Version - B.

[15] 5015

Question no:			Split Score	Total Score

SCHEME OF VALUATION

STRUCTORAL DESIGN - I

PART - A

I 1. A concreate is said to be workable if it is easily transported, placed, compacted and finished without any segregation. 2

2. Bond refers to the adhesion between concrete and steel which resist the slipping of steel ~~bar~~ bar from the concrete. It is responsible for transfer to stresses from steel to concrete and thereby providing composite action on steel. 2

3. Unrestrained slabs - The slabs in which corners are not prevented from lifting are called unrestrained slabs or slabs with corners not held down. 2

4. Slenderness ratio, $\lambda = l_{eff} / r_{min}$ 2

5. The footings which are provided under single columns are called isolated footing. 2 10

(A)

PART- B

II

1. • Balanced section – If the area of reinforcement is such that the strain in the 2 materials reaches maximum simultaneous the beam is said to be balanced section. The mode of failure is the sudden failure could occur with less alarming deflection ie., Compression failure and nature is brittle. Here $x_u = x_{umax}$. 2

• Under-reinforced section – The area of tension steel is less than that required for the balanced section. Here $x_u < x_{umax}$. Mode of failure of such section before collapse will deflect much and crack will be wider ie, there will be sufficient alarm before failure. The failure is called tension failure or ductile failure. 2

• Over-reinforced section – The area of steel provided will be more than that required in balanced section. Here $x_u > x_{umax}$ hence concrete attains maximum value first and steel is well defined. The failure will be due to crushing of concrete so it will be ~~due~~ sudden called compression failure or brittle failure. 2 6

2. (a) Plane sections normal to the axis remain plane after bending. It means that the strain at any point in

the cross-section is propoutional to the distance from the neutral point axis.

(b) The maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending.

(c) The relationship between the stress-strain distribution in Concrete is assumed to be parabolic

(d) The tensile strength of the concrete is ignored

(e) The stresses in the reinforcement are taken from the stress-strain curve for the type of steel.

(f) The maximum strain in the tension reinforcement in the section at failure shall not be less than : $\frac{f_y}{1.15 E_s} + 0.002$ $4 \times 1\frac{1}{2}$ 6

3. The following 3 types of shear reinforcement are used :

i) Vertical stirrups — These are the steel bars vertically placed around the tensile reinforcement at suitable spacing along the length of the beam. Their diameter varies from 6mm to 16mm. The free ends of the stirrups are anchored in the compression zone of the beam to the achor bars or the compressive reinforcement. 2

ii) Bent up bars along with stirrups — These bent up bars resist diagonal tension. Equal no. of bars are to be bent on both sides to maintain symmetry. The bars can be bentup at more than 1 point uniformly along the length of the beam. 2

iii) Inclined stirrups – It is provided generally at $45°$. This for resisting diagonal tension. They are provided throughout the length of the beam. 2 6

4

One Way Slab	Two way slab.
i, $l_y/l_x > 2.0$	$l_y/l_x \leq 2.0$
ii, the bending takes place in one direction only i.e., shorter span	the bending takes place in both the directions.
iii, Depth required is more.	Depth required is less.
iv, Main steel reinforcement is provided along the shorter span.	Main steel reinforcement is provided along both the spans.
v, Less economical as thickness is more and the amount of steel is also more	More economical as thickness is less and amount of steel is less

$4 \times 1\frac{1}{2} = 6$

5. (a) calculate shear force (V_u) at the critical section of the beam
(b) determine nominal shear stress, $\tau_v = V_u/bd$.
(c) Find out the strength of concrete (τ_c)
(d) compare τ_v and $\tau_{c\,max}$, if $\tau_v > \tau_{c\,max}$, design section.
(e) compare τ_v and τ_c : i, if $\tau_v >< \tau_c$, $\frac{A_{sv}}{b \cdot s_v} \geq \frac{0.4}{0.87 f_y}$

ii, if $\tau_v > \tau_c$,

- calculate V_{us}, $V_{us} = V_u - \tau_c bd$
- spacing governed by $\Rightarrow$ $V_s = \frac{0.87 f_y \cdot A_{sv} \cdot d}{s_v}$ 3+3 6
- if bent up bars are also used then $V'_{us} = 0.87 f_y A_{sv} \sin\alpha$

6. $P = 1500\,kN$ $f_{ck} = 25\ N/mm^2$ $f_y = 415\ N/mm^2$

Assuming column to be short and axially loaded

$$P_u = 1.5 \times 1500 = 2250\,kN = 0.4\,f_{ck} \cdot A_c + 0.67\,f_y\,A_{sc}$$

Assuming 1% steel, $A_{sc} = 0.01\,A_g$ and $A_c = A_g - 0.01\,A_g = 0.99\,A_g$

$$2250 \times 10^3 = 0.4 \times 25 \times 0.99\,A_g + 0.67 \times 415 \times 0.01\,A_g$$

$$A_g = 177438\ mm^2 = \pi/4\ D^2$$

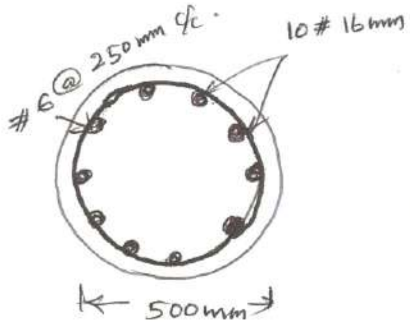
$D = 476\ mm$ so taking $D = 500mm$

$$A_{sc} = \frac{1}{100} \times \frac{\pi}{4} \times 500^2 = 1963\ mm^2$$

Using 16mm diameter bars, no. of bars required $= \frac{1963}{201} = 10.$ 2

Hence provide 10 bars of 16mm diameter

Lateral Ties : Using circular lateral ties



The diameter of ties :

i, $1/4 \times 16 = 4mm$ ii, 6mm

fig 2

Pitch of ties should be atleast of !

i, $D = 500mm$ ii, $16 \times 16 = 256mm$ iii, 300mm

Hence, providing 6mm dia ties @ 250 mm c/c 2 6

7. Different types of Foundation are as follows :

i) Isolated Footings
(a) square footings
(b) Rectangular footing
(c) circular footing

ii) Combined footing : 2
(a) Rectangular
(b) Oval
(c) Trapezoidal.

2

iii) continous or wall footings or strip footings
iv) Strap footings
v) Raft or mat footings
vi) Pile footings
vii) Well foundation. 2 6

PART - C

(Max. marks : 60)

MODULE I

III (a) $f_{ck} = 20 \ N/mm^2$ $\quad f_y = 415 \ N/mm^2$

$b = 250 \ mm$ $\quad d = 400 \ mm$

$A_{st} = 3 \times \pi/4 \times 20^2 = 942.5 \ mm^2$

Depth of neutral axis (x_u)

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

$$= \frac{0.87 \times 415 \times 942.5}{0.36 \times 20 \times 250 \times 400} = 0.4725$$

$x_u = 189 \ mm$. Limiting value of neutral axis depth (x_{umax}) 4

$$\frac{x_{umax}}{d} = 0.48$$

$x_{umax} = 0.48 \times 400 = 192 \ mm$ $\quad \therefore x_u < x_{umax}$

$\therefore$ The beam is under-reinforced. 2 6

(b) $b = 230 \ mm$ $\quad d = 500 \ mm$

$M_u = 200 \ kNm = 200 \times 10^6 \ Nmm$

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

$\frac{d'}{d} = \frac{50}{500} = 0.1$ $\quad f_{sc} = 353 \ N/mm^2$

$$\varepsilon_{sc} = 0.0035\left[1 - \frac{d'}{x_{u\,max}}\right] = 0.0035\left[1 - \frac{50}{240}\right]$$

$$\varepsilon_{sc} = 0.00277$$

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

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

$$x_u = 0.48\, d = 0.48 \times 500 = 240\text{ mm}$$

$$M_{u\,lim} = 0.36 \times 20 \times 230 \times 240\,(500 - 0.42 \times 240)$$

$$= 158658048\text{ Nmm}$$

$$M_{u_2} = M_u - M_{u\,lim} = 200 \times 10^6 - 158658048$$

$$= 41341952\text{ Nmm}$$ 3

Area of tension steel (A_{st}), $A_{st} = A_{st_1} + A_{st_2}$

$$A_{st_1} = \frac{M_{u\,lim}}{0.87\, f_y\,(d - 0.42\, x_{u\,max})}$$

$$= \frac{158658048}{0.87 \times 415\,[500 - 0.42 \times 240]} = 1100.7\text{ mm}^2$$

$$A_{st_2} = \frac{M_{u_2}}{0.87\, f_y\,(d - d')}$$

$$= \frac{41341952}{0.87 \times 415\,(500 - 50)} = 254.2\text{ mm}^2$$ 3

Total area of tension steel $= A_{st_1} + A_{st_2}$

$$= 1100.7 + 254.2 = 1354.9\text{ mm}^2$$

Area of one 20 mm bar $= \pi/4 \times 20^2 = 314\text{ mm}^2$

Number of bar required $= \frac{1354.9}{314} \Rightarrow 4.3$ say 5

Provide 5 – 20 mm diameter bars as compression steel.

Area of compression steel (A_{sc})

$$A_{sc} = \frac{M_{u2}}{f_{sc}(d-d')} = \frac{41341952}{353\,(500-50)} = 260.2\ mm^2$$

∴ Area of one 16 mm dia bar $= \pi/4 \times 16^2 = 201\ mm^2$

No. of 16 mm dia bars reqd. $= \frac{260.2}{201} = 1.3$ say 2

∴ Provide 2 – 16 mm dia bars as compression steel. 3 9

OR

IV (a) $b = 200$ mm $\quad d = 500 - 50 = 450$ mm

$f_{ck} = 20\ N/mm^2 \quad f_y = 250\ N/mm^2$

$A_{sc} = 4 \times \pi/4 \times 25^2 = 1963\ mm^2$

$d'/d = 50/450 = 0.11$, for $\frac{d'}{d}$ and Fe 250,

$f_{sc} = 217.5\ N/mm^2 \quad x_{umax} = 0.53d = 0.53 \times 450$
$= 238.5$ mm

Area of tension steel (A_{st})

A_{st} is calculated by equating total compression and tensile forces. $0.36 f_{ck}\, b.x_u + f_{sc}.A_{sc} = 0.87 f_y . A_{st}$

$0.36 \times 20 \times 200 \times 238.5 + 217.5 \times 1963 = 0.87 \times 250 \times A_{st}$

$A_{st} = 3542.0\ mm^2$ 4

Moment of Resistance (M_u) $\quad M_u = M_{ulim} + M_{u2}$

$M_{ulim} = 0.36 f_{ck}\, b\, x_{umax}(d - 0.42\, x_{umax})$

$= 0.36 \times 20 \times 200 \times 238.5\,(450 - 0.42 \times 238.5) = 120145615$ Nmm

$$M_{u_2} = f_{sc} A_{sc} (d - d') = 217.5 \times 1963 (450 - 50)$$
$$= 170781600$$

$$M_u = 120145615 + 170781000$$
$$= 290926615 \text{ Nmm} = \underline{\underline{290.9 \text{ kNm}}}.$$ 3 7

(b) $b = 200$ mm $\quad d = 400$ mm $\quad L = 5$m

$A_{st} = 3 \times \pi/4 \times 16^2 = 603.19 \text{ mm}^2$

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

Depth of neutral axis (x_u)

$$x_u = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck} \cdot b} = \frac{0.87 \times 415 \times 603.19}{0.36 \times 20 \times 200}$$
$$= 151.2 \text{mm}$$

Limiting depth of neutral axis (x_{umax})

$x_{umax} = 0.48d = 0.48 \times 400 = 192$ mm

$x_{umax} > x_u$ hence, the section is under-reinforced

Moment of resistance (M_u)

$$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 x_u)$$
$$= 0.87 \times 415 \times 603.19 (400 - 0.42 \times 151.2)$$
$$= 73.28 \times 10^6 \text{ N mm} = 73.28 \text{ kNm}$$ 6

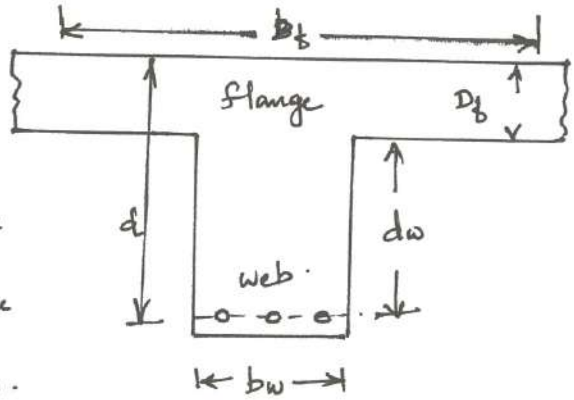
Ultimate load (W_u), Maximum moment $M = \frac{W_u \times l^2}{8}$

$M = M_u \quad \therefore \frac{W_u \times l^2}{8} = 73.28$

$\therefore W_u = \underline{\underline{23.45}}$ kN/m. 2 8

MODULE II

V (a)



bf - breadth of flange

Df - Depth of flange 3

dw - depth of web.

bw - breadth of web. 3 6

(b) b = 250 mm d = 450 m V = 150 kN

$A_{st} = 4 \times \pi/4 \times 18^2 = 1018\ mm^2$

Factored shear force (V_u)

$$V_u = 1.5 \times 150 = 225\ kN = 225000\ N$$

Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{225000}{250 \times 450}$$

$$\tau_v = 2.0\ N/mm^2$$

$\tau_{cmax} = 2.8\ N/mm^2$ $\tau_v < \tau_{cmax}$, hence OK 2

Design shear strength of concrete (τ_c)

$$P_t = \frac{100 . A_{st}}{bd} = \frac{100 \times 1018}{250 \times 450} = 0.9\%$$

For $P_t = 0.97\%$ and M_{20} concrete

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1.00 - 0.75} \times (0.9 - 0.75)$$

$$= 0.596 \ N/mm^2$$

$\tau_v > \tau_c$, hence shear reinforcement is to be provided. 2

Design of shear reinforcement

shear taken by stirrups $= V_{us}$

$V_{us} = V_s - \tau_c bd = 225000 - 0.596 \times 250 \times 450$

$= 157950 \ N$

Using 8mm ϕ 2 legged stirrups,

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

spacing of stirrups, $s_v = \dfrac{0.87 f_y \cdot A_{sv} \cdot d}{V_{us}}$

$$s_v = \frac{0.87 \times 415 \times 100.5 \times 450}{157950} = 103 \ mm$$

Spacing of nominal shear reinforcement

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

check for spacing, spacing of stirrup should be minimum of the following :

i) $0.75d = 0.75 \times 450 = 337$ mm iii) 103 mm

ii) 300 mm iv) 362 mm

Provide 2 legged 8mm ϕ stirrups @ 100mm c/c throughout length 5 9

OR

VI

(a) $b = 250mm$ $D = 500mm$

$d = 500 - 20 - 10 = 470mm$ $l = 5.5m$

A_{st} at support $= 2 \times \pi/4 \times 20^2 = 628\ mm$.

$V = 140kN = 140 \times 10^3\ N$

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

Moment of resistance (M_1) at the section

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

$$= 0.87 \times 415 \times 628 \times 470 \left[1 - \frac{628 \times 415}{250 \times 470 \times 20}\right]$$

$$= 94.748 \times 10^6\ Nmm$$

Check for development length

$$\frac{M_1}{V} + l_o \geq L_d \qquad L_d = \frac{0.87\ f_y\ \phi}{4\ \tau_{bd}}$$

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

$$L_d = \frac{0.87 \times 415 \times 20}{4 \times 1.92} = 940.2mm$$ 4

Providing a $90°$ bend at the centre of support

Anchoring length, $l_o = 8\phi = 8 \times 20 = 160mm$

$$\frac{M_1}{V} + l_o = \frac{94.748 \times 10^6}{140 \times 10^3} + 160 = 836.77 < L_d$$

Hence, codal requirements are not satisfied so there is a need to increase anchorage length.

Providing a U bend at the end of the bar

Anchorage length $= 16\phi = 16 \times 20 = 320$ mm

$$\frac{M_1}{V} + l_0 = \frac{94.748 \times 10^6}{140 \times 10^3} + 320 = 997\text{mm} > L_d$$

$\frac{M_1}{V} + l_0 > L_d$ Hence, Codal requirement are satisfied. 4 8

(b) $b_w = 300$mm $d = 580$ mm

$A_{st} = 8 \times \pi/4 \times 20^2 = 2513 \text{ mm}^2$

$D_f = 120$mm $L = 3.6$m $= 3600$ mm

Effective width of flange (b_f)

$b_f = \frac{l_0}{6} + b_w + 6D_f$ $l_0 = l = 3.6$m

$b_f = \frac{3600}{6} + 300 + 6 \times 120 = 1620$ mm

Clear span of the slab to the left and right of beam

$= 3000 - 300 = 2700$m

$b_f \not> 0.5(L_1 + L_2) + b_w$

$b_f \not> 0.5(2700 + 2700) + 300$

$b_f \not> 3000$ mm $b_f = 1620$ mm

Depth of neutral axis (x_u)

Assuming neutral axis to fall in the flange

$$x_u = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck} b_f} = \frac{0.87 \times 415 \times 2513}{0.36 \times 20 \times 1620}$$

$x_u = 77.8$ mm $< D_f$ Hence N.A lies on flange 4

$x_{umax} = 0.48d = 0.48 \times 580 = 278.4$mm $> x_u$

Hence it is an underreinforced section.

Moment of resistance (M_u)

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

$$= 0.87 \times 415 \times 2513 \times 580 \left[1 - \frac{2513 \times 415}{1620 \times 580 \times 20} \right]$$

$$= 496.61 \times 10^6 \text{ Nmm}$$

$$= 496.61 \text{ kNm.}$$ 3 7

MODULE III

VII (a)

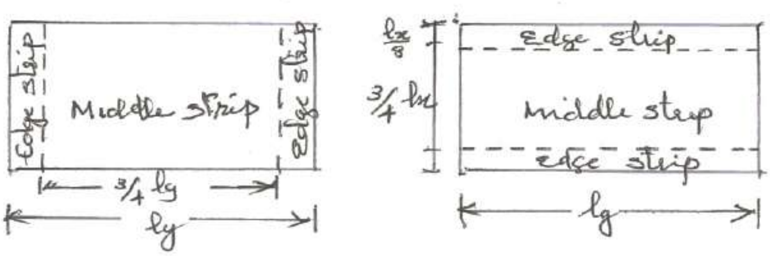
IS code method of design of Restrained two way slab

When the corners of a slab are prevented from lifting, the slab may be designed as specified in below:

1. The maximum bending moments per unit width in a slab are given by the following equations:

   $M_x = \alpha_x w l_x^2$ and $M_y = \alpha_y w l_x^2$

2. Slabs are considered as divided in each directions into middle strips and edge strips, the middle strips being three quarters of the width and each edge strip 1/8 of width



3. The maximum moments calculated as in (i) apply on to the middle strips and no redistribution shall be made.
4. Tension reinforcement provided at mid-span in the middle strip shall extend in the lower part of the slab to within 0.25l of a continous edge, or 0.15l of a discontinious edge
5. Over the continous edge of the middle strip; the tension reinforcement shall extended in the upper part of the slab a distance of 0.15l from the supports and atleast 50 percent shall extend a distance of 0.3l.
6. At a discontinuous edge, negative moments may arise They depend on degree of fixidity at the edge of the slab but, in general, tension reinforcement equal to 50% of that provided at mid span extending 0.1l into the span will be sufficient.
7. Reinforcement in edge strip, parallel to that edge shall comply with minimum given in chapter 9 and the requirement for torsion

8. Torsion reinforcement shall be provided at any corner when the slab is simply supported on both edges meeting at that corner. It shall consist of top and bottom reinforcement, each with layers of bar placed parallel to the sides of the slab and extending from the edges to a minimum distance of $1/5^{th}$ of shorter span. The area of reinforcement in each of these four layers shall be three quarters of the area required for the maximum mid span moment in the slab.
9. Torsion reinforcement equal to half that described in (8) shall be provided at a corner contained by edges over only one which the slab is continuous.
10. Torsion reinforcement need not be provided at any corners contained by edges over both of which slab is continuous.
11. Where l_y/l_x is greater than 2, the slabs shall be designed as spanning one way. 6

(b) Assuming total depth = 120mm

$d = 120 - 20 = 100mm$, assuming effective cover = 20mm

Effective span (l)

It should be atleast of the following:

i) Centre to centre spacing $= 2.5 + 0.23 = 2.73m$

ii) clear span + effective depth = 2.5 + 0.1 = 2.6m

Design load (W_u), factored moment (M_u) and shear force (V_u)

For 1m width of slab

Self weight of slab = $0.12 \times 1 \times 25 = 3\ kN/m^2$

Live load = $5\ kN/m^2$ Total load = $W = 5+3 = 8\ kN/m^2$

Design load = $W_u = W \times 1.5 = 8 \times 1.5 = 12\ kN/m^2$

Factored moment = $M_u = \dfrac{W_u \cdot l^2}{8} = \dfrac{12 \times 2.6^2}{8}$

$M_u = 10.14\ kNm$ or $10.14 \times 10^6\ Nmm$

Factored shear force = $V_u = \dfrac{W_u \cdot L}{2} = \dfrac{12 \times 2.5}{2} = 15\ kN$
$= 15000\ N$

Effective depth required

$x_{umax} = 0.48$

$R_u = 0.36 f_{ck} \dfrac{x_{umax}}{d} \; 1 - \left[\dfrac{0.42\ x_{umax}}{d}\right]$

$= 0.36 \times 20 \times 0.48\ (1 - 0.42 \times 0.48) = 2.76$

$b = 1000mm$.

$d\ reqd = \sqrt{\dfrac{M_u}{R \cdot b}} = \sqrt{\dfrac{10.14 \times 10^6}{2.76 \times 1000}} = 60mm < 100mm$
hence OK

Adopt $D = 120mm$, $d = 100mm$

$d > d\ reqd$. Hence the section is under-reinforced

Area of tension steel (A_{st})

$M_u = 0.87 f_y A_{st}\, d \left[1 - \dfrac{A_{st} f_y}{f_{ck}\, bd}\right]$

$10.14 \times 10^6 = 0.87 \times 415 \times A_{st} \times 100 \left[1 - \dfrac{A_{st} \times 415}{20 \times 1000 \times 100}\right]$

$10.14 \times 10^6 = 36105\ A_{st} - 7.49\ A_{st}^2$

$A_{st}^2 - 4820.4\ A_{st} + 1.35 \times 10^6 = 0$

$$Ast = \frac{4820.4 \pm \sqrt{(4820.4)^2 - 4 \times 1.35 \times 10^6}}{2}$$
$$= 300\ mm^2$$

Using 8mm ϕ bar

Area of one 8mm dia bars, $A\phi = \frac{\pi}{4} \times 8^2 = 50.3\ mm^2$

Spacing of bars $= \frac{1000 \cdot A\phi}{Ast} = \frac{1000 \times 50.3}{300}$
$= 167\ mm$ say $160\ mm$

The spacing should be less than

i) $3d = 3 \times 100 = 300\ mm$

ii) 300 mm

Ast provided $= \frac{1000 \times 50.3}{160} = 314\ mm^2$

As per IS code minimum area of reinforcement to be provided is $= 0.12\ bD/100 = 0.12 \times 1000 \times 120/100$
$= 144\ mm^2 < 300\ mm^2$ hence ok

∴ Provide 8mm ϕ bars @ 160 mm c/c in shorter span.

Bending half the bars at 375 mm $\left[\frac{l}{7} = \frac{2600}{7} = 371\ mm\right]$ from the centre of support or $375 - \frac{230}{2} = 260\ mm$ from the face of the support. Ast at supports is half of that at mid-span. Ast provided at supports $= \frac{314}{2} = 157\ mm^2 > 144\ mm^2$

Distribution steel

Area of distribution steel $= \frac{0.12bD}{100} = \frac{0.12 \times 1000 \times 120}{100}$
$= 144\ mm^2$

Using 6mm ϕ bars, $A\phi = \frac{\pi}{4} \times 6^2 = 28.3\ mm^2$

Spacing $= \frac{1000\ A\phi}{Ast} = \frac{1000 \times 28.3}{144} = 196.5\ mm$ say 190 mm

∴ Provide 6 ϕ @ 190mm c/c in longer direction.

Check for shear

$V_u = 15000 N$ Nominal shear stress = $\tau_v = \frac{V_u}{bd}$

$$= \frac{15000}{100 \times 100} = 0.15\ N/mm^2$$

At support, $A_{st} = 157\ mm^2$, $P_t = \frac{100\ A_{st}}{bd}$

$$= \frac{100\ A_{st}}{bd} = \frac{100 \times 157}{100 \times 1000} = 0.15\ \%$$

For 120mm depth $k = 1.3$

τ_c for slab $= k \cdot \tau_c = 0.28 \times 1.3 = 0.36\ N/mm^2 > \tau_v$

Check for deflection

$$P_t = \frac{100 \cdot A_{st}}{bd} = \frac{100 \times 314}{100 \times 1000} = 0.31\ \%$$

$$f_s = 0.58\ f_y \left[\frac{A_{st}\ reqd}{A_{st}\ provided}\right] = 0.58 \times 415 \left[\frac{300}{314}\right] = 230\ N/mm^2$$

For $P_t = 0.31\ \%$ $f_s = 230\ N/mm^2$

For $f_s = 190\ N/mm^2$ $P_t = 0.31$ $k_t = 1.89$

For $f_s = 240\ N/mm^2$ $P_t = 0.31$ $k_t = 1.47$

∴ For $f_s = 230\ N/mm^2$.

$$K_t = 1.89 - \frac{(1.89 - 1.47)}{(240 - 190)} \times (230 - 190) = 1.56$$

$$\left(\frac{l}{d}\right)_{max} = 20 \times k_t = 20 \times 1.55 = 31.$$

$$\left(\frac{l}{d}\right)_{provided} = \frac{2600}{100} = 26 < 31$$

$(l/d)_{max} > (l/d)_{provided}$. Hence OK

Check for development length

At support, $M_{u1} = 0.87 f_y \cdot A_{st} d \left[1 - \frac{A_{st} f_y}{bd f_{ck}}\right]$

$= 0.87 \times 415 \times 157 \times 100 \left[1 - \frac{157 \times 415}{100 \times 100 \times 20}\right] = 5483819.9 \text{ Nmm}$

$V_u = 15000 N$

Using no hook $l_o = 0$

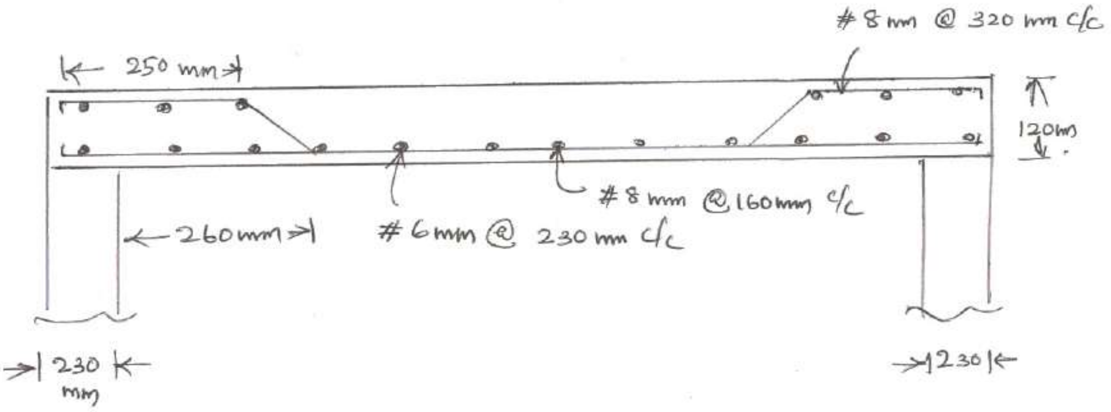
$\frac{M_{u1}}{V_u} + l_o = \frac{5483819.9}{15000} + 0 = 366 \text{ mm}$

$L_d = \frac{\phi(0.87 f_y)}{4 \cdot \tau_{bd}} = \frac{8 \times 0.87 \times 415}{4 \times 1.6 \times 1.2} = 376 \text{mm}$

$\frac{M_{u1}}{V} + l_o \not> L_d$ Hence, codal requirement are not satisfied. Therefore providing a 90° bend at the centre of support. $l_o = 8\phi = 8 \times 8 = 64$ mm

$\therefore \frac{M_1}{V} + l_o = 366 + 64 = 430 \text{ mm} > L_d$ 3

Hence, codal requirement are satisfied. The arrangement of reinforcement is given in Fig 8.4.



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OR

VIII (a)

Two way slab are those slabs which are supported on all the four edges and having $l_y/l_x \leq 2.0$. In these type of slabs bending occurs in both the direction i.e., longer direction as well as shorter direction. There are two kinds of two way slabs : 2

(a) Restrained slabs – When a two way slab is loaded, the corners get lifted up. These corners can be prevented from lifting by providing fixidity at the supports by beam or walls. Such type of slabs in which the corners are prevented from lifting are called as the restrained slabs. In these slabs special torsion reinforcement is provided at the edges to prevent cracking of the corners. These are also called as slabs with corners held down.

The Indian Standard specifies two ways of providing fixidity in the case of restrained slabs :

i) slab is simply supported on 4 edges by beams and the beams and slabs are cast monolithic (at the same time) 2

ii) slab is supported on the edges by the four walls and there are superstructure walls also above the slabs

(b) Unrestrained slab – Slab in which corners are not prevented from lifting are called as unrestrained slabs or slabs with corners not held down. 2 6

(b) $l = 1.25\,m$ Imposed load $= 1000\ N/m^2 + 800\ N/m^2$
$= 1800\ N/m^2$

$f_{ck} = 20\ N/mm^2$ $f_y = 415\ N/mm^2$
$D = 150\,mm$ $d = 150 - 20 - 5 = 125$

Design load (W_u) and factored moment (M_u)

self wt. of slab $= 0.15 \times 1 \times 25000 = 3750\ N/m$
Imposed load $= 1800 \times 1 = 1800\ N/m$
Total load, $w = 5550\ N/m$
Designed load $= W \times \gamma_f = 5550 \times 1.5$
$W_u = 8325\ N/m$

Factored moment $= M_u = \dfrac{W_u \cdot l^2}{2} = \dfrac{8325 \times 1.25^2}{2}$
$M_u = 6504\ Nm = 6.504 \times 10^6\ Nmm$

Factored shear force, $V_u = W_u \cdot l = 8.325 \times 1.25 = 10406.25\,N$

Minimum effective depth required, $\dfrac{x_{umax}}{d} = 0.48$

$R_u = 0.36 f_{ck} \dfrac{x_{umax}}{d}\left[1 - 0.42 \dfrac{x_{umax}}{d}\right]$
$= 0.36 \times 20 \times 0.48 (1 - 0.42 \times 0.48) = 2.76$

$d\ reqd = \sqrt{\dfrac{M_u}{R_u \cdot b}} = \sqrt{\dfrac{6.504 \times 10^6}{2.76 \times 1000}}$

$d\ reqd = 49\,mm < 125\,mm$ Hence ok

∴ The section is under reinforced. 3

Area of steel (Ast)

$$Mu = 0.87\, f_y \cdot Ast\, d \left[1 - \frac{f_y\, Ast}{bd\, f_{ck}}\right]$$

$$6.504 \times 10^6 = 0.87 \times 415 \times 125\, Ast \left[1 - \frac{415\, Ast}{1000 \times 125 \times 20}\right]$$

$$6.504 \times 10^6 = 45131.25\, Ast - 7.49\, A_{st}^2$$

$$A_{st}^2 - 6025.5\, A_{st} + 868357.8 = 0$$

$$Ast = \frac{6025.5 \pm \sqrt{(6025.5)^2 - 4 \times 868357.8}}{2}$$

$$= 148\ mm^2$$

This area of steel should be more than minimum area of reinforcement i.e., 0.12% of x-sectional area

$$= \frac{0.12\, bD}{100} = \frac{0.12 \times 1000 \times 150}{100} = 180\ mm^2 > 148\ mm^2$$

Providing $Ast = 180\ mm^2$ Using 8mm ϕ bar $A_\phi = \frac{\pi}{4} \times 8^2$

$= 50.3\ mm^2$

Spacing of 8mm bar $= \frac{1000 \times 50.3}{180} = 279$ mm say 270mm

Maximum spacing is limited to

i) $3d = 3 \times 100 = 300$ mm

ii) 300mm

$\therefore$ Provide 8mm ϕ @ 270mm c/c as main steel.

Also providing 8mm ϕ @ 270mm c/c in other directions also as distribution reinforcement.

Check for shear

$V_u = 4617.2\,N$

Ast provided $= \dfrac{1000 \times 50.3}{270} = 186.3\,mm^2$

Nominal shear stress, $\tau_v = \dfrac{V_u}{bd} = \dfrac{10406.25}{1000 \times 125} = 0.08\,N/mm^2$

Design shear strength of concrete (τ_c),

$P_t = \dfrac{100 \cdot Ast}{bd} = \dfrac{100 \times 186.3}{1000 \times 125} = 0.15\,\%$

For $P_t = 0.15\%$ and M_{20} concrete, $\tau_c = 0.28\,N/mm^2$

$k = 1.3$. $K \cdot \tau_c = 1.3 \times 0.08 = 0.36\,N/mm^2 >> \tau_v$

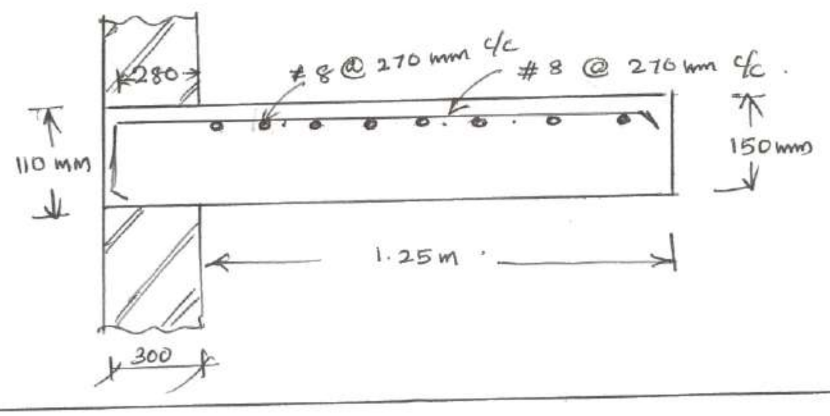
Hence ok.

Checking for development length.

The code requires that the reinforcement must extend at least Ld into support

$L_d = \dfrac{\phi\,(0.87\,f_y)}{4\,\tau_{bd}} = \dfrac{8 \times 0.87 \times 415}{4 \times 1.6 \times 1.2} = 376\,mm$

∴ extending bars into the support after giving a 90° bend for a distance 390 mm (280 + 110) as shown in fig. 8.6 3



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check for deflection

$P_t = 0.15\%$ $f_s = 0.58 f_y \left[\frac{Ast\ reqd}{Ast\ provided}\right]$

$= 0.58 \times 415 \times \left[\frac{180}{193.5}\right] = 223\ N/mm^2$

For $P_t = 0.15\%$ and $f_s = 223\ N/mm^2$

$k_t = 2$ $\left(\frac{l}{d}\right)_{max} = 7 \times 2 = 14$ $\left(\frac{l}{d}\right)_{provd} = \frac{1250}{125} = 10$

$\left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$ hence OK

MODULE IV

IX

(a) The transverse reinforcement is provided along the lateral directions of the column in the form of ties or spirals enclosing the main steel. The functions of transverse steel are as following :

i) To hold the longitudinal bars in position

ii) To prevent buckling of the main longitudinal bars

iii) To resist diagonal tension caused due to transverse shear developed because of any moment or load

iv) To impart ductility to the column

v) To prevent longitudinal splitting or bulging out of concrete by confining it in the core. $4 \times 1\frac{1}{2}$ 6

(b) L = 3m P = 1200 kN D = 400mm

$f_{ck} = 25\ N/mm^2$ $\quad f_y = 415\ N/mm^2$

Effective length $l_{eff} = 1l$

Slender ratio $= \frac{l}{d} = \frac{3000}{400} = 7.5 < 12$

Hence it is a short column.

Minimum eccentricity (e_{min})

$e_{min} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{400}{30} = 19.33mm > 20mm$

$e_{min} = 20mm$

$\frac{e_{min}}{D} = \frac{20}{400} = 0.05$ ∴ It is designed as axially loaded column.

Area of steel (A_{sc})

$P_u = 1.5 \times P = 1.5 \times 1200 = 1800\ kN = 1800 \times 10^3 N$

For a column with helical ties, the load carrying capacity is given by $P_u = 1.05\ (0.4 f_{ck} A_c + 0.67 f_y A_s)$

$A_g = \pi/4 \times 400^2 = 125663.7\ mm^2$

$A_c = A_g - A_{sc} = 125663.7 - A_{sc}$

$1800 \times 10^3 = 10.5\ [0.4 \times 25 \times (125663.7 - A_s) + 0.67 \times 415 \times A_{sc}]$

$268.05\ A_{sc} = 480531.15 = 1793 mm^2$

Percentage of steel $= \frac{1793 \times 100}{125663.7} = 1.43\%$

It is between 0.8 and 4% hence ok

Using 20mm dia bar $A\phi = 314\ mm^2$

No. of bars reqd $= \frac{1793}{314} = 4.7$ say 6

$\therefore$ Provide 6–20 mm dia bars (A_{sc} provided $= 6 \times 314$
$= 1884\ mm^2$)

Helical reinforcement, core diameter, $D_c = 400 - 2 \times 50 = 300mm$ 4

Area of core $= \pi/4 \times 300^2 - 1884 = 6880.8\ mm^2$

Assuming pitch $= p$

Volume of core per pitch $= 6880.8 \times p$

Using 8mm ϕ spiral

Volume of one spiral per pitch

$= \pi/4 \times 8^2 \times \pi\ (300 - 8)$

$= 46110.8\ mm^2$

$$\frac{\text{volume of helical reinforcement}}{\text{volume of core}} = \frac{46110.8}{68801.8p}$$

As per IS code

$$\frac{46110.8}{68801.8p} \leq 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y}$$

$$\leq 0.036\left[\frac{125663.7}{68801} - 1\right] \frac{\times 25}{415}$$

$p \geq 37mm$

Maximum pitch. It should not be more than 75mm or

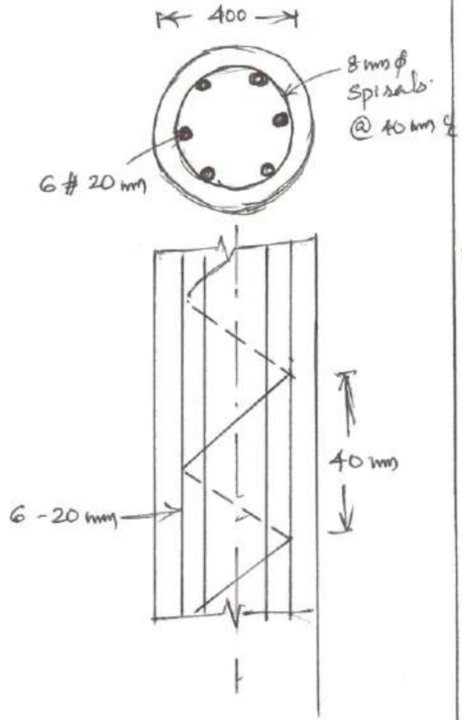
$\frac{\text{Core diameter}}{6}$ ie., $\frac{300}{6} = 50mm$

Minimum Pitch :

i) 25mm

ii) 3 x dia of helical reinforcement $= 3 \times 8 = 24mm$

∴ Provide of 8mm φ spirals @ 40mm c/c 3



2

OR

X

(a) Flight – is the length of the stair bet. two landings

Landing – Intermediate horizontal portion

Rise – The vertical height of a step varies from 150mm to 180mm

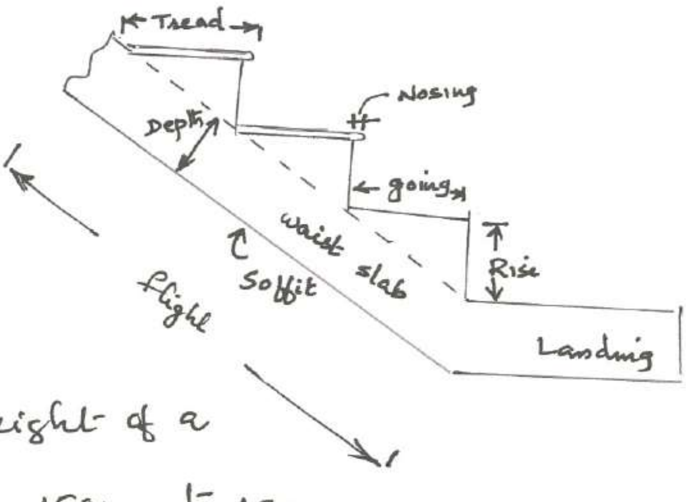


fig (3)

Tread – Horizontal distance bet. two risers on a step.

Going and Nosing – Horizontal distance between two risers is known as going. Projecting out portion from riser surface is called Nosing.

Soffit :– It is the bottom surface of waist slab. (3) 6

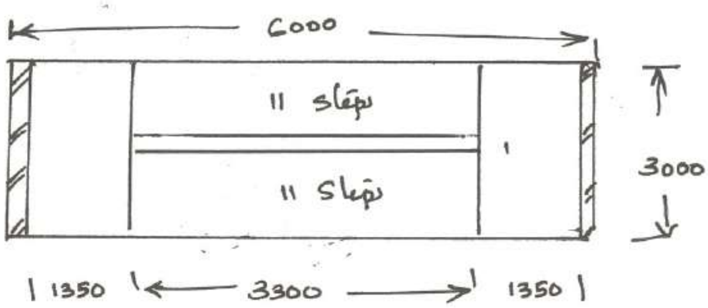
(b) Available width of stair case = 3m

considering 2 flight of dog logged stair case, let us assume width of each flight as 1.35m

Space between the two flights $= 3.0 - 2\times1.35 = 0.30m$

Floor to floor height = 3.5m

As there will be 2 flights, each flight will have a height of $\frac{3.50}{2} = 1.75$ m



Assuming height of risers as 150mm as it is public

Number of risers $= \frac{1750}{150} = 11.66$ say 12

No. of treads required = No. of risers − 1 = 12 − 1 = 11

Let width of each tread = 300mm

Total going $= 300\times 11 = 3300$ mm = 3.30 m

Total length available = 6m

∴ width of each landing $= \frac{6-3.30}{2} = 1.35m = 1350mm$ 3

Design of stair case ; Effective span of ~~the~~ flight = centre to centre distance of wall $= 6 + \frac{0.23}{2} + \frac{0.23}{2} = 6.23m$

Thickness of waist slab = 1/20 of span (approx)

$= \frac{6.23}{20} \times 1000 = 311.5$ mm

Let us take d = 300mm and D = 325mm

Loads :

i, Weights of waist slab in plan (per m width of flight)

$= D\sqrt{1+\frac{R^2}{T^2}} \times 25 = 0.325\sqrt{1+\frac{150^2}{300^2}} \times 25$

$= 9.1$ kN/m.

ii, Weight of steps $= \frac{25\,RT}{2T} = \frac{1}{2} \times \frac{0.15 \times 0.30}{0.30} \times 25$

$= 1.875$ kN/m

Total dead load = 9.1 + 1.875 = 10.975 kN/m

Live Load = 5 kN/m² = 5 kN/m per m width of stair case

Total DL + LL = w = 10.975 + 5 = 15.975 kN/m

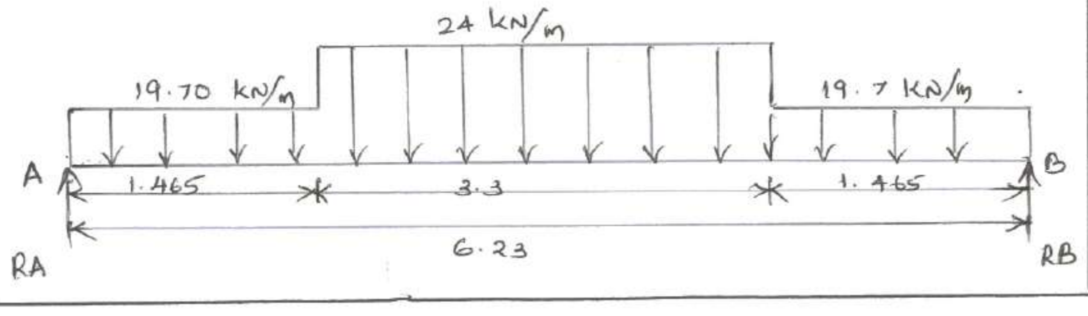
Factored load, w_u = 16 × 1.5 = 24 kN/m

DL = 0.325 × 25 × 1.0 = 8.125 kN/m

LL = 1 × 5 kN/m² = 5 kN/m²

Total DL + LL = 13.125 kN/m Factored Load = 19.7 kN/m

Load diagram of the stair shall be as follows :



Design moment, Reaction of support, $R_A = R_B$

$$= \frac{(2\times 19 - 70\times 1.465) + (24 \times 3.30)}{2} = 68.5 \text{ kN}$$

BM at mid span, $M_u = \left(68.5\times\frac{6.23}{2}\right) - \left[19.70 \times 1.465 \times \left(\frac{1.465 + 3.30}{2}\right)\right] - \left[24 \times \frac{3.30}{2} \times \frac{3.30}{4}\right] = 112 \text{ kNm}$

$M_{u\,lim} = 0.138\ f_{ck}\ bd^2 = 0.138 \times 20 \times 1000 \times 300^2 \times \frac{1}{10^6}$

$= 2484 \text{ kNm} > 112 \text{ kNm}$

Hence the section can be designed as singly reinforced

Area of reinforcement

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

$112\times 10^6 = 0.87 \times 415 \times A_{st} \times 300 \left[1 - \frac{A_{st}}{1000 \times 300} \times \frac{415}{20}\right]$

$= 108315\ A_{st} - 7.49 A_{st}^2$

$A_{st}^2 - 14461.28\ A_{st} + 14.9\times 10^6 = 0.$

$A_{st} = \frac{14461.28 \pm \sqrt{(14461.28)^2 - 4 \times 14.9\times 10^6}}{2}$

$= 1117 \text{ mm}^2$

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

Spacing $= \frac{201}{1117} \times 1000 = 179 \text{mm}$

Provide 16 mm ϕ @ 170 c/c

Distribution steel = 0.12% of area $= \frac{0.12}{100} \times 1000 \times 325$

$= 390 \text{mm}^2$

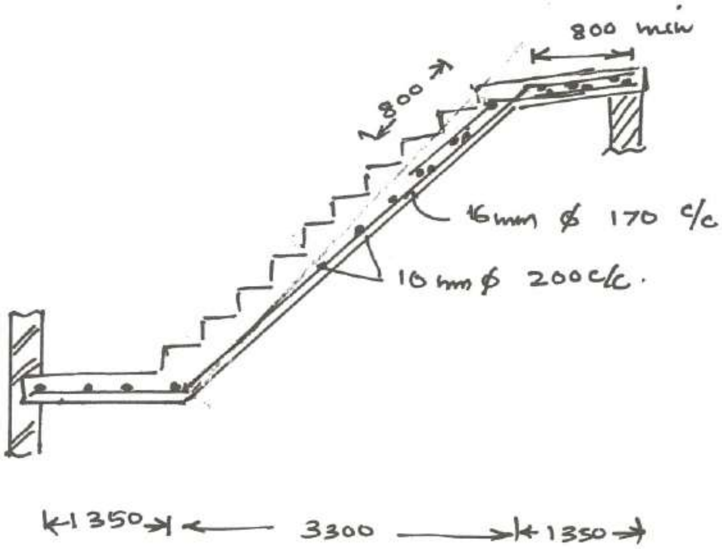
Spacing of 10 ϕ bars $= \frac{78.5}{390} \times 1000 = 201$ mm

Provide 10 ϕ @ 200 mm c/c

Development length

$$L_d = \phi \cdot \frac{(0.87 f_y)}{4 \tau_{bd}} = \frac{16 \times 0.87 \times 415}{4 \times 1.6 \times 1.2} = 752 \text{mm}$$ 4

Providing 800 mm length of bars at points where L_d is required as shown in Figure.



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