)$
- This simplifies to a quadratic equation, solving it yields $A_{st} = 322.15 \text{ mm}^2$
- Minimum reinforcement,
 $A_{st, \min} = 0.12\% \times B \times D = 0.0012 \times 1400 \times 450 = 756 \text{ mm}^2$
- Since $A_{st, \min} > A_{st}$, we will provide the minimum reinforcement. Let's use 12 mm diameter bars.



- Area of one 12 mm bar = $\frac{\pi}{4} (12)^2 = 113.1 \text{ mm}^2$
- Number of bars required = $\frac{756 \text{ mm}^2}{113.1 \text{ mm}^2} \approx 6.68$, so we provide 7 bars.
- Spacing = $\frac{1400 \text{ mm}}{7 \text{ bars}} \approx 200 \text{ mm}$
- Let's check the spacing requirements from IS 456: max spacing is $3D = 3 \times 450 = 1350 \text{ mm}$ or 300 mm. The spacing is well within limits.



64.

Sol.

1. Calculations and assumptions

- Factored axial load (P_u): 1800 kN (The problem states the load is factored, so no need to multiply by a load factor).
- Column length (L): 3m = 3000 mm.
- Grade of concrete: M-20, so $f_{ck} = 20 \text{ N/mm}^2$
- Grade of steel: Fe-415, so $f_y = 415 \text{ N/mm}^2$
- Effective length (L_{eff}): For a column with both ends fixed, the effective length is 0.65 L.
 $L_{eff} = 0.65 \times 3000 \text{ mm} = 1950 \text{ mm}$

2. Check if the column is short or slender

- A column is considered short if the ratio of effective length to the smaller lateral dimension is less than 12.
- Let's assume a square column with a size of 450 mm × 450 mm.

$$\text{Ratio of effective length to lateral dimension} = \frac{L_{eff}}{D} = \frac{1950 \text{ mm}}{450 \text{ mm}} \approx 4.33.$$

- Since $4.33 < 12$, the column is a short column and can be designed using the short column formulas.

3. Determine the required area of steel reinforcement (A_{sc})

- The design capacity of the column is given by the formula:

$$P_u = 0.4 \times f_{ck} \times A_c + 0.65 \times A_{sc} \times f_y$$

$$\therefore A_c = A_g - A_{sc}$$

$$P_u = 0.4 \times f_{ck} \times (A_g - A_{sc}) + 0.65 \times A_{sc} \times f_y$$

$$1800 \times 10^3 = 0.4 \times 20 (202500 - A_{sc}) + 0.65 \times A_{sc} \times 415$$

$$= 1620000 - 8 A_{sc} + 269.75 A_{sc}$$

$$A_{sc} = 687.67 \text{ mm}^2$$

$$A_{sc \text{ max.}} = 6\% \text{ of } A_g = 12150 \text{ mm}^2$$

$$A_{sc \text{ min.}} = 0.8\% \text{ of } A_g = 1620 \text{ mm}^2$$



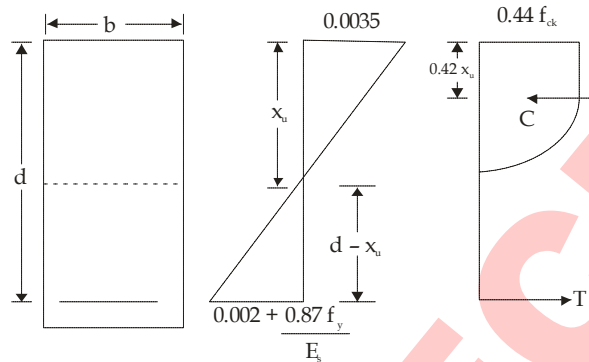
So minimum $A_{sc} = 1620 \text{ mm}^2$ should be provided.

65.

Sol.

(a) Depth of neutral axis

Let x_u be the depth of neutral axis and for balance section width of beam is b , and effective depth of beam is d . Now from below diagram



$$\frac{0.0035}{x_u} = \frac{0.002 + 0.87 \frac{f_y}{E_s}}{d - x_u} = \frac{0.002 + \frac{0.87 \times 415}{2 \times 10^5}}{d - x_u}$$

On solving $x_u = 0.479 d$ (i)

(b) Limiting Moment

$$M_u = C \times \text{lever arm} = C \times (d - 0.42 x_u)$$

$$C = \text{Total compressive force} = 0.36 f_{ck} b x_u$$

Since $x_u = 0.479 d$

$$\begin{aligned} \therefore M_u &= 0.36 f_{ck} \times b \times x_u \times (d - 0.42 x_u) \\ &= 0.36 f_{ck} b \times 0.479 d (d - 0.42 \times 0.479 d) \\ &= 0.138 f_{ck} b d^2 \text{ (ii)} \end{aligned}$$

(c) Area of steel

For determination of steel area, equate total compressive force to total tensile force, $C = T$

$$\therefore 0.36 f_{ck} b x_u = 0.87 f_y \times A_{st}$$

$$\text{So, } A_{st} = \frac{0.36 \times f_{ck} \times b \times x_u}{0.87 f_y}$$

$$A_{st} = \frac{0.36 \times f_{ck} \times b \times 0.479 d}{0.87 f_y}$$

$$A_{st} = 4.776 \times 10^{-4} f_{ck} b d \text{ (iii)}$$

Hence, in question given all relations are proved.

66.

Sol. Given :

Column : $300 \text{ mm} \times 300 \text{ mm}$

Axial load $P = 500 \text{ kN}$

$q_0 = 200 \text{ kN/m}^2$

M20 | Fe415

Adopting WSM

Weight of footing $= 0.1 P$

$$\text{Area of footing required} = \frac{1.1P}{q_0} = \frac{1.1 \times 500}{200} = 2.75 \text{ m}^2$$

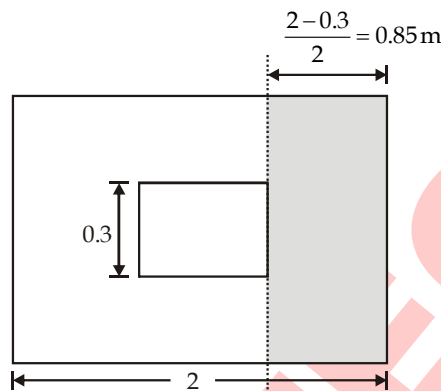
Let $B \times B$ be the size of footing.

$$\therefore B = \sqrt{A} = \sqrt{2.75} = 1.66 \text{ m}$$

Let us take $2 \text{ m} \times 2 \text{ m}$ size of footing.

$$\text{Net soil pressure, } W = \frac{P}{A} = \frac{500}{4} = 125 \text{ kN/m}^2$$

Check for Bending Moment :



$$M = W \times \frac{(B-b)^2}{8} \times B$$

$$M = 125 \times \frac{(2-0.3)^2}{8} \times 2 = 90.3125 \text{ kN-m} \quad \dots(i)$$

Depth required:

$$d = \sqrt{\frac{M}{QB}} \quad \dots(ii)$$

As we know that,

$$Q = \frac{1}{2} \sigma_{cbc} j k$$

where,

$$\sigma_{cbc} = 7 \text{ N/mm}^2 \text{ (for M20)}$$

$$k = \frac{280}{280 + 3\sigma_{st}} \text{ (here } \sigma_{st} = 230 \text{ N/mm}^2)$$

so,

$$k = \frac{280}{280 + 3 \times 230} = 0.29$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.29}{3} = 0.903$$

Hence,

$$Q = \frac{1}{2} \times 7 \times 0.903 \times 0.29 = 0.92 \quad \dots(iii)$$

Using equation (i) and (iii) in equation (ii), we get

$$d = \sqrt{\frac{90.3125 \times 10^6}{0.92 \times 2000}} = 221.55 \text{ mm}$$

Let us take $d = 350 \text{ mm}$ and effective cover = 70 mm

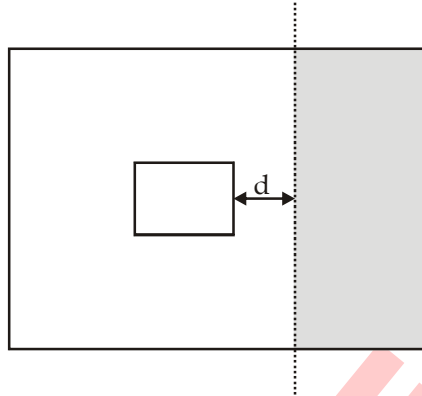
$$\therefore \text{Depth, } D = 420 \text{ mm}$$

Check for one-way-shear :

Critical section is at distance d from face of the column.

$$\left(\frac{B-b}{2} - d\right) = 0.85 - 0.35 = 0.5 \text{ m}$$

$$V_u = W \times \left(\frac{B-b}{2} - d\right) \times B = 125 \times 0.5 \times 2 = 125 \text{ kN}$$



$$\tau_v = \frac{V_u}{Bd} = \frac{125 \times 10^3}{2000 \times 350} = 0.178 \text{ N/mm}^2 < 0.18 \text{ N/mm}^2 \{\tau_{c, \min}\}$$

Check for punching shear,

Critical section is at distance $d/2$ from face of the column.

$$\tau_p = \frac{P - W(b+d)^2}{4 \times (b+d) \times d} = \frac{500 \times 10^3 - 125 \times 10^{-3} (0.65 \times 10^3)^2}{4 \times (0.65 \times 10^3) \times 350}$$

$$= 0.49 \text{ N/mm}^2$$

$$\tau_{p, \text{per}} = k_B \times 0.16 \times \sqrt{f_{ck}}$$

$$k_B = 0.5 \left(1 + \frac{b}{a}\right) \nless 1$$

Here,

$$b = a = 300 \text{ mm}$$

So,

$$k_B = 1$$

Hence,

$$\tau_{\text{per}} = 1 \times 0.16 \times \sqrt{20} = 0.72 \text{ N/mm}^2 > \tau_p$$

Hence, safe.

Area of steel required.

$$A_{st_x} = A_{st_y} = \frac{M}{\sigma_{st} \cdot j \cdot d} = \frac{90.3125 \times 10^6}{230 \times 0.9 \times 350}$$

$$= 1246.55 \text{ mm}^2$$

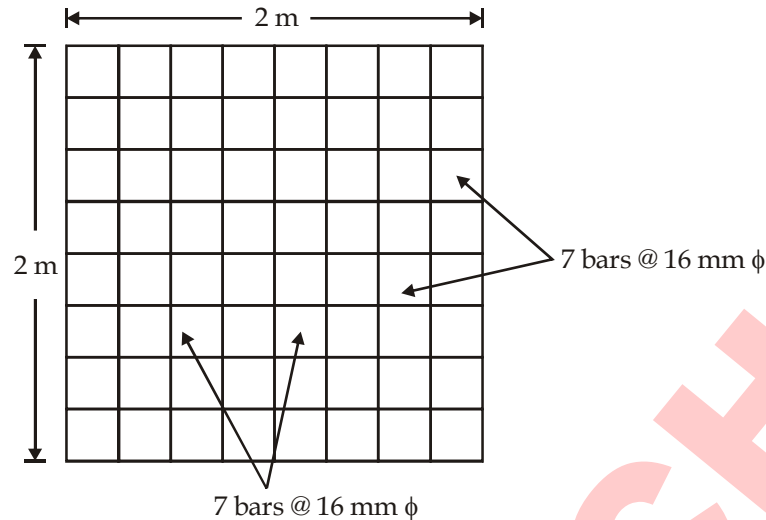
Minimum reinforcement,

$$A_{st \min} = \frac{0.12}{100} \times 2000 \times 350$$

$$= 840 \text{ mm}^2 < 1246.55 \text{ mm}^2 \text{ (ok)}$$

$$\text{Number of 16 mm } \phi \text{ bars} = \frac{1246.55}{\frac{\pi}{4} \times 16^2} = 6.19 \text{ say } 7$$

$\therefore$ Provide 7 - 16 mm ϕ bars in x and y direction.



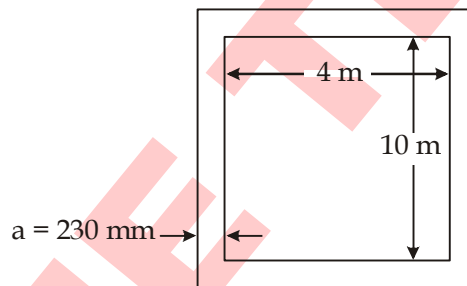
67.

Sol.

Given:

Dimension of slab is 4 m × 10 m

Thickness of supporting wall = 230 mm

**Step-I Calculation of depth of slab :**To start with, we assume the effective span to be $(L_c + a) = (4000 + 230) = 4230$ mm

$$\text{Now, } \left(\frac{l}{d}\right) < \left(\frac{l}{d}\right)_{\text{basic}} \cdot k_1 k_2 k_3$$

$$\text{Assume } k_2 = k_3 = 1$$

$$k_1 = 1.2$$

$$\Rightarrow \left(\frac{4230}{d}\right) < 20 \times 1.2$$

$$\Rightarrow d > 176.25 \text{ mm}$$

$$\text{Adopt } d = 180 \text{ mm}$$

Take effective cover = 20 mm

$$\therefore \text{Overall depth, } D = 180 + 20 = 200 \text{ mm}$$

Assume floor finish of 40 mm

$$\therefore \text{Dead load, } w_d = (0.20 + 0.04) \times 25 = 6 \text{ kN/m}^2$$

Let us assume live load = 3 kN/m²

$$\therefore \text{Total factored load, } w_t = 1.5(6 + 3) = 13.5 \text{ kN/m}^2$$

Step-II : Calculation of moment :

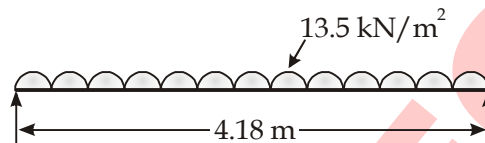
Effective span for simply supported slab = $\min \begin{cases} L_c + a \\ L_c + d \end{cases}$

$$L_x = \min \begin{cases} (4000 + 180) \text{ mm} \\ (4000 + 230) \text{ mm} \end{cases} = 4180 \text{ mm} = 4.18 \text{ m}$$

$$L_y = \min \begin{cases} (10000 + 180) \text{ mm} \\ (10000 + 230) \text{ mm} \end{cases} = 10.18 \text{ m}$$

So, $\frac{L_y}{L_x} = 2.43 > 2 \Rightarrow$ one-way slab.

One way slab will bend about the shorter span.



Maximum bending moment, $M_{\max} = \frac{13.5 \times 4.18^2}{8} = 29.48 \text{ kN-m/m}$ (per m width)

Maximum shear force, $V_{\max} = \frac{13.5 \times 4.18}{2} = 28.2 \text{ kN/m}$ width of slab

Step-III : Calculation of area of steel :

$$M_{\max} = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b} \right)$$

$$\Rightarrow 29.48 \times 10^6 = 0.87 \times 415 \times A_{st} \left(180 - \frac{415 A_{st}}{20 \times 1000} \right)$$

On solving, we get

$$A_{st} = 480.19 \text{ mm}^2$$

For minimum steel,

$$\frac{A_{st \min}}{bd} = \frac{0.85}{f_y}$$

$$\Rightarrow A_{st \min} = \frac{0.85}{415} \times 180 \times 1000 = 368 \text{ mm}^2 < A_{st}$$

Let us provide 10 mm ϕ bars

$$\therefore \text{Spacing} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{480.19} = 163.56 \text{ mm}$$

$$< 300 \text{ mm}$$

$$< 3d \text{ (OK)}$$

Hence, Provide 10 mm ϕ bars @ 160 mm c/c spacing for main reinforcement.

$\Rightarrow$ **Distribution Steel :**

Provide $A_{st} = 0.12\%$ of Gross area

$$= 0.12 \times 200 \times \frac{1000}{100} \text{ mm}^2 = 240 \text{ mm}^2$$

Adopt 8 mm ϕ bars

$$\therefore \text{Spacing} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{240} = 209.44 \text{ mm}$$

$$< 450 \text{ mm}$$

$$< 5d \text{ (OK)}$$

Hence, Provide 8 mm ϕ bars @ 200 mm c/c spacing for distribution steel.

Step-IV : Check for shear :

$$A_{st \text{ provided}} = \frac{1000 \times \frac{\pi}{4} \times 10^2}{160} = 490 \text{ mm}^2$$

$$p_t = \frac{490}{1000 \times 180} \times 100 = 0.27\%$$

For M20 concrete and $p_t = 0.27\%$

For M20 concrete

p_t	τ_c
0.25	0.36
0.50	0.48

So, for $p_t = 0.27$, τ_c is obtained by linear interpolation.

$$\tau_{c, p_t=0.27} = 0.36 + \frac{0.48 - 0.36}{0.50 - 0.25} \times (0.27 - 0.25)$$

$$\tau_c = 0.3696 \approx 0.37$$

So,

$$\tau_c = 0.37 \text{ N/mm}^2$$

$$\text{Maximum shear stress, } \tau_v = \frac{V_{\max}}{bd} = \frac{28.2 \times 1000}{180 \times 1000} = 0.16 \text{ N/mm}^2$$

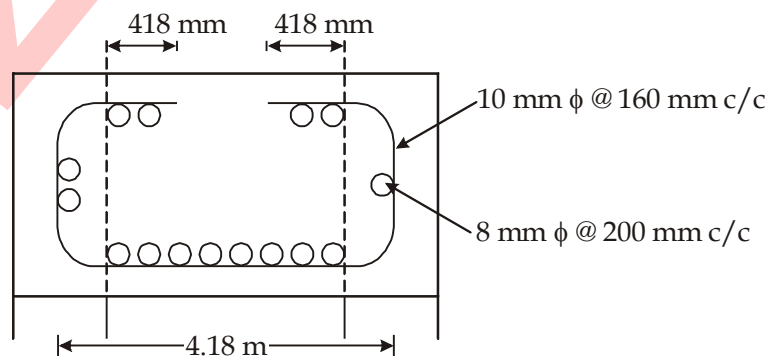
For

$$D = 200 \text{ mm}$$

$$k = 1.2$$

$$k \cdot \tau_c = 1.2 \times 0.37 = 0.444 > \tau_v \text{ (OK)}$$

Slab is safe in shear without shear reinforcement.



68.

Sol. Given : $P = 550 \text{ kN}$
 Effective length, $L = 3.3 \text{ m}$
 $\sigma_{cc} = 5 \text{ N/mm}^2$
 $\sigma_{sc} = 190 \text{ N/mm}^2$

One side of column = 250 mm

Step-I Calculation of dimension of column :

Let us provide 1% reinforcement

$$A_{sc} = 1\% \text{ of } A_g = \frac{A_g}{100}$$

$$P = \sigma_{cc}(A_g - A_{sc}) + \sigma_{sc}A_{sc}$$

$$\Rightarrow 550 \times 10^3 = 5 \times \left(A_g - \frac{A_g}{100} \right) + 190 \times \frac{A_g}{100}$$

On solving, we get

$$A_g = 80291.97 \text{ mm}^2$$

One side = 250 mm

$$\therefore \text{Other side} = \frac{80291.97}{250} = 321.17 \text{ mm} \approx 325 \text{ mm}$$

Let us provide rectangular column of size (250 × 325)

Step-II : Calculation of Area of Steel :

Area of steel (A_{sc}) = 1% of A_g

$$= \frac{1}{100} \times (325 \times 250) \text{ mm}^2 = 812.5 \text{ mm}^2$$

Let us provide 20 mm ϕ bars

$$\therefore \text{Number of bars} = \frac{812.5}{\frac{\pi}{4} \times 20^2} = 2.586$$

Provide 4 number of 20 mm ϕ bars.

Step-III : Design of Lateral Ties :

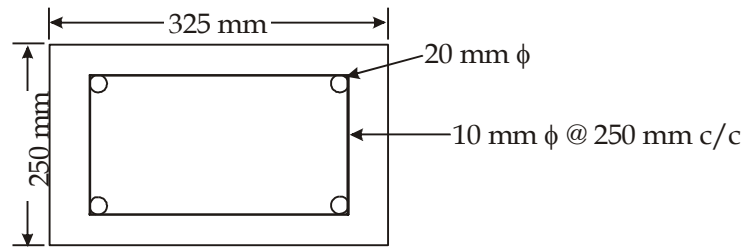
$$\text{Tie diameter i.e. } \phi_t = \text{Maximum of } \begin{cases} 10 \text{ mm} \\ \frac{\phi}{4} = \frac{20}{4} = 5 \text{ mm} \\ 6 \text{ mm} \end{cases}$$

So, tie diameter, $\phi_t = 10 \text{ mm}$

$$\text{Pitch} = \text{Minimum of } \begin{cases} \text{Least lateral dimension} = 250 \text{ mm} \\ 16\phi = 320 \text{ mm} \\ 300 \text{ mm} \end{cases}$$

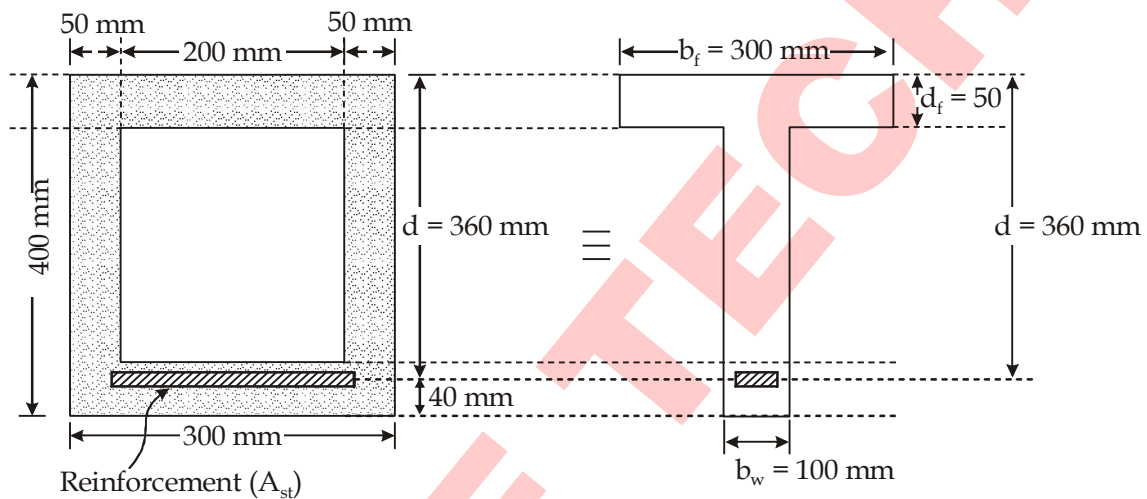
So, pitch = 250 mm

Thus, provide 10 mm ϕ ties @ 250 mm c/c



69.

Sol.



The hollow concrete block can be assumed to be a T-beam of

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

$$D = 400 \text{ mm}$$

$$d = 360 \text{ mm}$$

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

Let us assume,

$$x_u < d_f$$

∴ At $x_u = d_f$

$$M_u = 0.36 f_{ck} b_f d_f (d - 0.42 d_f)$$

$$= 0.36 \times 25 \times 300 \times 50 (360 - 0.42 \times 50)$$

$$= 45.765 \text{ kNm} > M_{u, \text{applied}}$$

∴ NA lies in flange

$$\therefore M_{u, \text{applied}} = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$$

$$\therefore 40 \times 10^6 = 0.36 \times 25 \times 300 x_u (360 - 0.42 x_u)$$

$$\Rightarrow x_u = 43.34 \text{ mm}$$

$$C = T$$

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$\Rightarrow A_{st} = \frac{0.36 \times 25 \times 300 \times 43.34}{0.87 \times 415} = 324.13 \text{ mm}^2$$

Provide 3 # 12ϕ bars

$$\Rightarrow A_{st, \text{provided}} = 339 \text{ mm}^2$$

Check for minimum reinforcement

$$\frac{0.85}{f_y} = \frac{A_{st, \text{min}}}{bd}$$

$$\Rightarrow A_{st, \min} = \frac{0.85 \times 300 \times 360}{415} = 221.2 \text{ mm}^2 < A_{st, \text{provided}} \text{ (OK)}$$

70.

Sol. Given :

$$P = 1500 \text{ kN}$$

$$P_u = 1.5 \times 1500 = 2250 \text{ kN}$$

$$\text{Circular dia.} = 450 \text{ mm}$$

$$L_{\text{eff}} = 3.5 \text{ m} \therefore L_{\text{eff}} = 1 \times L$$

Check for slenderness ratio

$$\frac{L_{\text{eff}}}{D} = \frac{3500}{450} = 7.78 < 12$$

Therefore, column can be designed as short column

Check for minimum eccentricity i.e. $e_{\min}$

$$e_{\min} = \left[\frac{L_{\text{eff}}}{500} + \frac{D}{30} \text{ or } 20 \text{ mm} \right] \text{ whichever is greater}$$

$$e_{\min} = \left[\frac{3500}{450} + \frac{450}{30} = 22.8 \text{ mm or } 20 \text{ mm} \right] \text{ whichever is greater}$$

So,

$$e_{\min} = 22.8 \text{ mm}$$

As,

$$0.05 D = 22.5 \text{ mm}$$

 $\approx e_{\min} \Rightarrow$ Codal formula for axially compressed short column may be used

$$P = 1.05 (0.4 f_{ck} A_c + 0.67 f_y A_{sc})$$

Area of concrete,

$$A_c = \frac{\pi}{4} \times 450^2 - A_{sc}$$

$$A_{sc} = \text{Area of steel}$$

$$2250 \times 10^3 = 1.05 \left(0.4 \times 25 \times \left(\frac{\pi}{4} \times 450^2 - A_{sc} \right) + 0.67 \times 415 A_{sc} \right)$$

$$A_{sc} = 2060.9 \text{ mm}^2$$

$$A_{sc, \min} = 0.8 \% \text{ of } A_g = \frac{0.8}{100} \times \frac{\pi}{4} \times 450^2 = 1272.35 \text{ mm}^2$$

Providing 8 bars of 20 mm ϕ

$$\Rightarrow A_{sc, \text{provided}} = 8 \times \frac{\pi}{4} \times 20^2 = 2513.27 \text{ mm}^2 > 2069.9 \text{ mm}^2 \text{ (OK)}$$

Design of spiral reinforcement

Assuming a clear cover of 40 mm over spirals,

$$\text{Core diameter} = 450 - (40 \times 2) = 370 \text{ mm}$$

Assuming a spiral bar diameter of 6 mm and pitch S_t

$$\rho_s = \frac{\text{Volume of spiral reinforcement}}{\text{Volume of core}} \text{ per unit length of column}$$

$$= \frac{\left(\frac{\pi}{4} \times 6^2 \right) \times \pi \times \frac{(370-6)}{S_t}}{\left(\frac{\pi}{4} \times 370^2 \right)} = \frac{0.3007}{S_t}$$

As per code :

$$\rho_s \geq 0.36 \left[\frac{A_g}{A_{core}} - 1 \right] \left[\frac{f_{ck}}{f_y} \right]$$

$$\Rightarrow \frac{0.3007}{S_t} \geq 0.36 \left[\frac{\frac{\pi}{4} \times 450^2}{\frac{\pi}{4} \times 370^2} - 1 \right] \left[\frac{25}{415} \right]$$

$$\Rightarrow \frac{0.3007}{S_t} \geq 0.36 \times (0.479) \times (0.0602)$$

$$S_t \leq 28.94 \text{ mm}$$

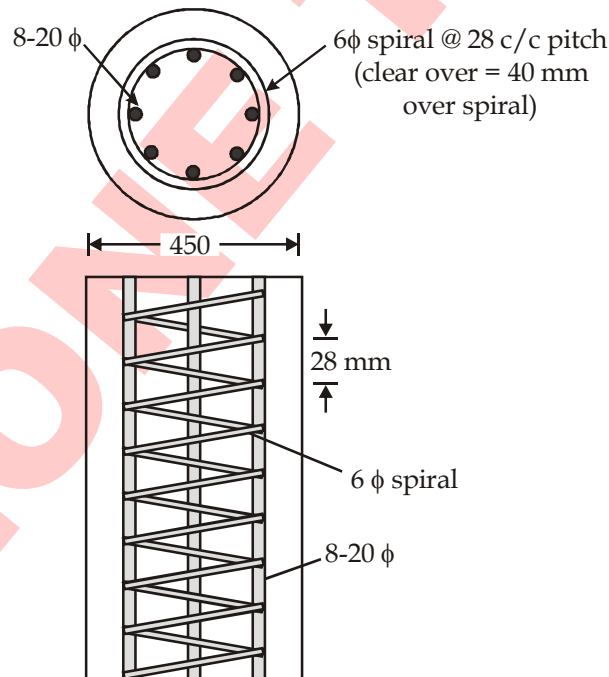
As per code

$$S_t < \begin{cases} 75 \text{ mm} \\ \frac{\text{Core dia.}}{6} = 61.67 \text{ mm} \end{cases}$$

and

$$S_t > \begin{cases} 25 \text{ mm} \\ 3\phi_t = 18 \text{ mm} \end{cases}$$

Providing 6 ϕ spiral at 28 mm c/c pitch.



71.

Sol.

Given :

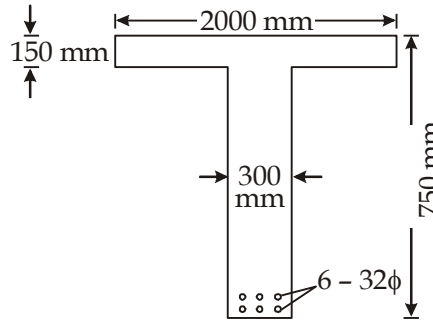
Flange width, (b) = 2000 mm

Span, (l_0) = 9 m = 9000 mm

Flange thickness, (D_f) = 150 mm

Overall depth, (D) = 750 mm

Rib width, (b_w) = 300 mm



$$\therefore A_{st} = 6 \times \frac{\pi}{4} \times 32^2 = 4825.486 \text{ mm}^2$$

Effective width of flange for an isolated T-beam is given as

$$b_f = \frac{l_0}{\left(\frac{l_0}{b}\right) + 4} + b_w = \frac{9000}{\frac{9000}{2000} + 4} + 300$$

$$\Rightarrow b_f = 1358.82 \text{ mm} < b$$

Now to calculate the moment of resistance of the beam, we need to calculate the actual depth of neutral axis.

Assuming that the NA lies in the flange.

$$\text{Force of compression, (C)} = 0.36f_{ck} b_f x_u$$

$$\text{Force of tension, (T)} = 0.87f_y A_{st}$$

$$\text{But, } C = T$$

$$\therefore 0.36f_{ck} b_f x_u = 0.87f_y A_{st}$$

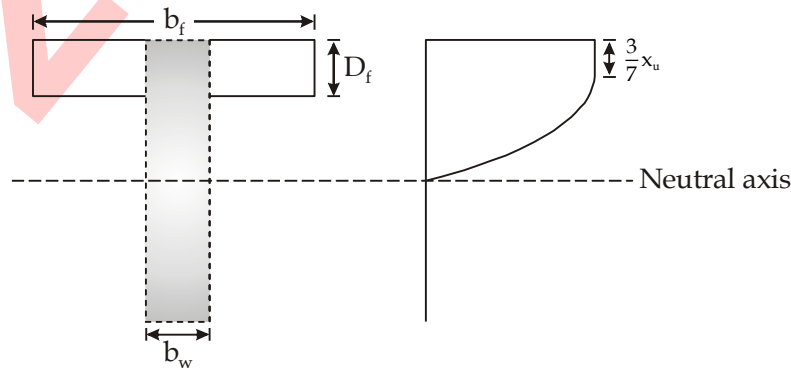
$$\Rightarrow x_u = \frac{0.87f_y A_{st}}{0.36f_{ck} b_f}$$

$$\Rightarrow x_u = \frac{0.87 \times 500 \times 4825.486}{0.36 \times 25 \times 1358.82} = 171.64 \text{ mm}$$

$$\therefore x_u > D_f$$

Hence, our assumption is wrong.

$\therefore$ NA lies in the web.



As we can see from the stress diagram above, depth of rectangular portion of stress diagram

$$= \frac{3}{7} x_u = \frac{3}{7} \times 171.64 = 73.56 \text{ mm}$$

$$\therefore D_f > \frac{3}{7} x_u$$

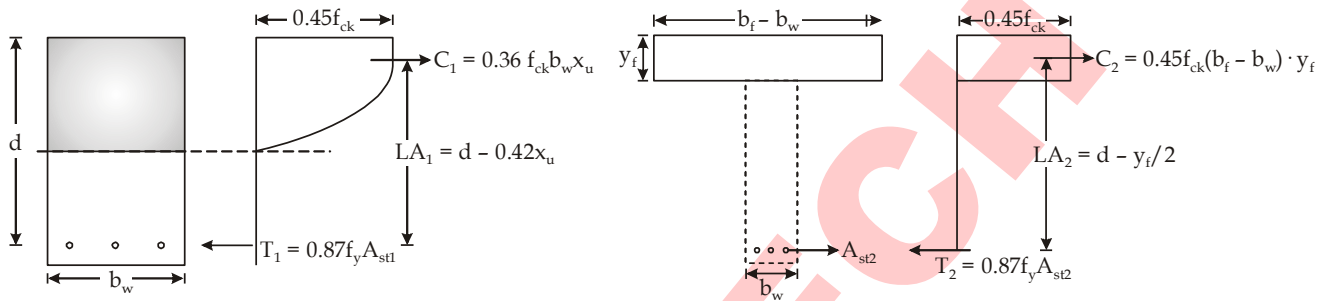
It means that depth of flange is greater than the rectangular portion of stress diagram.

In this case it is assumed that depth of rectangular portion of the stress block is equal to y_f

where,

$$y_f = 0.15x_u + 0.65D_f \geq D_f$$

$\therefore$ The given T-beam can be shown as



$\therefore$

$$\begin{aligned} C &= C_1 + C_2 \\ &= 0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) y_f \\ T &= T_1 + T_2 \\ &= 0.87 f_y A_{st1} + 0.87 f_y A_{st2} = 0.87 f_y A_{st} \end{aligned}$$

But,

$$\begin{aligned} \therefore 0.36 f_{ck} b_w x_u + 0.45 f_{ck} (b_f - b_w) y_f &= 0.87 f_y A_{st} \\ \Rightarrow 0.36 \times 25 \times 300 \times x_u + 0.45 \times 25 \times (1358.82 - 300) y_f &= 0.87 \times 500 \times 4825.486 \\ \Rightarrow 2700 x_u + 11911.725 \times (0.15 x_u + 0.65 \times 150) &= 2099086.41 \end{aligned}$$

Solving

$$x_u = 209 \text{ mm}$$

$\therefore$

$$x_u > D_f \text{ (OK)}$$

and

$$\frac{3}{7} x_u = \frac{3}{7} \times 209 = 89.57 \text{ mm}$$

$\therefore$

$$D_f > \frac{3}{7} x_u \text{ (OK)}$$

$\therefore$ Moment of the resistance of the beam with respect to concrete is given by

$$M_R = C_1 \times L_1 + C_2 \times L_2$$

$$= 0.36 f_{ck} b_w x_u \times (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2} \right)$$

Assuming effective cover = 50 mm

$\therefore$

$$\text{Effective depth, } (d) = D - 50$$

$$= 750 - 50 = 700 \text{ mm}$$

$$y_f = 0.15 x_u + 0.65 D_f$$

$$y_f = 0.15 \times 209 + 0.65 \times 150 = 128.85 \text{ mm}$$

$\therefore$

$$M_R = 0.36 \times 25 \times 300 \times 209 \times (700 - 0.42 \times 209) + 0.45 \times 25 \times (1358.82 -$$

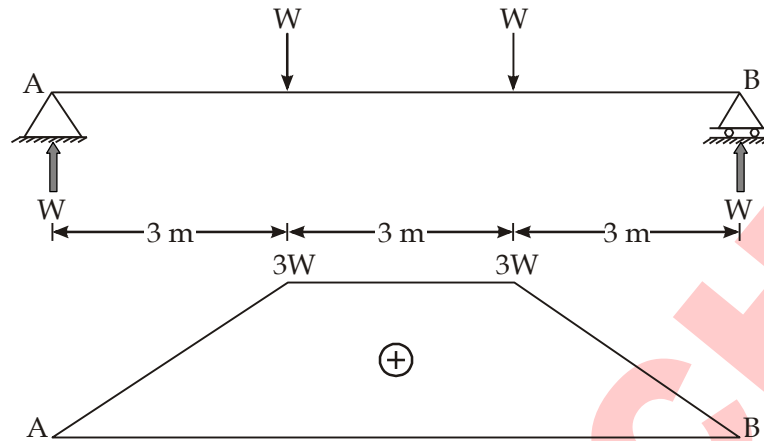
$$300) \times 128.85 \times \left(700 - \frac{128.85}{2} \right)$$

$$= 345475746 + 975496886.4$$

$$= 1320972632 \text{ N-mm}$$

$$= 1320.973 \text{ kN-m}$$

Now, to calculate the magnitude of point load, equating the bending moment to moment of resistance



$$W_d = (2 \times 0.15 + 0.3 \times 0.75) \times 25$$

$$W_d = 13.125 \text{ kN/m}$$

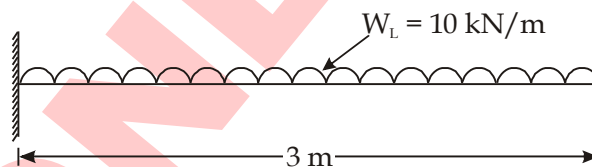
$$\frac{M_u}{1.5} = M_w = 880.65 \text{ kN-m}$$

$$880.65 = \frac{W_d l^2}{8} + \frac{Wl}{3} = \frac{13.125 \times 9^2}{8} + \frac{Wl}{3}$$

$$W = 249.253 \text{ kN}$$

72.

Sol. Given :



For M20 grade concrete and Fe415 steel,

$$\sigma_{cbc} = 7 \text{ N/mm}^2$$

For moderate exposure condition minimum grade of concrete is M25, but in question given M20.

So, we used according to question

$$\sigma_{st} = 230 \text{ N/mm}^2$$

Step-I : Calculation of Bending Moment :

For Cantilever beam, $\frac{L}{d} \leq 7$

$$\Rightarrow d \geq \frac{3000}{7} = 428.57 \text{ mm}$$

Let us provide $d = 450 \text{ mm}$

Assume effective cover of 50 mm

Then,

$$\text{Total Depth, } D = (450 + 50) \text{ mm} = 500 \text{ mm}$$

Let us assume width, $b = 300 \text{ mm}$

$$\therefore \text{Self weight, } w_d = 0.3 \times 0.5 \times 25 = 3.75 \text{ kN/m}$$

Hence,

$$\text{Total load, } w = w_L + w_d = 10 + 3.75 = 13.75 \text{ kN}$$

$$\therefore \text{Maximum bending moment, } M = \frac{wL^2}{2} = \frac{13.75 \times 3^2}{2} = 61.875 \text{ kN-m}$$

Step-II : Check for the sectional dimensions :

$$\text{Modular ratio} = \frac{280}{3\sigma_{cbc}} = 13.33$$

For M20 concrete and Fe 415 steel,

$$Q = 0.91$$

$$j = 0.9$$

Thus,

$$\text{Moment, } M = Qbd^2$$

$$\Rightarrow 61.875 \times 10^6 = 0.91(300)d^2$$

$$\Rightarrow d = 476.07 \text{ mm} > 450 \text{ mm}$$

Thus,

$$\text{Provide effective depth, } d = 480 \text{ mm}$$

$$\text{Total depth, } D = 480 + 50 = 530 \text{ mm}$$

$$\begin{aligned} \text{Hence, Revised bending moment} &= (10 + 0.3 \times 0.53 \times 25) \times \frac{3^2}{2} \text{ kN-m} \\ &= 62.88 \text{ kN-m} \end{aligned}$$

Step-III : Calculation of Area of steel :

$$\text{Area of tensile steel, } A_{st} = \frac{M}{\sigma_{st}jd} = \frac{62.88 \times 10^6}{230 \times 480 \times 0.9} \text{ mm}^2 = 632.85 \text{ mm}^2$$

Let us provide 4 no. of 16 mm ϕ bars

$$A_{st} \text{ provided} = 4 \times \frac{\pi}{4} \times 16^2 = 804.24 \text{ mm}^2 \approx 804 \text{ mm}^2 > 632.85 \text{ mm}^2$$

Step-IV : Calculation of shear reinforcement :

$$\text{Max shear force} = (10 + 0.53 \times 0.3 \times 25) \times 3 = 41.925 \text{ kN}$$

$$\therefore \text{Shear stress, } \tau_v = \frac{V}{bd} = \frac{41.925 \times 1000}{300 \times 480} \text{ N/mm}^2 = 0.29 \text{ N/mm}^2$$

$$\% \text{ reinforcement, } p_t = \frac{804}{300 \times 480} \times 100 = 0.558\%$$

For M20 concrete and $p_t = 0.558\%$

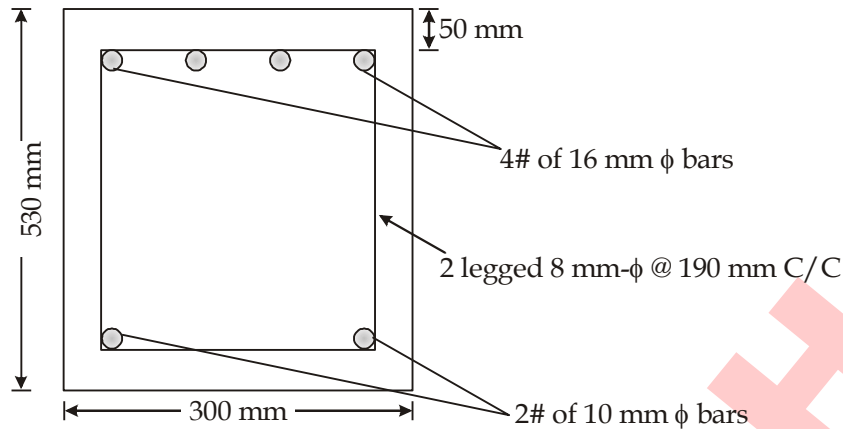
$$\tau_c = 0.3 \text{ N/mm}^2 > \tau_v$$

Hence, let us provide nominal shear reinforcement,

Provide 8 mm ϕ , 2-legged stirrups, for 0.3 N/mm² shear stress.

$$\text{Spacing, } s_v = \frac{\sigma_{st}A_{sv}}{0.4b} = \frac{230 \times 2 \times 50}{0.4 \times 300} = 191.67 \text{ mm}$$

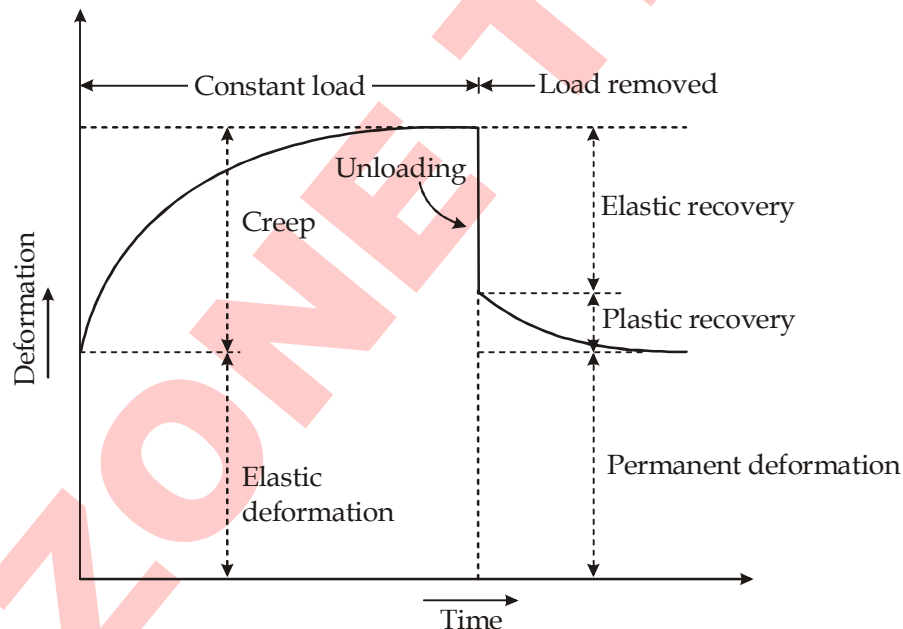
Hence, provide 8 mm- ϕ 2-legged stirrups at 190 mm centre to centre spacing



73.

Sol. Creep :

- It is continued deformation with time under a constant load, it is also known as plastic flow or time yield.
- Rate of creep decrease with time and the creep strains at five years are taken as terminal values. It increases rapidly with the stress, loading at an early age of concrete, using broken ballast, soft and porous aggregates, poorly graded and improperly compacted concrete.



Deformation of hardened concrete

- Creep may be due to closer of internal voids, viscous flow of the cement paste and flow of water out of the cement gel. Creep causes large deformations and deflections.

Beneficial effects of creep are :

- Reduction in stress induced by restrained shrinkage resulting in reduction in cracking.
- In indeterminate structures, stress induced due to settlement of support is reduced due to creep.

In general Creep increases when

- Cement-content is high
- Water-cement ratio is high
- Aggregate content is low
- Air entrainment is high
- Relative humidity is low



- Temperature (causing moisture loss) is high
- Loading occurs at an early age
- Loading is sustained over a long period and
- Size/thickness of the member is small.

Shrinkage :

Shrinkage can be defined as the volume changes in concrete due to loss of moisture at different stages due to different reasons. Shrinkage can be classified in the following ways :

- Plastic Shrinkage
- Drying Shrinkage
- Thermal Shrinkage
- Autogenous Shrinkage
- Carbonation Shrinkage

(i) Plastic Shrinkage

Plastic Shrinkage can be defined as loss of water from the surface of concrete in wet condition. High temperature, low humidity and wind.

(ii) Drying Shrinkage

It is defined as Shrinkage in volume due to loss of water in the gel pores. Under drying conditions, the gel water is lost progressively over a long time.

(iii) Thermal Shrinkage

The seasonal and daily variation of temperature cause concrete to shrink and expand.

(iv) Autogenous Shrinkage

It is not due to movement of moisture, but a volume change at a constant temperature. It is of minor importance and is not applicable in practice to many situations except that of mass of concrete in the interior of concrete dam.

(v) Carbonation Shrinkage

When carbon dioxide from the atmosphere gets penetrated in the concrete, shrinkage is caused by the dissolution of crystals of calcium hydroxide and deposition of calcium carbonate in its place. As the new product is less in volume than the product replaced, shrinkage takes place.

Methods to reduce shrinkage :

- Moisture sugrade and formworks to prevent absorption
- Dampen dry aggregate
- Use evaporation retarder
- Use of spray to increase humidity
- Prolonged curing
- Providing reinforcement, fibres etc.

74.

Sol. The bearing capacity of the soil = 200 kN/m^2

and $P = 600 \text{ kN}$

Total load on soil = load on column + self weight of footing

$$= 600 + 600 \times \frac{10}{100} = 660 \text{ kN.}$$

$$\text{Area of footing} = \frac{\text{Total load on soil}}{\text{Bearing capacity}} = \frac{660}{200} = 3.3 \text{ m}$$

∴ Size of the footing will be $B(\text{m}) \times B(\text{m})$.

$$B^2 = 3.3$$

⇒

$$B = 1.817 \text{ m} \approx 2 \text{ m.}$$

∴ Provide footing of size $2 \text{ m} \times 2 \text{ m}$



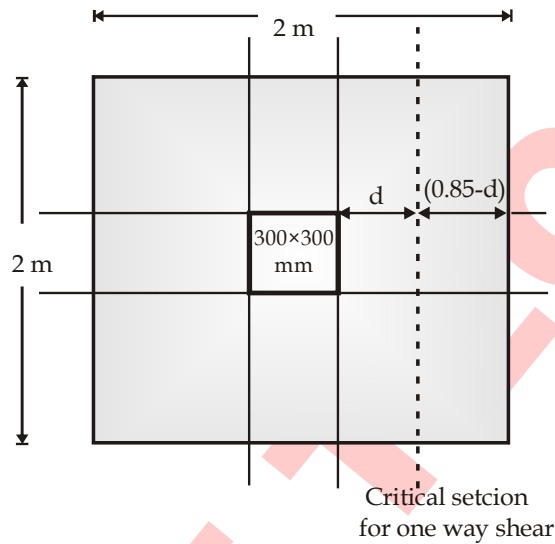
$$\text{Factored net soil pressure} = 1.5 \times \frac{600}{4} = 225 \text{ kN/m}^2$$

Calculation of depth of footing

(i) For one way shear

$$V = 2 \times (1 - 0.15 - d) \times 225 \text{ kN/m}^2$$

$$\tau_v = \frac{450(0.85 - d)}{2d} \text{ kN/m}^2$$



Assume 0.25% steel,

$$\tau_c = 0.36 \text{ N/mm}^2 \text{ for M25 grade concrete}$$

Assuming $K = 1$, then (for overall depth of slab $\geq 300 \text{ mm}$)

$$\tau_v < k\tau_c$$

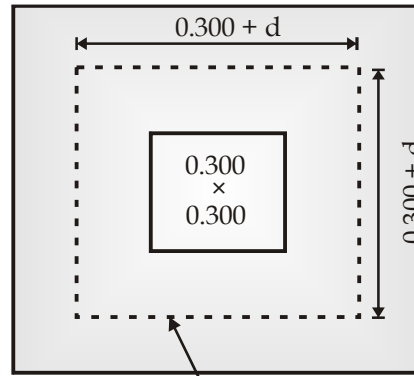
$$\frac{450(0.85 - d)}{2d} \times 10^{-3} < 0.36$$

$$\therefore d > 0.327 \text{ m}$$

(ii) For two way shear

$$\text{Net punching force, } F_p = \{2 \times 2 - (0.3 + d)^2\} \times 225 \text{ kN}$$

$$F_p = (4 - (0.3 + d)^2) \times 225 \text{ kN}$$



Critical Section for
two way shear

$$\text{Punching shear} = \frac{F_p}{\text{Perimeter} \times d}$$

$$\tau_{vp} = \frac{\{4 - (0.3 + d)^2\} \times 225}{4(0.3 + d)d} \text{ kN/m}^2$$

$$\tau_{vp} < k_s \cdot \tau_{cp}$$

$$\tau_{cp} = 0.25\sqrt{f_{ck}} = 1.118 \text{ N/mm}^2$$

$$k = 0.5 + \beta \left. \begin{matrix} \\ 1 \end{matrix} \right\} \min$$

$$\beta = \frac{B}{L}$$

$\Rightarrow$

$$k_s = 1$$

$\therefore$

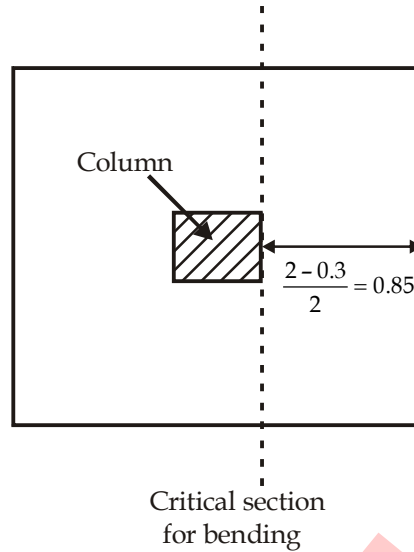
$$\tau_{vp} < k_s \cdot \tau_{cp}$$

$$\therefore \frac{\{4 - (0.3 + d)^2\} \times 225}{4(0.3 + d)d} \times 10^{-3} < 1.118 \times 1$$

$\therefore$

$$d > 0.303 \text{ m}$$

(iii) For Flexure



$$\begin{aligned}
 BM &= \frac{wl^2}{2} \times 2 \\
 &= \frac{225 \times (0.85)^2}{2} \times 2 \\
 &= 162.562 \text{ kNm.}
 \end{aligned}$$

For under reinforced section $M_u \leq M_{u_{lim}}$

$$162.562 \times 10^6 \leq 0.36 f_{ck} x_{u_{lim}} b [d - 0.42 x_{u_{lim}}]$$

$$162.562 \times 10^6 \leq 0.36 \times 25 \times 0.48 \times d \times 2000 [d - 0.42 \times 0.48d]$$

$$d \geq 153.51 \text{ mm}$$

Hence provide a effective depth of footing slab $d = 350 \text{ mm}$

Area of steel calculation

$$M_u = 0.87 f_y A_{st} \left[d - \frac{0.42 \times 0.87 f_y \times A_{st}}{0.36 f_{ck} b} \right]$$

$$\therefore 162.562 \times 10^6 = 0.87 \times 415 \times A_{st} \left[350 - \frac{0.42 \times 0.87 \times 415 \times A_{st}}{0.36 \times 25 \times 2000} \right]$$

$$\therefore 450245.88 = A_{st} [350 - 0.010531 A_{st}]$$

$$\therefore A_{st} = 1340.5 \text{ mm}^2$$

$$\therefore \text{No. of } 12\phi \text{ bars are required} = \frac{1340.5}{\frac{\pi}{4} \times 12^2} = 11.85 \approx 12 \text{ no.}$$

$$\therefore \% \text{ of steel provided} = \frac{12 \times \pi / 4 \times 12^2}{2000 \times 350} \times 100 = 0.194\% < 0.25\%$$

(Note that 0.25% steel has been adopted while checking for one way shear)

Let us provide % of steel = 0.25%

$$A_{st} = \frac{0.25}{100} (2000 \times 350)$$

$$\therefore A_{st} = 1750 \text{ mm}^2$$

$$\therefore 12 \text{ mm } \phi \text{ bars provided} = 15.47 \approx 16 \text{ no.}$$

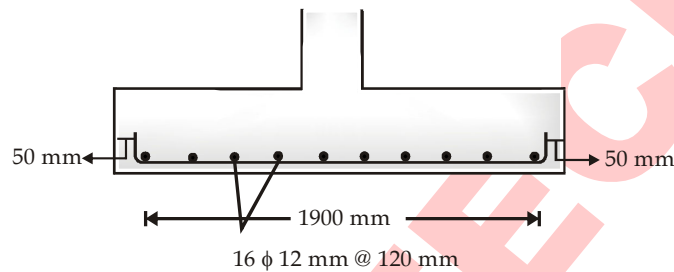
Thus reinforcement is greater than 0.12% OK

$$\therefore \text{Spacing will be} = \frac{2000 - 50 \times 2 - (12 \times 2) - 6 \times 2}{15} = \frac{1864}{15} = 124.26 \text{ mm}$$

$$\text{Spacing} \nlessgtr 3d \text{ i.e. } 3 \times 350 \text{ mm}$$

$$\nlessgtr 300 \text{ mm}$$

Hence spacing is OK



Check for development length:

Critical section for checking the development length of slab reinforcement is at the face of column
development length req. for Fe415 grade HYSD steel and M25 grade concrete.

$$\begin{aligned} &= \frac{0.87 f_y \phi}{4 \tau_{bd}} \\ &= \frac{0.87 \times 415 \phi}{4 \times 1.6 \times 1.4} [\tau_{bd} \text{ for M25 grade concrete} = 1.4 \text{ N/mm}^2] \\ &= 40.296 \phi \approx 41 \phi \\ &= 41 \times 12 = 492 \text{ mm} \end{aligned}$$

$$\text{available length} = \frac{2000 - 300}{2} = 850 \text{ mm}$$

$$\Rightarrow 850 > 492 \text{ mm}$$