

**DIPLOMA EXAMINATION IN ENGINEERING/TECHNOLOGY/
MANAGEMENT/COMMERCIAL PRACTICE — APRIL, 2019**

STRUCTURAL DESIGN – I

[Time : 3 hours

(Maximum marks : 100)

[Note:— Use of IS 456-2000, SP-16 are permitted.]

PART — A

(Maximum marks : 10)

Marks

I Answer *all* questions in one or two sentences. Each question carries 2 marks.

1. Define water cement ratio.
2. What is a T-beam ?
3. What is a one way slab ?
4. Define effective length of column.
5. Define the term nosing.

(5×2 = 10)

PART — B

(Maximum marks : 30)

II Answer *any five* of the following questions. Each question carries 6 marks.

1. What is limit state ? What are the various limit states for section is designed ?
2. Differentiate between singly and doubly reinforced beams.
3. What do you understand by the term development length ?
4. Determine the ultimate moment of resistance of a 150mm thick slab reinforced with 10mm bars at 200mm c/c. The effective cover is 25mm. Use M20 concrete and Fe 415 steel.
5. What are the points to be considered in designing one way cantilever slab ?
6. Find the ultimate load carrying capacity and allowable load for a short column of size 500mm × 500 mm. The column is reinforced with 4 - 25 mm diameter bars. Use M20 concrete and HYSD grade Fe 415 steel. Assume $e_{min} < 0.05D$.
7. Distinguish between short and long column.

(5×6 = 30)

PART — C

(Maximum marks : 60)

(Answer *one* full question from each unit. Each full question carries 15 marks.)

UNIT — I

- III (a) Give steps for determining moment of resistance of a singly reinforced beam. 6
- (b) Design a doubly reinforced beam of rectangular section using the following data :
Effective span = 5m, Section = 250 mm × 500mm (overall), Total load = 40kN/m,
Effective cover = 50mm.
Use M20 and Fe 415. 9

OR

- IV (a) What is a doubly reinforced beam ? What are the conditions in which we design a doubly reinforced beam ? 7
- (b) Determine the moment of resistance of a beam of dimension 250mm × 350mm. The area of steel consists of 3 bars of 12mm diameter placed at a distance of 40mm from bottom of beam. Use M20 and Fe 415 steel. 8

UNIT — II

- V (a) Find the moment of resistance of a T-beam having a web width of 240mm, effective depth of 400mm flange width of 740mm and flange thickness equal to 100mm. The beam is reinforced with 5 -16mm², Fe 415 bars. Use M20 concrete. 8
- (b) A reinforced beam of span 5m is 250mm wide and 500mm deep (effective). It has 4 bars of 22mm as tensile reinforcement. The beam carries a load of 30kN/m inclusive of self weight. Design the beam for shear. Use M20 and Fe 415 steel. 7

OR

- VI (a) Write notes on :
(i) Bond (ii) Types of shear reinforcement (iii) Curtailment of bars. 6
- (b) Determine the limiting moment of resistance and limiting area of steel for a reinforced concrete T-beam having flange width of 1600 mm, effective depth of 350mm and thickness of flange is 100mm. The width of web is 250mm. Use M20 grade of concrete and Fe 500 steel. 9

UNIT — III

- VII (a) What is the function of providing distribution steel in a slab ? 6
- (b) Design a reinforced concrete simply supported slab with corners free to lift for a room measuring 4m × 5m from inside. The slab carries a live load of 3kN/m² inclusive of floor finishes etc. Use M20 concrete and Fe 415 steel. 9

OR

	Marks
VIII (a) What are the difference between one way and two way slab ?	6
(b) Design a simply supported roof slab for a room $7.5\text{m} \times 3.5\text{m}$ clear in size. The Slab is carrying an imposed load of 5kN/m^2 . Use M20 mix and Fe 415 steel.	9

UNIT — IV

IX (a) What is a column ? Give the classification of column.	6
(b) Design a column of size $450\text{mm} \times 600\text{mm}$ and having 3m unsupported length. The column is subjected to a load of 2000kN and is effectively held in position but not restrained against rotation. Use M20 concrete and Fe 415 steel.	9

OR

X (a) What are footing ? How are they classified ?	6
(b) Design a square footing of uniform thickness for an axially loaded column of $450\text{mm} \times 450\text{mm}$ size. The safe bearing capacity of soil is 190kN/m^2 . Load on column is 850kN. Use M20 concrete and Fe 415 steel.	9

33

15 5015

Version - A

Question No:

SCHEME OF VALUATION

Split Score
Total Score

STRUCTURAL DESIGN - I

PART - A

- | Question No. | Answer | Split Score | Total Score |
|--------------|--|-------------|-------------|
| I 1. | The water cement ratio is the ratio of the weight of cement water to the weight of cement used in a concrete mix. | 2 | |
| 2. | In RCC constructions, slab and beams are cast monolithically. The intermediate beams supporting the slab are called as T-beams. | 2 | |
| 3. | One way slab are those slabs in which the l_y/l_x ratio is greater than 2. | 2 | |
| 4. | The effective length of the column is defined as that length of the column which takes part in buckling under the action of loads. This is also defined as the length between the points of contraflexure of the buckled column. | 2 | |
| 5. | The horizontal distance between two risers is going and the portion projecting out from the riser surface is called Nosing. Nosing is provided when the available horizontal distance for a tread is less. | 2 | |

PART - B

1. The aim of this method is that the structure should be able to withstand safely all the load that are liable to act on it throughout its life and it should also get satisfied the serviceability requirements of limiting deflection and cracking. The 2 types of limit state collapse:

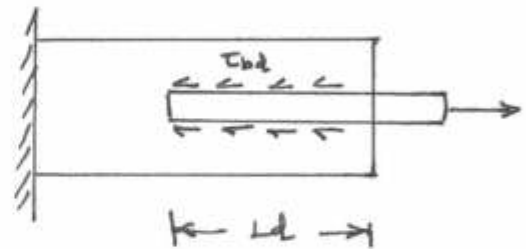
- i) Limit state of collapse.
 - ii) Limit state of serviceability.
- i) The following limit state of collapse are considered in designs:
- (a) Limit state of collapse in flexure (bending).
 - (b) Limit state of collapse in compression.
 - (c) Limit state of collapse in shear.
 - (d) Limit state of collapse in torsion.
- ii) Limit states of serviceability of following states:
- (a) Limit state of deflection.
 - (b) Limit state of cracking.

2. Singly reinforced beam — Beam is a flexural member which resists loads mainly by bending. Singly reinforced beams have steel reinforcement in the tension zone but in doubly reinforced beams steel reinforcement is provided in the tension as well as compression zone.

Doubly reinforced beam - The maximum moment of resistance of the singly reinforced section is $M_{u,lim}$, so for a beam if the bending moment is more than this limiting value two alternatives are available :

- (i) To increase the depth of the section.
- (ii) To provide compression reinforcement.

3. Development length - For the understanding of bond and development length, let us consider a steel bar embedded



in concrete as shown in fig. The bar is subjected to a tensile strength of T (force). Due to this tensile force, the steel bar will tend to come out and slip out of the concrete. This tendency of slipping is resisted by the bond stress developed over the surface of the bar. To avoid slipping

$$T \leq \tau_{bd} \times 2\pi \times \frac{\phi}{2} \times L_d \quad [\text{surface area} = 2\pi r \times L_d]$$

$$T = 0.87 f_y \times \frac{\pi}{4} \times \phi^2 \quad [\because T = 0.87 f_y \cdot A_\phi]$$

$$0.87 f_y \cdot \frac{\pi}{4} \phi^2 \leq \tau_{bd} \times 2\pi \times \frac{\phi}{2} \times L_d$$

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

$L_d =$ is the embedded length of steel bar.

4. $D = 150 \text{ mm}$ $d = 150 - 25 = 125 \text{ mm}$

$A_{st} = 10 \text{ mm } \phi @ 200 \text{ mm } c/c$

$f_{ck} = 20 \text{ N/mm}^2$ $f_y = 415 \text{ N/mm}^2$

Taking $b = 1000 \text{ mm}$ for slab, calculating A_{st} ,

$$A_{st} = \frac{1000 \cdot A_{\phi}}{\text{spacing}} = \frac{1000 \cdot \frac{\pi}{4} \times 10^2}{200} = 393 \text{ mm}^2$$

Depth of neutral axis (x_u)

$$x_u = \frac{0.87 f_y \cdot A_{st}}{0.86 f_{ck} \cdot b} = \frac{0.87 \times 415 \times 393}{0.86 \times 20 \times 1000} = 19.6 \text{ mm}$$

$x_{u\max} = 0.48 d = 0.48 \times 125 = 60 \text{ mm}$.

$x_u < x_{u\max}$, Hence the section is under-reinforced.

$$\begin{aligned} M_u &= 0.87 f_y \cdot A_{st} (d - 0.42 x_u) \\ &= 0.87 \times 415 \times 393 (125 - 0.42 \times 19.6) \\ &= 16568520.96 \text{ Nmm} = \underline{\underline{16.56 \text{ kNm}}} \end{aligned}$$

5. The points to be considered in design of one way slab:

- i, Effective span of cantilever is equal to projecting length of slab
- ii, Effective depth at fixed end is maximum and is assumed to be about $\text{Span}/10$ to $\text{Span}/12$.
- iii, Depth required at the free end is minimum and is kept $1/2$ or $1/3$ of the depth at fixed end
- iv, Main reinforcement is provided at the top and is to be curtailed at appropriate point.

6. $b = 500 \text{ mm}$ $d = 500 \text{ mm}$

$$A_{sc} = 4 \times \frac{\pi}{4} \times 25^2 = 1964 \text{ mm}^2$$

$$f_{ck} = 20 \text{ N/mm}^2 \quad f_y = 415 \text{ N/mm}^2$$

$$A_g = 500 \times 500 = 250000 \text{ mm}^2$$

$$A_c = A_g - A_{sc} = 250000 - 1964 = 248036 \text{ mm}^2$$

$$e_{min} < 0.05D$$

$$P_u = 0.4 f_{ck} \cdot A_c + 0.67 f_y \cdot A_s$$

$$= 0.4 \times 20 \times 248036 + 0.67 \times 415 \times 1964$$

$$= 2530378.2 \text{ N}$$

Ultimate load = 2530.38 kN

Allowable service load = $2530 / 1.5 = \underline{\underline{1686.92 \text{ kN}}}$.

Sl. No.	Short column	Long column
1.	Short if the ratio of effective length to least lateral dimension ≤ 12	Long if ratio of effective length to least lateral dimension > 12 .
2.	Ratio of eff. length to least radius of gyration is less than or equal to 40	Ratio of eff. length to least radius of gyration is greater than 40
3.	Buckling tendency is very low	Long and slender column easily
4.	The failure is by crushing	it fails in buckling.
5.	The load carrying capacity is higher as compared to long column of the same cross section area.	load carrying capacity is less as compared to short column of same cross-sectional area.

PART - C

MODULE I

III

a) The moment of resistance of a singly reinforced beam can be obtained as follows (IS 456:2000, Annex 9):

1. For the given grade of concrete and steel (f_{ck} and f_y known), find the depth of neutral axis of the given section:

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

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

2. Find limiting value of neutral axis i.e., $x_{u \max}$ by the following expression or refer Table 4.2 of (page 40 of IS 456)

$$\frac{x_{u \max}}{d} = \frac{0.0033}{\frac{0.87 f_y}{E_s} + 0.0033}$$

3. Compare x_u and $x_{u \max}$

(i) If $\frac{x_u}{d} = \frac{x_{u \max}}{d}$, the beam is designed as balanced section and moment of resistance of the section is given by following expression:

$$M_{u \lim} = 0.36 f_{ck} \frac{x_{u \max}}{d} \left(1 - \frac{0.42 x_{u \max}}{d}\right) b d^2$$

(ii) If $\frac{x_u}{d} < \frac{x_{u,max}}{d}$, the beam is under reinforced and moment of resistance is calculated by the following equation :

$$M_u = 0.87 f_y \cdot A_{st} \cdot d \left[1 - \frac{0.42 x_u}{d} \right]$$

(iii) If $\frac{x_u}{d} > \frac{x_{u,max}}{d}$, the moment of the resistance of the section is equal to $M_{u,lim}$ but the code recommends that the section is to be redesigned as it is a case of over reinforced section.

b)

$$l = 5m$$

$$b = 250mm,$$

$$D = 500mm$$

$$w = 40 kN/m$$

$$d = 500 - 50 - 450mm \quad d' = 50mm$$

$$f_{ck} = 20 N/mm^2$$

$$f_y = 415 N/mm^2$$

$$M_{u,lim} = 0.36 f_{ck} b \cdot x_{u,max} (d - 0.42 x_{u,max})$$

$$x_{u,max} = 0.48d = 0.48 \times 450$$

$$= 216mm$$

$$M_{u,lim} = 0.36 \times 20 \times 250 \times 216 (450 - 0.42 \times 216)$$

$$= 140 kNm$$

$$W_u = 1.5 \times 40 = 60 \text{ kN/m}$$

$$M_u = \frac{W_u \cdot l^2}{8} = \frac{60 \times 5^2}{8}$$

$$M_u = 187.5 \text{ kNm} > M_{u \text{ lim}}$$

$$M_{u_2} = M_u - M_{u \text{ lim}} = 187.5 - 140 \\ = 47.5 \text{ kNm}$$

Area of tensile steel (A_{st})

$$A_{st} = A_{st_1} + A_{st_2}$$

$$A_{st_1} = \frac{M_{u \text{ lim}}}{0.87 f_y d - 0.42 \lambda_{u \text{ max}}} \\ = \frac{140 \times 10^6}{0.87 \times 415 (450 - 0.42 \times 216)} = 1080 \text{ mm}^2$$

$$A_{st_2} = \frac{M_{u_2}}{0.87 f_y (d - d')} \\ = \frac{47.5 \times 10^6}{0.87 \times 415 (450 - 50)} = 330 \text{ mm}^2$$

$$A_{st} = 1080 + 330 \text{ mm}^2 = 1410 \text{ mm}^2$$

using 20mm diameter bars,

$$\text{Number of bars required} = \frac{1410}{314}$$

≈ 5 bars.

Area of compressive steel

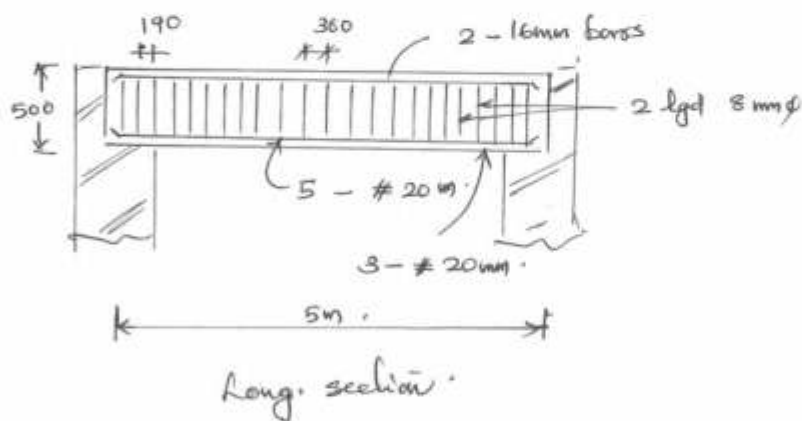
$$\frac{d'}{d} = \frac{50}{450} = 0.11 \text{ next higher value} = 0.15 \text{ (from table 6.1)}$$

$$f_{sc} = 342 \text{ N/mm}^2$$

$$A_{sc} = \frac{M_{u2}}{f_{sc} (d - d')} = \frac{47.5 \times 10^6}{342 (450 - 50)} = 348 \text{ mm}^2$$

Using 16 mm diameter bar,

$$\text{Number of bars} = \frac{348}{201} \approx 2 \text{ bars.}$$



OR

(a) The maximum moment of resistance of a singly reinforced section is $M_{u \text{ lim}}$. so for a beam if the bending moment is more than this limiting

IV

value two alternatives are available.

- 1) To increase the depth of the section.
- 2) To provide compression reinforcement.

In many cases the maximum of the section will be restricted from architectural or other consideration. The beam with reinforcement provided in tension side and compression side is called doubly reinforced beam.

Doubly reinforced sections are used in the following conditions :

(*)

1. When the dimensions (bxd) of the beam are restricted due to any constraints like availability of head room, architectural or space considerations and the moment of resistance of singly reinforced section is less than the external moment.
2. When the external loads may occur on either face of the member i.e., the loads are alternating or reversing and may cause tension on both faces of the member.
3. When the loads are eccentric.
4. When the beam is subjected to accidental or sudden lateral loads.

3

5. In the case of continuous beam or slab, the sections at support are generally designed as doubly reinforced sections.

(b)

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

$$d = 350 - 40 = 310 \text{ mm}$$

$$A_{st} = 3 \times \frac{\pi}{4} \times 12^2 = 339 \text{ mm}^2$$

$$f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 415 \text{ N/mm}^2$$

Depth of neutral axis (x_u)

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 339}{0.36 \times 20 \times 250} = 68 \text{ mm}$$

$x_{u \max}$ (limiting depth of neutral axis)

$$x_{u \max} = 0.48 d$$

$$= 0.48 \times 310$$

$$= 148.8 \text{ mm} > 68 \text{ mm}$$

$\therefore x_{u \max} > x_u$, hence, it is an under-reinforced section.

Moment of resistance (M_u)

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

$$= 0.87 \times 415 \times 339 (310 - 0.42 \times 68)$$

$$= 34447116.16 \text{ Nmm} = 34.44 \text{ kNm}$$

MODULE II

√ (a)

$b_w = 240 \text{ mm}$

$d = 400 \text{ mm}$

$b_f = 740 \text{ mm}$

$D_f = 100 \text{ mm}$

$A_{st} = 5 \times \pi/4 \times 16^2 = 1005.3 \text{ mm}^2$

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 415 \text{ N/mm}^2$

Assuming the neutral axis to fall in the flange

$$x_u = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times 1005.3}{0.36 \times 20 \times 740}$$

$= 68.1 \text{ mm} < D_f$ Hence, N.A. lies in flange

$x_{u,max} = 0.48d = 0.48 \times 400 = 192 \text{ mm}$ 3

$x_u < x_{u,max}$; hence the section is under reinforced.

Moment of resistance (M_u)

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

$$= 0.87 \times 415 \times 1005.3 \times 400 \left[1 - \frac{1005.3 \times 415}{740 \times 400 \times 20} \right]$$

$= 134953189.7 \text{ Nmm}$

$M_u = \underline{\underline{134.95 \text{ kNm}}}$

(b)

$l = 5m$ $d = 500mm$

$b = 250 mm$

$A_{st} = 4 \times \pi/4 \times 22^2 = 1516 mm^2$

$w = 30 kN/m$

$f_y = 415 N/mm^2$

$f_{ck} = 20 N/mm^2$

Factored load = $1.5 \times 30 = 45 kN/m$

Factored shear force = $V_u = \frac{45 \times 5}{2} = 112.5 kN$

Nominal shear stress (τ_v)

$\tau_v = \frac{V_u}{bd} = \frac{112.5 \times 10^3}{250 \times 500} = 0.9 N/mm^2$

Shear strength of concrete (τ_c)

$P_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times 1516}{250 \times 500} = 1.21\%$

For $P_t = 1.5\%$ and M20 concrete, $\tau_c = 0.66 N/mm^2$

$\tau_v > \tau_c$

Hence, shear reinforcement is necessary

$V_{us} = V_u - \tau_c bd$
 $= 112.5 \times 10^3 - 0.66 \times 250 \times 500$
 $= 30000 N$

Using 2 legged vertical shear stirrups of diameter 8mm.

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.5 \text{ mm}^2$$

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

$$= \frac{0.87 \times 415 \times 100.5 \times 500}{80000} = 604 \text{ mm}$$

3

But this spacing can't be more than following :

(i) spacing of nominal reinforcement

$$S_v = \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 415 \times 100.5}{0.4 \times 250} = 362 \text{ mm}$$

(ii) $0.75 d = 375 \text{ mm}$

(iii) 300 mm

Hence, providing 2 Igd 8mm dia stirrups @ 300mm c/c.

2

7

vi a)

(i) Bond - The term bond refers to the adhesion between concrete and steel which resist the slipping of steel bar from the concrete. It is this bond which is responsible for transfer of stresses from steel to concrete and thereby providing composite action of steel and concrete in R.C.C.

The bond develops due to setting of concrete on drying which results in gripping of the steel bar. 2

(ii) Types of shear reinforcement - the following three types of shear reinforcement are used :

1. vertical stirrups
2. Bent up bars along with stirrups
3. Inclined stirrups

2

(iii) Curtailment of bars

(a) For curtailment, reinforcement shall extended beyond the point at which it is no longer required to resist bending force a distance equal to d or 12ϕ whichever is more except at simple support.

(b) Positive Moment Reinforcement :

- i, At least $\frac{1}{8}^{rd}$ of positive moment in simply supported beams and $\frac{1}{4}$ of continuous beam should go into the support of $Ld/3$
- ii, At simple supports and at the point of inflexion positive moments reinforcement shall be limited to diameter such that

$$Ld = \frac{0.87 f_y \sigma_{st}}{4 \sigma_b} < \frac{M_1}{V} + l_0 \quad L_b \leq \frac{M_1}{V} + l_0$$

2 6

(b)

$b_f = 1600 \text{ mm}$

$d = 350 \text{ mm}$

$D_f = 100 \text{ mm}$

$b_w = 250 \text{ mm}$

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 500 \text{ N/mm}^2$

Limiting depth of neutral axis ($x_{u \text{ max}}$)

$x_{u \text{ max}} = 0.46 d = 0.46 \times 350 = 161 \text{ mm}$

$\frac{D_f}{d} = \frac{100}{350} = 0.28 > 0.2$ Since $\frac{D_f}{d} > 0.2$,
it is the case G2.2.1 of IS code

$$Y_f = 0.15 x_{u\max} + 0.65 D_f$$

$$= 0.15 \times 161 + 0.65 \times 100 = 89.15 \text{ mm}$$

Limiting moment of resistance ($M_{u\lim}$)

$$\frac{x_{u\max}}{d} = 0.46$$

$$M_{u\lim} = \frac{0.36 x_{u\max}}{d} \left[1 - 0.42 \frac{x_{u\max}}{d} \right] f_{ck} b_w d^2 +$$

$$0.45 f_{ck} [b_f - b_w] Y_f \left[d - \frac{Y_f}{2} \right]$$

$$= 0.36 \times 0.46 \left(1 - 0.42 \times 0.46 \right) 20 \times 250 \times 350^2 +$$

$$0.45 \times 20 (1600 - 250) 89.15 \left[350 - \frac{89.15}{2} \right]$$

$$= 81833724 + 330827960.8$$

$$= 412661684.8 \text{ Nmm} = 412.66 \text{ kNm.}$$

Limiting area of steel ($A_{st\lim}$)

It can be found by equating total compression and total tension.

$$\text{Total compression} = 0.36 f_{ck} b_w \cdot x_{u\max} + 0.45 f_{ck} (b_f - b_w) Y_f$$

$$\text{Total tension} = 0.87 f_y A_{st\lim}$$

$$\text{Total compression} = \text{Total tension}$$

$$0.36 f_{ck} b_w \cdot x_{u\max} + 0.45 f_{ck} (b_f - b_w) Y_f = 0.87 f_y \cdot A_{st\lim}$$

$$A_{st\lim} = \frac{0.36 f_{ck} b_w \cdot x_{u\max} + 0.45 f_{ck} (b_f - b_w) Y_f}{0.87 f_y}$$

$$= \frac{0.36 \times 20 \times 250 \times 161 + 0.45 \times 20 (1600 - 250) \times 89.15}{0.87 \times 500}$$

$$A_{st} \text{ lim} = \underline{\underline{3156.25 \text{ mm}^2}}$$

MODULE III

- a)
- (i) To distribute the concentrated loads coming on the slab
 - (ii) To protect against shrinkage and temperature stresses
 - (iii) To keep the main steel bar in position

The distribution steel is kept above the main steel and is not provided with hooks at the ends.

b) Slab = 4m x 5m

live load = 3 kN/m²

$$\frac{l_y}{l_x} = \frac{5}{4} = 1.25 < 2 \text{ hence, this is a two-way slab } \left[\frac{l}{d} \approx 25 \right]$$

Assuming, D = 150mm

$$d = 150 - 16 - 4 = 131 \text{ mm}$$

Load calculation, self weight = 0.15 x 25 = 2.5 kN/m²

live load including finishes cement = 3 kN/m²

Total load = 5.5 kN/m²

Factored load, $W_u = 1.5 \times 5.5 = 8.25 \text{ kN/m}^2$

Effective span in shorter direction = $4 + 0.131 = 4.131 \text{ m}$

Effective span in longer direction = $5 + 0.131 = 5.131 \text{ m}$

Design moments and shear

The slab is simply supported with corner free to lift up (unrestrained slab) hence, referring table 9.2

$$M_x = \alpha_x w l_x^2$$

$$M_y = \alpha_y w l_x^2$$

For $\frac{l_y}{l_x} = 1.25$

$$\alpha_x = \frac{0.093 + 0.084}{2} = 0.0885$$

$$\alpha_y = \frac{0.055 + 0.059}{2} = 0.057$$

$$M_x = 0.0885 \times 8.25 \times 4.131^2 = 12.5 \text{ kNm}$$

$$M_y = 0.057 \times 8.25 \times 4.131^2 = 8.1 \text{ kNm}$$

Depth check

$$d_{\text{reqd}} = \sqrt{\frac{M_u}{R_u \cdot b}} = \sqrt{\frac{12.5 \times 10^6}{2.76 \times 1000}}$$

$$= 68 \text{ mm} < d_{\text{assumed}}$$

hence, adopt $D = 150 \text{ mm}$

Area of steel

(i) shorter direction $M_x = 12.5 \times 10^6 \text{ Nmm}$

$$M_u = 0.67 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} b d} \right]$$

$$12.5 \times 10^6 = 0.87 \times 415 A_{st} \times 131 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 131} \right]$$

$$A_{st} \approx 275 \text{ mm}^2$$

$$A_{st \text{ min}} = 0.12\% = \frac{0.12 \times 1000 \times 150}{100} = 180 \text{ mm}^2 < 275 \text{ mm}^2$$

$$\text{spacing of } 8\text{mm diameter bars} = \frac{50.3 \times 1000}{275} = 182 \text{ mm}$$

Hence, provide 8mm bars @ 180mm c/c in shorter direction and bending alternate bars at the $\frac{L}{7} = \frac{4.131}{7} \approx 0.6 \text{ m}$ from the centre of each support

(ii) longer direction $M_y = 8.1 \text{ kNm}$

Effective depth in longer direction = $131 - 8 = 123 \text{ mm}$

$$8.1 \times 10^6 = 0.87 \times 415 \times A_{st} \times 123 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 123} \right]$$

$$A_{st} \approx 190 \text{ mm}^2$$

$$\text{spacing of } 8\text{mm diameter bars} = 264 \text{ mm}$$

Hence, provide 8mm bars @ 250 mm c/c in longer direction and bending half of the bars at $\frac{6.131}{7} \approx 780 \text{ mm}$ from centre of support

check for shear

$$V_u = \frac{w_u L_x}{2} = \frac{8.25 \times 4}{2} = 16.5 \text{ kN}$$

$$\tau_v = \frac{16.5 \times 1000}{1000 \times 123} = 0.14 \text{ N/mm}^2$$

$$\tau_c = 0.25 \text{ N/mm}^2 \quad \text{For minimum } A_{st}$$

$k = 1.3$ for 150 mm thick slab

$$\therefore k \cdot \tau_c = 1.3 \times 0.28 = 0.36 \text{ N/mm}^2$$

Hence $\tau_v < k \cdot \tau_c$, the beam is safe in shear

Check for Development Length

Let the slab has a bearing of 200 mm at the ends. The area of steel available at support is

$$A_{st1} = \frac{1000 \times 50.3}{260} = 193 \text{ mm}^2$$

$$x_u = \frac{0.87 f_{fy} A_{st1}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 193}{0.36 \times 20 \times 1000}$$

$$= 9.68 \text{ mm}$$

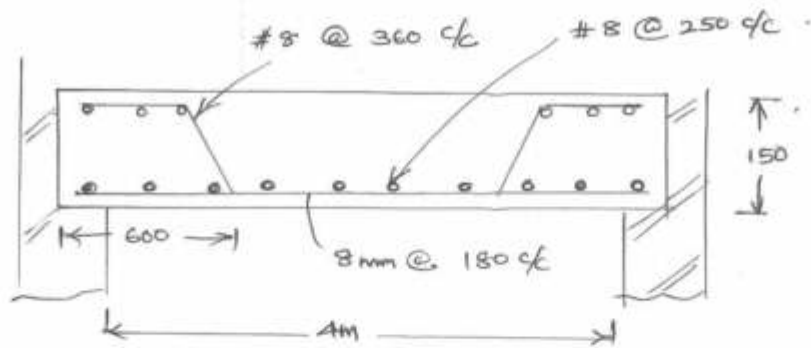
$$\begin{aligned} M_{u1} &= 0.87 f_{fy} A_{st1} (d - 0.42 x_u) \\ &= 0.87 \times 415 \times 193 (131 - 0.42 \times 9.68) \\ &= 8.85 \times 10^6 \text{ Nmm} \end{aligned}$$

$$l_0 = \frac{l_s}{2} - 2' = \frac{200}{2} - 20 = 80 \text{ mm}$$

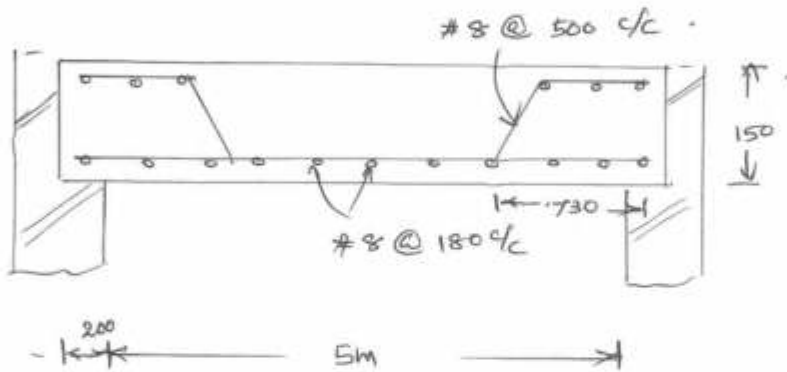
$$L_d = 47 \phi = 47 \times 8 = 376 \text{ mm}$$

$$\frac{1.3 M_{u1}}{V_u} + l_0 = \frac{1.3 \times 8.85 \times 10^6}{13.11 \times 10^3} + 80 \quad 3$$

$= 957 \text{ mm} > L_d$. Similarly check for development length in longer span. Hence, beam is safe in bond.



a) section along short span.



b) section along long span.

OR

VIII (a)

S.No	One Way slab	Two Way Slab
1.	$l_y/l_x > 2.0$	$l_y/l_x \leq 2.0$
2.	The bending take place in one direction only i.e., shorter span.	The bending takes place in both the directions.

3. Depth required is more	Depth required is less
4. Main steel reinforcement is provided along shorter span	Main steel reinforcement is provided along both span
5. Less economical as thickness is more and the amount of steel is also more	More economical as thickness of slab is less and amount of steel required is less

A x 1/2 6

(b) $l_y/l_x = 7.5/3.5 > 2$ hence, it is a one way slab

Assuming total depth = 150 mm

$$d = 150 - 20 - 5 = 125 \text{ mm}$$

(clear cover 20mm and dia. of main bar = 10mm)

Effective span (l)

It should be atleast of the following

i) Centre to centre distance = $8.5 + 0.2 = 8.7 \text{ m}$

ii) clear span + effective depth = $8.5 + 0.125 = 8.625 \text{ m}$

$$\therefore l = 8.625 \text{ m}$$

Design load (kN/m) and factored moment (M_u)

Self wt of slab = $0.15 \times 1.0 \times 25 = 3.750 \text{ kN/m}$

Imposed load = $5 \times 1 = 5 \text{ kN/m}$

Total load (w) = 8.75 kN/m

Design load (kN/m) = $8.75 \times 1.5 = 13.125 \text{ kN/m}$

[Load factor = 1.5]

$$\text{Factored moment} = \frac{W_u \cdot l^2}{8} = \frac{13.125 \times 3.625^2}{8} = 21.6 \text{ kNm}$$

$$M_u = 21.6 \times 10^6 \text{ Nmm}$$

Effective depth required

$$\frac{x_{u\max}}{d} = 0.48$$

$$R_u d = 0.86 f_{ck} \frac{x_{u\max}}{d} \left[1 - 0.42 \frac{x_{u\max}}{d} \right]$$

$$= 0.86 \times 20 \times 0.48 (1 - 0.42 \times 0.48) = 2.76$$

$$d_{\text{required}} = \sqrt{\frac{M_u}{R_u \cdot b}} = \sqrt{\frac{21.6 \times 10^6}{2.76 \times 1000}}$$

$$= 88 \text{ mm} < 125 \text{ mm} \text{ hence OK.}$$

$d > d_{\text{required}}$. Hence, the section is under reinforced

Area of tensil steel (A_{st})

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

$$21.6 \times 10^6 = 45131.25 A_{st} - 7.49 A_{st}^2$$

$$A_{st}^2 - 6025.5 A_{st} + 2.88 \times 10^6 = 0$$

$$A_{st} = \frac{6025.5 \pm \sqrt{(6025.5)^2 - 4 \times 2.88 \times 10^6}}{2}$$

$$= 523.4 \text{ mm}^2$$

Using 10 mm dia bars, $A_\phi = \frac{\pi}{4} \times 10^2 = 78.5 \text{ mm}^2$

$$\text{Spacing of 10 mm dia bars} = \frac{1000 \times A_\phi}{A_{st}}$$

$$= \frac{1000 \times 78.5}{523.4} = 150.17 \text{ mm}$$

∴ Provide 10mm φ @ 150 mm c/c

This spacing is less than

i) $8d = 8 \times 125 = 875 \text{ mm}$

ii) 300 mm

Bending alternate bars at 510 mm $\left[\frac{l}{4} = \frac{3625}{4} = 517 \text{ mm} \right]$
from centre of support or 10 mm from the face of support.

Distribution steel

Distribution reinforcement is provided in the longer direction i.e., 7.5m, = 0.12% of x-sectional area

$$= \frac{0.12}{100} \times 1000 \times 150 = 180 \text{ mm}^2$$

Using 6mm φ bar $A_\phi = \pi/4 \times 6^2 = 28.3 \text{ mm}^2$

$$\begin{aligned} \therefore \text{spacing of 6mm } \phi \text{ bar} &= \frac{1000 \times A_\phi}{A_{st}} \\ &= \frac{1000 \times 28.3}{180} = 157.2 \text{ mm} \end{aligned}$$

∴ Provide 6mm φ @ 150 mm c/c in the longer direction

Check for shear

$$\text{Factored shear force} = V_u = \frac{wL \cdot L}{2}$$

$$V_u = \frac{13.125 \times 3.5}{2} = 22.97 \text{ kN} = 22970 \text{ N}$$

Nominal shear stress, τ_v

$$\tau_v = \frac{V_u}{bd} = \frac{22970}{1000 \times 125} = 0.18 \text{ N/mm}^2$$

Design shear strength of concrete (\bar{I}_c)

$$P_t = \frac{100 \times A_{st}}{bd} = \frac{100 \times 261.5}{1000 \times 125} = 0.21\%$$

From table 5.5, $P_t = 0.21\%$ and M₂₀ concrete

$$\bar{I}_c = 0.28 + \left[\frac{0.36 - 0.28}{0.25 - 0.15} \right] (0.21 - 0.15)$$

$$= 0.328 \text{ N/mm}^2 \quad \therefore \bar{I}_v < \bar{I}_c, \text{ Hence OK}$$

Check for deflection

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 523}{1000 \times 125} = 0.4\%$$

$$f_s = 0.58 f_y \left[\frac{A_{st \text{ reqd}}}{A_{st \text{ provide}}} \right] = 0.58 \times 415 \left(\frac{523}{523} \right) = 240 \text{ N/mm}^2$$

For $P_t = 0.4\%$, $f_s = 240 \text{ N/mm}^2$, $k_t = 1.55$

$$(l/d)_{\max} = 20 \times k_t = 20 \times 1.55 = 31$$

$$(l/d)_{\text{provide}} = 3625/125 = 29$$

$$(l/d)_{\max} > (l/d)_{\text{provided}} \quad \text{hence OK.}$$

Check for development length

Moment of resistance at support by 10mm ϕ bar

$$\text{@ } 300 \text{ mm e/c} = M_1 \quad A_{st} = \frac{523}{2} = 266 \text{ mm}^2$$

$$M_1 = 0.87 f_y A_{st} \cdot d \left[1 - \frac{f_y \cdot A_{st}}{f_{ck} \cdot bd} \right]$$

$$= 0.87 \times 415 \times 266 \times 125 \left[1 - \frac{415 \times 266}{20 \times 1000 \times 125} \right]$$

$$= 11.47 \times 10^6 \text{ Nmm}$$

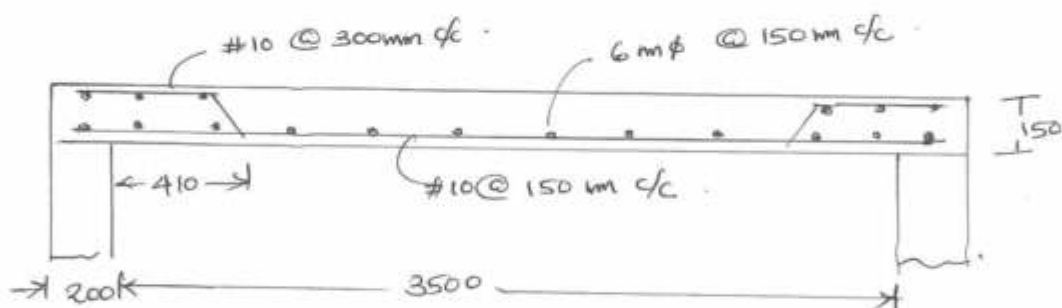
$$V_u = 22970 \text{ N}$$

Providing no hooks $l_0 = 0$

$$\frac{M_1}{V} + l_0 = \frac{11.47 \times 10^6}{22970} = 500 \text{ mm}$$

$$L_d = \frac{(0.87 f_y) \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 10}{4 \times 1.2 \times 1.6} = 470 \text{ mm}$$

$\therefore \frac{M_1}{V} + l_0 > L_d$ Hence, codal requirements are satisfied. The arrangement of reinforcement is shown below.



MODULE IV

(a) A column is defined as a vertical compression member which is mainly subjected to axial loads and the effective length of which exceeds three times its least lateral dimensions.

Columns are classified based on different criteria as

1. shapes of cross-section
2. Materials of construction
3. Types of loading
4. Slenderness ratio
5. types of lateral reinforcement.

(b)

$$b = 450 \text{ mm}$$

$$d = 600 \text{ mm}$$

$$L = 3 \text{ m}$$

$$P = 2000 \text{ kN}$$

$$f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 415 \text{ N/mm}^2$$

$$l_{\text{eff}} = 1.0L = 1.0 \times 3 = 3 \text{ m} = 3000 \text{ mm}$$

$$\text{slenderness ratio} = \frac{\text{Effective length}}{\text{Least lateral dimension}} = \frac{3000}{450} = 6.67 < 12$$

Hence, it is a short column

Minimum eccentricity (e_{min})

For $d = 600 \text{ mm}$

$$e_{\text{min}} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{600}{30} = 26 \text{ mm} > 20 \text{ mm}$$

$$\therefore e_{\text{min}} = 20 \text{ mm}$$

$$0.05 D = 0.05 \times 600 = 30.0 \text{ mm} > e_{\text{min}}$$

For $b = 450 \text{ mm}$

$$e_{\text{min}} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{450}{30} = 21 \text{ mm} > 20 \text{ mm}$$

$$\therefore e_{\text{min}} = 20 \text{ mm}$$

$$0.05 b = 0.05 \times 450 = 22.5 \text{ mm} > e_{\min}$$

$$e_{\min} = 20 \text{ mm}$$

2

Hence, it is designed as a short axially loaded column

Factored load, $P_u = 0.5 P$

$$= 1.5 \times 2000 = 3000 \text{ kN or } 3000 \times 10^3 \text{ N}$$

Area of longitudinal steel (A_{sc})

$$A_g = 450 \times 600 = 270000 \text{ mm}^2$$

$$A_c = A_g - A_{sc} = 270000 - A_{sc}$$

For an axially loaded short column

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

$$3000 \times 10^3 = 0.4 \times 20 (270000 - A_{sc}) + 0.67 \times 415 \times A_{sc}$$

$$270000 - 0.05 A_{sc} = 840000$$

$$A_{sc} = 3110.5 \text{ mm}^2$$

$$\text{Percentage of reinforcement} = \frac{A_{sc}}{A_g} = \frac{3110.5}{270000} = 1.15\%$$

3

It is between 0.8 to 4% hence OK

Using 4 - 25 mm ϕ bars = 4 x 490.6 = 1962.5 mm²

and 4 - 20 mm ϕ bars = 4 x 314 = 1256

$$A_{sc} \text{ Provided} = 3218.5 \text{ mm}^2 > 3110.5 \text{ mm}^2$$

Lateral ties

The diameter of lateral ties should be more than

i, $\frac{1}{4} \times 25 = 6.25$

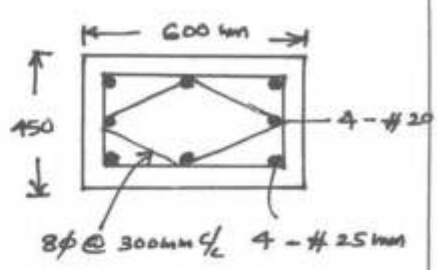
ii, 6 mm using 8 mm dia ties

The pitch of ties should not exceed the following

i, Least lateral dimension = 450 mm

ii, $16 \times 20 = 320$ mm

iii, 800 mm



∴ Provide 8mm ϕ ties @ 300 mm c/c

The arrangement of reinforcement is shown in fig below.

OR

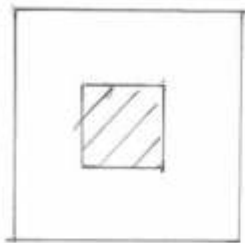
X
(a)

Footings are an important part of foundation construction. They are made of concrete with rebar reinforcement that has been poured into an excavated trench.

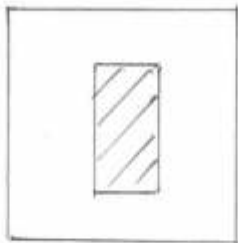
Classification of footing :

- | | |
|--|--------------------------|
| (i) Isolated footing : | (ii) Combined footing : |
| (a) Square footing | (a) Rectangular |
| (b) Rectangular footing | (b) Oval |
| (c) Circular footing | (c) Trapezoidal |
| (iii) Continuous or wall footings or strip footing | |
| (iv) Strap footings | (v) Raft or mat footings |
| (vi) Pile footings | (vii) well foundations. |

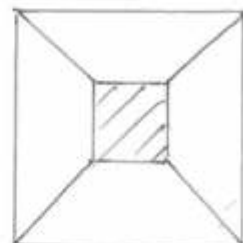
The purpose of footing is to support the foundation and prevent settling.



a) square footing

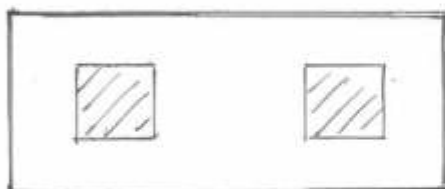


b) Rectangular footing

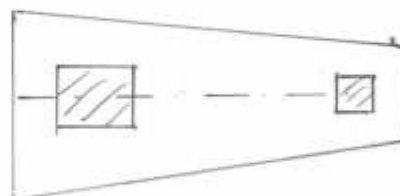


c) Square column with sloping footing

isolated footing:

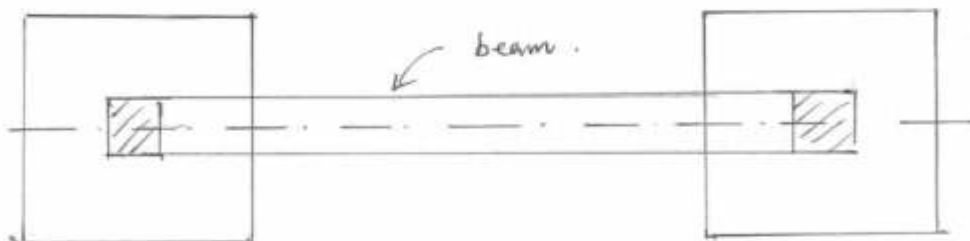


(a) Rectangular



b) Trapezoidal

combined footing



strap footing

2

6

(b)

$$W_c = 850 \text{ kN}$$

$$\text{Bearing capacity} = 190 \text{ kN/m}^2$$

$$f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 415 \text{ N/mm}^2$$

Load calculation

$$W_c = 850 \text{ kN}$$

Self weight of footing, $W_f = 10\%$ of $W_c = 85 \text{ kN}$

$$W_c + W_f = 850 + 85 = 935 \text{ kN}$$

$$\text{Area of footing, } A = \frac{W_c + W_f}{q_u} = \frac{935}{190} = 4.92 \text{ m}^2$$

$$\text{Side of square footing} = \sqrt{4.92} = 2.22 \text{ m say } 2.25 \text{ m}$$

Factored soil pressure due to column load only

$$P_u = \frac{1.5 \times 850}{2.25 \times 2.25} = 251.85 \text{ kN/m}^2$$

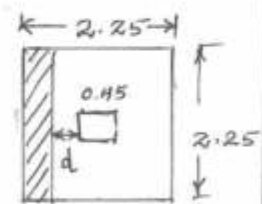
2

Depth of footing by one way shear criterion

Critical section shall be at a distance

d from the face of the column.

Shear force due to factored soil pressure at critical section



$$= 2.25 \left[\frac{2.25 - 0.45}{2} - d \right] \times 251.85$$

$$= 566.66 (0.9 - d) \quad \text{--- (i)}$$

Assuming 0.2% steel, $\bar{I}_c = 0.32 \text{ N/mm}^2$ from table

Shear force resisted by the section = $\bar{I}_c \times d$

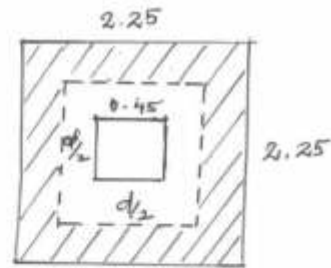
$$= \frac{0.32 \times 10^6}{10^3} \times 2.25 \times d = 720 d \quad \text{--- (ii)}$$

Equating (i) and (ii) we get :

$$566.66 (0.9 - d) = 720 d$$

$$1286.66 d = 509.99$$

$$d = \frac{509.99}{1286.66} = 0.396 \text{ m}$$



[A]

Depth of footing by two way shear (from above fig.)

Considering critical section is at $d/2$ from face of column

$$\text{Perimeter of critical section} = 4(0.45 + d) = 1.80 + 4d$$

$$\text{Shear force at critical section} = 251.85 \times [2.25 \times 2.25 - (0.45 + d)^2]$$

$$= 1274.99 - 251.85(0.2025 + d^2 + 0.9d) \quad \text{--- (iii)}$$

Shear force resisted by the critical section

$$\text{Max allowable shear stress} = 0.25 \sqrt{f_{ck}}$$

$$= 0.25 \sqrt{20} = 1.118 \text{ N/mm}^2 = 1118 \text{ kN/m}^2$$

$$\text{Shear force resisted} = 1118(1.80 + 4d) \times d$$

$$= 2012.4d + 4472 d^2 \quad \text{--- (iv)}$$

Equating (iii) and (iv)

$$1274.99 - 251.85(0.2025 + d^2 + 0.9d) = 2012.4d + 4472 d^2$$

$$d^2 - 0.423 d - 0.29 = 0$$

$$d = \frac{-0.423 \pm \sqrt{(-0.423)^2 \div 4 \times (-0.29)}}{2}$$

$$= \underline{0.367 \text{ m}}$$

[B]

Depth of footing by bending moment criterion critical section is at the face of column.

Bending moment at the critical section.

$$M_u = 251.85 \times 2.25 \times \left[\frac{2.25 - 0.45}{2} \right] \times \left[\frac{2.25 - 0.45}{4} \right]$$

$$= 566.66 \times \frac{1.80^2}{8} = 229.498 \text{ kN} = 229.498 \times 10^6 \text{ Nmm} \quad \text{(v)}$$

Moment of resistance at critical sections :

$$\frac{x_{u\max}}{d} = 0.48 \text{ and } R_u = 2.76 \text{ for M20 concrete and Fe 415 steel}$$

$$M_{u\text{lim}} = R_u \times b d^2 = 2.76 \times 2250 \times d^2 = 6210 d^2 \quad \text{(vi)}$$

Equating (v) and (vi), we get

$$229.498 \times 10^6 = 6210 d^2$$

$$d = 192.24 \text{ mm} = 0.192 \text{ m} \quad \text{[c]}$$

From [A], [B], [C] the highest value of d obtained is 0.896 m. Let us adopt, $d = 400 \text{ mm}$

$$\text{overall depth} = 400 + 8 + 50 = 458 \text{ say } 460 \text{ mm}$$

Area of steel reinforcement

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

$$229.498 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{2250 \times 400 \times 20} \right]$$

$$\text{or } A_{st}^2 - 43369.87 A_{st} + 68918818.32 = 0$$

$$A_{st} = \frac{48369.87 \pm \sqrt{48369.87^2 - (4 \times 68918318.32)}}{2}$$

$$= 1652.03 \text{ mm}^2$$

minimum reinforcement required = $\frac{0.12 \times 2250 \times 460}{100}$

$$= 1242 \text{ mm}^2 < 1652.03 \text{ mm}^2$$

Using 16 bars, $A_{\phi} = \pi/4 \times 16^2 = 201 \text{ mm}^2$

spacing = $\frac{201 \times 2250}{1652.03} = 273 \text{ mm}$

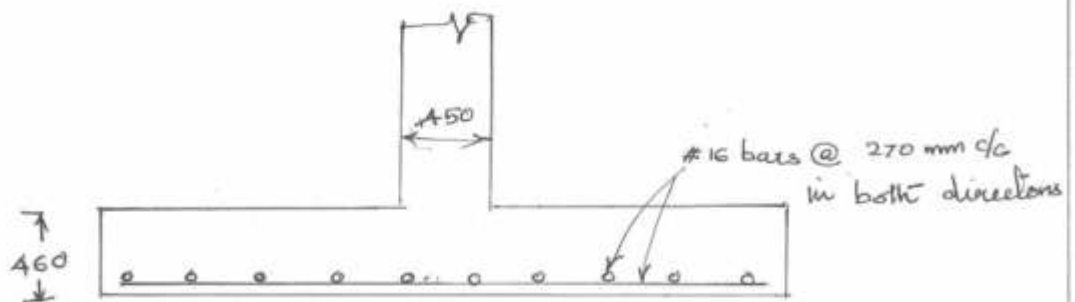
Provide 16 ϕ bars @ 270 mm ϕ in each direction.

check for development length

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 16}{4 \times 1.92} = 752.2 \text{ mm}$$

Available length of bars = $\frac{2250 - 0.45 \times 1000}{2} = 900 \text{ mm}$

hence OK.



Sectional elevation.