

DESIGN OF STAIR CASE WITH CENTRAL STRINGER BEAM

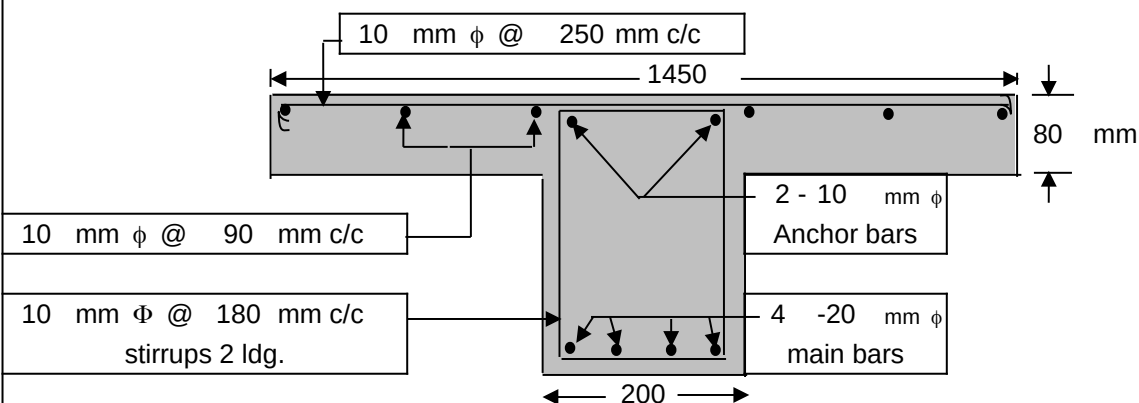
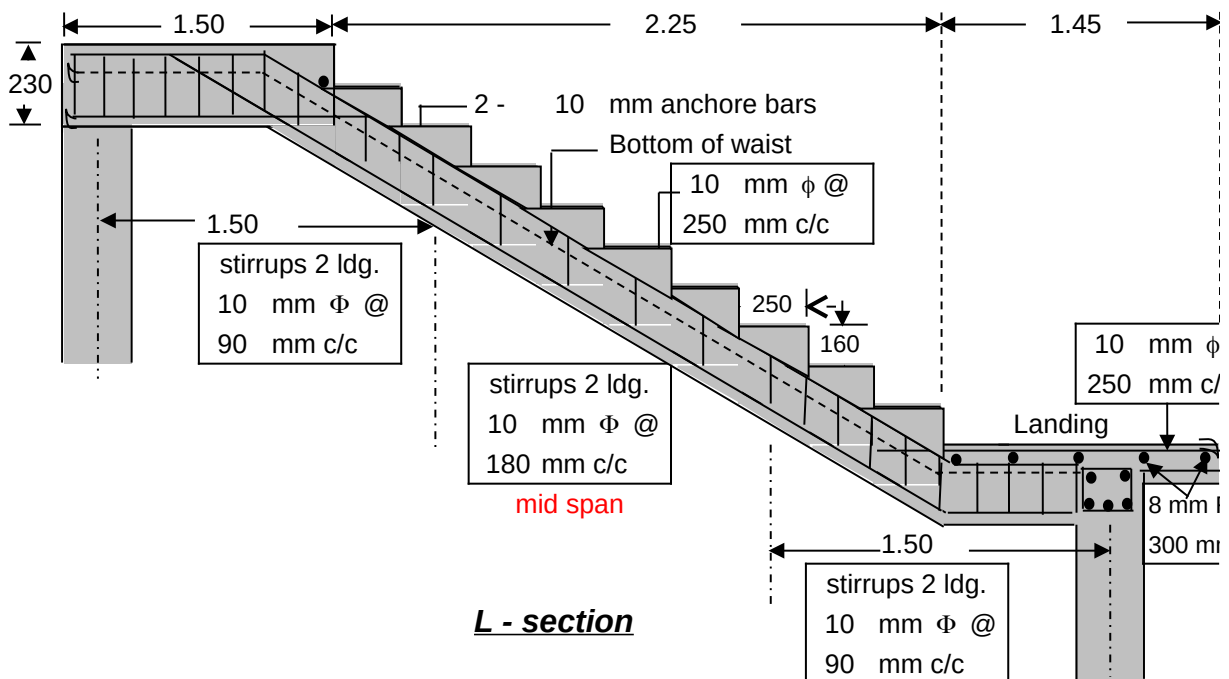
Project name :-

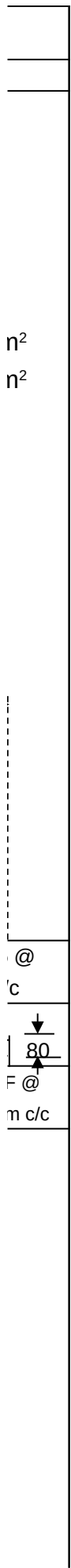
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Space available for staircase	5.20	x	4.40	m
Vertical distance of floor	3.60	m	3600	mm
width of stair case	1.40	mtr	1400	mm
Risers	0.16	mtr	160	mm
Treads	0.25	mtr	250	mm
Live load	4000	N/m ²		
Concrete	M - 20		m	13.33
	wt. 25000	N/mm ³	σ _{cbc}	7 N/mm
Steel	fy 415		σ _{st}	230 N/mm
Nominal cover	25	mm	Effective cover	30
West slab thickness	80	mm		
assume width of stringer beam	200	mm		

Reinforcement

Main Bars	20	mm Φ	Bars Required	4	Nos.
Anchor bars	10	mm Φ	Bars Required	2	Nos.
Waist slab bars	10	mm Φ	Sapcing c/c	250	mm
Strirrups	10	mm Φ	Sapcing c/c	90	mm





DESIGN OF STAIR CASE WITH CENTRAL STRINGER BEAM

width of stair case	1.40 mtr	1400 mm			
Risers	0.16 mtr	160 mm	Treads	0.25 mtr	250
live load	4000 N/m ²				
Concrete	M- 20		Steel	fy- 415	N/mm ²
Nominal cover	25 mm		Effective cover	30 mm	

1 Design Constants:- For HYSD Bars Concrete M = 20

$$\begin{aligned}\sigma_{st} &= 230 \text{ N/mm}^2 & \text{wt. of concrete} &= 25000 \text{ N/mm}^2 \\ \sigma_{cbc} &= 7 \text{ N/mm}^2 \\ m &= 13.33 \\ k &= \frac{m^2 c}{m^2 c + \sigma_{st}} = \frac{13.33 \times 7}{13.33 \times 7 + 230} = 0.289 \\ j &= 1 - k/3 = 1 - 0.289/3 = 0.904 \\ R &= 1/2 \times c \times j \times k = 0.5 \times 7 \times 0.904 \times 0.289 = 0.913\end{aligned}$$

2 Loading on waist slab:-

Assume waist slab thickness = 80 mm

The weight of waist slab on the slope should be multiplied by the factor

$$\begin{aligned}\frac{\sqrt{R^2 + T^2}}{T} \quad \text{where } R &= 160 \text{ mm and } T = 250 \text{ mm} \\ &= \sqrt{\frac{160^2 + 250^2}{250}} = 1.19 \quad \text{to get the equivalent weight of horizontal plane}\end{aligned}$$

Considered 1 m width of slab.

Load per metre horizontal run will be as follows.

Self weight	=	(0.08 x 1 x 1 x 25000) x 1.19	=	2380 N
Weight of steps	=	0.5 x 0.16 x 1 x 1 x ###	=	2000 N
Loading of finishing	=	L.S.	=	100 N
Live load	=		=	4000 N
				Total = 8480 N

The loading on landing will be lesser : however , for simplicity , we will take the same loading through.

3 Design of waist slab:-

the waist slab is supported on central stringer beam . Hence the worst condition may be when we considered concentrated live load of 4000 N to act to one side only.

$$\text{Dead weight} = 2380 + 2000 + 100 = 4480 \text{ N}$$

Assume width of stringer beam = 200 mm

$$\text{Projection of slab beyond the rib of beam} = \frac{1.40 - 0.2}{2} = 0.60 \text{ mtr}$$

$$\text{B.M. due to dead load} = \frac{wL^2}{2} = \frac{4480 \times (0.60)^2}{2} = 806 \text{ N-m}$$

$$\text{B.M. due to U.D. live load} = \frac{wL^2}{2} = \frac{4000 \times (0.60)^2}{2} = 720 \text{ N-m}$$

$$\text{B.M. due to concentrated live load} = 4000 \times 0.60 = 2400 \text{ N-m}$$

$$\text{Max. B.M.} = 806 + 2400 = 3206 \text{ N-m}$$

$$\text{Effective depth required} = \sqrt{\frac{BM}{R_{xb}}} = \sqrt{\frac{3206000}{0.913 \times 1000}} = 60 \text{ mm} \quad \text{But available} = 80$$

$$\text{However, keep minimum total depth} = 80 \text{ mm. Effective depth} = 80 - 25 = 55$$

$$A_{st} = \frac{BM \times 1000}{\sigma_{st} \times j \times D} = \frac{3206000}{230 \times 0.904 \times 55} = 280 \text{ mm}^2$$

$$\text{using } ### \text{ mm } \Phi \text{ bars } A = \frac{3.14 \times d^2}{4 \times 100} = \frac{3.14 \times 10 \times 10}{4 \times 100} = 79$$

$$\text{spacing of Bars} = A \times 1000 / A_{st} = 79 \times 1000 / 280 = 280$$

However, keep spacing = **250** mm, one bar per step

Distribution reinforcement = $1.2 \times 80 = 96 \text{ mm}^2$

using 8 mm Φ bars $A = \frac{3.14 \times \text{dia}^2}{4 \times 100} = \frac{3.14 \times 8 \times 8}{4 \times 100} = 50$

spacing of Bars = $A \times 1000 / A_{st} = 50 \times 1000 / 96 = 523$

Maximum permissible spacing = $45 \times 8 = 360 \text{ mm}$ say 300 mm

However, keep spacing = 300 mm, Maximum

4 Design of stringer beam :-

The stringer beam will act as T- beam. Flight CD is longest,
Hence we will design the stringer beam CD

$$\text{Effective span} = \left[1.50 - \frac{0.2}{2} \right] + 2.25 + \frac{1.45}{2} = 4.38 \text{ m}$$

The loading on stringer beam will be as follows,

assuming the web to be 200 mm wide and 200 mm deep

(a) Weight of rib /m run = $0.20 \times 0.20 \times \text{###} \times 1.19 = 1190 \text{ N}$

(b) Load from waist slab = $8480 \times 1 \times 1.50 = 12720 \text{ N}$

Total = 13910 N say 14000

Assuming partial fixity at ends,

$$M = \frac{wL^2}{10} = \frac{14000 \times 4.38^2 \times 1000}{10} = 26796875 = 26.8 \times 10^6$$

Taking lever arm = $0.9 \times d$, balance depth is given by Eq.

$$M = 0.45 b_f \sigma_{cbc} D_f \left[\frac{2k_c d - D}{k_c} \right] \text{ where } b_f = \text{flange width of isolated T-Beam given b}$$

$$b_f = \frac{l_0}{\left(\frac{l_0}{b} \right) + 4} + b_w \quad \left(\text{where } l_0 = L = 4.38 \text{ m ; } b = \text{actual width} = 1.50 \text{ and } b_w = 0.20 \text{ m} \right)$$

$$b_f = \frac{4.38}{\left(\frac{4.38}{1.50} \right) + 4} + 0.20 = 0.833 \text{ m} = 833 \text{ mm}$$

$$\text{Hence, } = 0.45 \times 833 \times 7 \times 80 \left[\frac{2 \times 0.289 d - 80}{0.289} \right] = 26796875$$

$$\therefore 209916 \left[\frac{2 \times 0.289 d - 80}{0.289} \right] = 26796875$$

$$\therefore 2d - \frac{80}{0.289} = 127.66$$

$$d = \frac{127.66 + 277.2}{2} = 202.00 \text{ mm}$$

Also, $d = \frac{V_e}{b_w \cdot \tau_{c \max}}$ where $\tau_{c \max} = 1.8 \text{ N/mm}^2$ for m-20 concrete

$$V_e = V + 1.6 \frac{T}{b_w} \quad \text{Where } V = \frac{wL}{2} = \frac{14000 \times 4.38}{2} = 30625 \text{ N}$$

T = torsional moment, which will be induced due to live load acting only to one side of step.

$$T = \left[\frac{4000 \times 0.60}{2} \right] \times \frac{4.38}{2} \times 1000 = 1575000 \text{ N-mm}$$

$$\text{or } T = (4000 \times 0.60) \times \frac{4.4}{2} \times 1000 = 5250000 \text{ N-mm}$$

$\therefore T = 5250000$ which ever is more

$$V_e = 30625 + 1.6 \times \left(\frac{5250000}{200} \right) = 72625 \text{ N}$$

$$\text{Hence, } d = \frac{72625}{\frac{200}{1.6}} = 227 \text{ mm} \quad \text{However, keep total depth} = 230$$

using 20 mm main bars, 10 mm Φ ring and cover 25 mm

$$\text{Net available } d = 230 - 10 - 10 - 25 = 185 \text{ mm}$$

$$A_{st} = \frac{BM \times 1000}{\sigma_{st} \times j \times D} = \frac{26796875}{230 \times 0.904 \times 185} = 697 \text{ mm}^2$$

$$\text{using } \text{###} \text{ mm } \Phi \text{ bars } A = \frac{3.14 \times \text{dia}^2}{4} = \frac{3.14 \times 20 \times 20}{4} = 251$$

$$\text{spacing of Bars} = \frac{A_{st}}{A} = \frac{4 \times 100}{697 / 314} = 3 \text{ Nos. Say } 4$$

$$\text{Actual } A_{st} \text{ provided} = 4 \times 314 = 1256 \text{ mm}^2$$

Note:- the above reinforcement is for bending requirements only. there will be additional longitudinal reinforcement for torsion, as computed later.

Location of N.A. Assuming the N.A. falls within the flange, we have

$$= \frac{833 \times n^2}{2} = 13.33 \times 1256 \times (185 - n)$$

$$\therefore 833 \times n^2 = 6194718 - 33484.96 n$$

$$\therefore n^2 = 7437 - 40.20 n$$

$$\therefore n^2 + 40.2 n - 7437 = 0$$

$$\therefore n = \frac{40.2 \pm (1616 + 29747)^{0.5}}{2 \times 1}$$

$$\therefore n = 68.4 \text{ mm} \text{ Hence the resultant falls inside the flanges } \bar{y} = 22.82$$

$$\therefore \text{L.A. a} = d - \bar{y} = 185 - 22.82 = 162.2 \text{ mm}$$

$$\text{Stress in steel} = \frac{M}{A_{st} \cdot A} = \frac{26796875}{1256 \times 162} = 132 \text{ N/mm}^2 < 230 \text{ Hence}$$

Corresponding stress in concrete is given by

$$c = \frac{t \times n}{m d - n} = \frac{132}{13.33} \times \frac{68.4}{185 - 68.4} = 5.80 \text{ N/mm}^2 < 7 \text{ Hence}$$

5 Design for torsion :-

As computed earlier, $T = 5250000 \text{ N-mm}$

$$v = 30625 \text{ N and } v_e = 72625 \text{ N}$$

$$\therefore \tau_{ve} = \frac{V_e}{b_w \cdot d} = \frac{72625}{200 \times 185} = 1.96 \text{ N/mm}^2$$

$$\frac{100 \times A_{st}}{b \times d} = \frac{100 \times 1256}{200 \times 185} = 3.395 \% \text{ hence from table 3.2 } t_c = 0.51 \text{ N/mm}^2$$

Since $\tau_{ve} > t_c$ shear reinforcement required

(a) Longitudinal reinforcement:-

$$M_{e1} = M + M_T \text{ Where } M = 26796875$$

$$M_T = T \frac{(1 + D/bw)}{1.7} = 5250000 \frac{1 + 230 / 200}{1.7} = 6639705.9$$

$$\therefore M_{e1} = 26796875 + 6639706 = 33436581 \text{ N-mm}$$

$$A_{st} = \frac{M_{e1}}{\sigma_{st} \times j \times D} = \frac{33436581}{230 \times 162.18} = 896 \text{ mm}^2$$

Hence the provision of 4 bars of 20 mm ϕ , giving 1256 mm² is O.K.

Near the column D, take the bars straight up. Provide 2- 20 mm ϕ bars at the lower face under the land

(b) Transverse reinforcement:-

Transverse reinforcement will be provided in the form of vertical stirrups.

Let provide 25 mm clear cover all round

b_1 = center to center distance between corner bars in the direction of width

$$= 200 - 2 \times 25 - 10 = 140 \text{ mm}$$

d_1 = center to center distance between corner bars in the direction of depth

$$= 230 - 2 \times 25 - 10 = 170 \text{ mm}$$

$$\text{using } \text{mm } \Phi \text{ stirrups bars } A = 2 \times \frac{3.14 \times d^2}{4 \times 100} = 2 \times \frac{3.14 \times 10 \times 10}{4 \times 100} = 157$$

$$\text{now, } A_{sv} = \frac{T_{sv}}{b_1 d_1 \sigma_{sv}} + \frac{V_{sv}}{2.5 d_1 \sigma_{sv}} \text{ or}$$

$$157 = \left(\frac{5250000}{140 \times 170 \times 230} + \frac{30625}{2.5 \times 170 \times 230} \right) S_v$$

$$157 = (0.9591 + 0.313) S_v \text{ or } S_v = 157 / 1.272 \text{ or } S_v = 123$$

However, the spacing should not exceed the least of $x_1, (x_1 + y_1)/4$ and 300 mm

$$\text{where } x_1 = \text{short dimension of stirrups} = 140 + 20 + 10 = 170 \text{ mm}$$

$$y_1 = 170 + 20 + 10 = 200 \text{ mm}$$

$$(x_1 + y_1)/4 = (170 + 200)/4 = 92.5 \text{ mm}$$

Hence $S_v = 123.39 \text{ mm}$ is not permissible. Keep $S_v = 90 \text{ mm c/c}$. In case the spacing

200 mm c/c in the mid span where both transverse shear as well as torsional shear are minimum.
provide 2 - 10 mm f holding bars. Keep the same setion for other flight.

pk_nandwana@yahoo.co.in

mm

3.

mm

mm

mm²

mm

mm²

mm

N/m

N-mm

yEq.

m }
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mm

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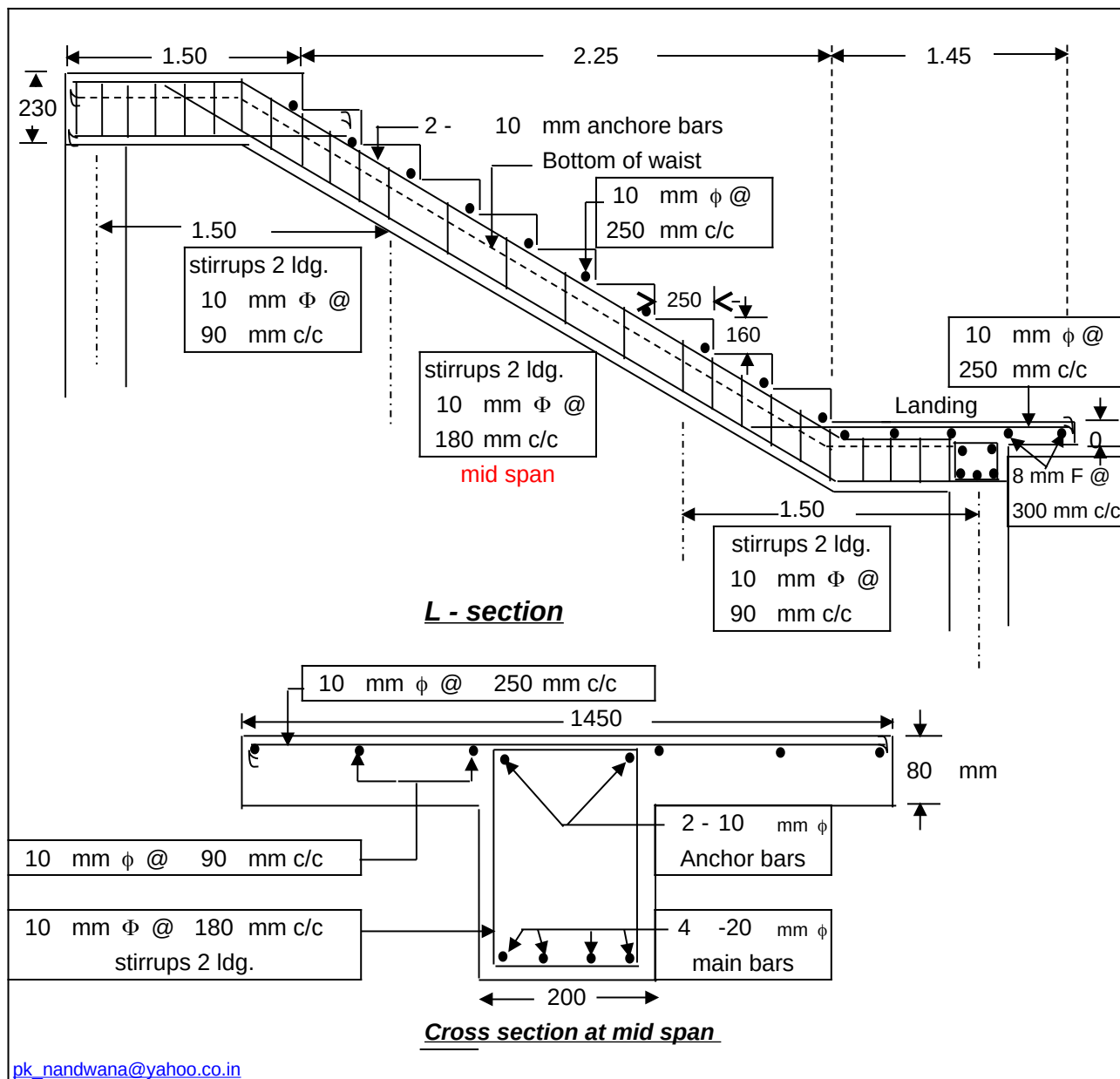
N-mm

ng.

mm²

mm

to



VALUES OF DESIGN CONSTANTS							
Grade of concrete	M-15	M-20	M-25	M-30	M-35	M-40	
Modular Ratio	18.67	13.33	10.98	9.33	8.11	7.18	
σ_{cbc} N/mm ²	5	7	8.5	10	11.5	13	
$m \sigma_{cbc}$	93.33	93.33	93.33	93.33	93.33	93.33	
(a) $\sigma_{st} = 140$ N/mm ² (Fe 250)	k_c	0.4	0.4	0.4	0.4	0.4	0.4
	j_c	0.867	0.867	0.867	0.867	0.867	0.867
	R_c	0.867	1.214	1.474	1.734	1.994	2.254
	P_c (%)	0.714	1	1.214	1.429	1.643	1.857
(b) $\sigma_{st} = 190$ N/mm ²	k_c	0.329	0.329	0.329	0.329	0.329	0.329
	j_c	0.89	0.89	0.89	0.89	0.89	0.89
	R_c	0.732	1.025	1.244	1.464	1.684	1.903
	P_c (%)	0.433	0.606	0.736	0.866	0.997	1.127
(c) $\sigma_{st} = 230$ N/mm ² (Fe 415)	k_c	0.289	0.289	0.289	0.289	0.289	0.289
	j_c	0.904	0.904	0.904	0.904	0.904	0.904
	R_c	0.653	0.914	1.11	1.306	1.502	1.698
	P_c (%)	0.314	0.44	0.534	0.628	0.722	0.816
(d) $\sigma_{st} = 275$ N/mm ² (Fe 500)	k_c	0.253	0.253	0.253	0.253	0.253	0.253
	j_c	0.916	0.916	0.916	0.914	0.916	0.916
	R_c	0.579	0.811	0.985	1.159	1.332	1.506
	P_c (%)	0.23	0.322	0.391	0.46	0.53	0.599

Grade of concrete
 τ_{bd} (N / mm²)

Grade of concrete	
M 15	
M 20	
M 25	
M 30	
M 35	
M 40	
M 45	
M 50	

Permissible shear stress Table τ_v in concrete (IS : 456-2000)						
$\frac{100A_s}{bd}$	Permissible shear stress in concrete τ_v N/mm ²					
	M-15	M-20	M-25	M-30	M-35	M-40
≤ 0.15	0.18	0.18	0.19	0.2	0.2	0.2
0.25	0.22	0.22	0.23	0.23	0.23	0.23
0.50	0.29	0.30	0.31	0.31	0.31	0.32
0.75	0.34	0.35	0.36	0.37	0.37	0.38
1.00	0.37	0.39	0.40	0.41	0.42	0.42
1.25	0.40	0.42	0.44	0.45	0.45	0.46
1.50	0.42	0.45	0.46	0.48	0.49	0.49
1.75	0.44	0.47	0.49	0.50	0.52	0.52
2.00	0.44	0.49	0.51	0.53	0.54	0.55
2.25	0.44	0.51	0.53	0.55	0.56	0.57
2.50	0.44	0.51	0.55	0.57	0.58	0.60
2.75	0.44	0.51	0.56	0.58	0.60	0.62
3.00 and above	0.44	0.51	0.57	0.6	0.62	0.63

Grade of concrete	
M 10	
M 15	
M 20	
M 25	
M 30	
M 35	
M 40	
M 45	
M 50	

Maximum shear stress $\tau_{c,max}$ in concrete (IS : 456-2000)						
Grade of concrete	M-15	M-20	M-25	M-30	M-35	M-40
$\tau_{c,max}$	1.6	1.8	1.9	2.2	2.3	2.5

table-3.2

Shear stress t_c		Reinforcement %	
$\frac{100A_s}{bd}$	<i>M-20</i>	<i>M-20</i>	$\frac{100A_s}{bd}$
0.15	0.18	0.18	0.15
0.16	0.18	0.19	0.18
0.17	0.18	0.2	0.21
0.18	0.19	0.21	0.24
0.19	0.19	0.22	0.27
0.2	0.19	0.23	0.3
0.21	0.2	0.24	0.32
0.22	0.2	0.25	0.35
0.23	0.2	0.26	0.38
0.24	0.21	0.27	0.41
0.25	0.21	0.28	0.44
0.26	0.21	0.29	0.47
0.27	0.22	0.30	0.5
0.28	0.22	0.31	0.55
0.29	0.22	0.32	0.6
0.3	0.23	0.33	0.65
0.31	0.23	0.34	0.7
0.32	0.24	0.35	0.75
0.33	0.24	0.36	0.82
0.34	0.24	0.37	0.88
0.35	0.25	0.38	0.94
0.36	0.25	0.39	1.00
0.37	0.25	0.4	1.08
0.38	0.26	0.41	1.16
0.39	0.26	0.42	1.25
0.4	0.26	0.43	1.33
0.41	0.27	0.44	1.41
0.42	0.27	0.45	1.50
0.43	0.27	0.46	1.63
0.44	0.28	0.46	1.64
0.45	0.28	0.47	1.75
0.46	0.28	0.48	1.88
0.47	0.29	0.49	2.00
0.48	0.29	0.50	2.13
0.49	0.29	0.51	2.25
0.5	0.30		
0.51	0.30		
0.52	0.30		
0.53	0.30		
0.54	0.30		
0.55	0.31		
0.56	0.31		
0.57	0.31		
0.58	0.31		
0.59	0.31		

0.6	0.32
0.61	0.32
0.62	0.32
0.63	0.32
0.64	0.32
0.65	0.33
0.66	0.33
0.67	0.33
0.68	0.33
0.69	0.33
0.7	0.34
0.71	0.34
0.72	0.34
0.73	0.34
0.74	0.34
0.75	0.35
0.76	0.35
0.77	0.35
0.78	0.35
0.79	0.35
0.8	0.35
0.81	0.35
0.82	0.36
0.83	0.36
0.84	0.36
0.85	0.36
0.86	0.36
0.87	0.36
0.88	0.37
0.89	0.37
0.9	0.37
0.91	0.37
0.92	0.37
0.93	0.37
0.94	0.38
0.95	0.38
0.96	0.38
0.97	0.38
0.98	0.38
0.99	0.38
1.00	0.39
1.01	0.39
1.02	0.39
1.03	0.39
1.04	0.39
1.05	0.39
1.06	0.39
1.07	0.39
1.08	0.4
1.09	0.4
1.10	0.4

1.11	0.4
1.12	0.4
1.13	0.4
1.14	0.4
1.15	0.4
1.16	0.41
1.17	0.41
1.18	0.41
1.19	0.41
1.20	0.41
1.21	0.41
1.22	0.41
1.23	0.41
1.24	0.41
1.25	0.42
1.26	0.42
1.27	0.42
1.28	0.42
1.29	0.42
1.30	0.42
1.31	0.42
1.32	0.42
1.33	0.43
1.34	0.43
1.35	0.43
1.36	0.43
1.37	0.43
1.38	0.43
1.39	0.43
1.40	0.43
1.41	0.44
1.42	0.44
1.43	0.44
1.44	0.44
1.45	0.44
1.46	0.44
1.47	0.44
1.48	0.44
1.49	0.44
1.50	0.45
1.51	0.45
1.52	0.45
1.53	0.45
1.54	0.45
1.55	0.45
1.56	0.45
1.57	0.45
1.58	0.45
1.59	0.45
1.60	0.45
1.61	0.45

1.62	0.45
1.63	0.46
1.64	0.46
1.65	0.46
1.66	0.46
1.67	0.46
1.68	0.46
1.69	0.46
1.70	0.46
1.71	0.46
1.72	0.46
1.73	0.46
1.74	0.46
1.75	0.47
1.76	0.47
1.77	0.47
1.78	0.47
1.79	0.47
1.80	0.47
1.81	0.47
1.82	0.47
1.83	0.47
1.84	0.47
1.85	0.47
1.86	0.47
1.87	0.47
1.88	0.48
1.89	0.48
1.90	0.48
1.91	0.48
1.92	0.48
1.93	0.48
1.94	0.48
1.95	0.48
1.96	0.48
1.97	0.48
1.98	0.48
1.99	0.48
2.00	0.49
2.01	0.49
2.02	0.49
2.03	0.49
2.04	0.49
2.05	0.49
2.06	0.49
2.07	0.49
2.08	0.49
2.09	0.49
2.10	0.49
2.11	0.49
2.12	0.49

2.13	0.50
2.14	0.50
2.15	0.50
2.16	0.50
2.17	0.50
2.18	0.50
2.19	0.50
2.20	0.50
2.21	0.50
2.22	0.50
2.23	0.50
2.24	0.50
2.25	0.51
2.26	0.51
2.27	0.51
2.28	0.51
2.29	0.51
2.30	0.51
2.31	0.51
2.32	0.51
2.33	0.51
2.34	0.51
2.35	0.51
2.36	0.51
2.37	0.51
2.38	0.51
2.39	0.51
2.40	0.51
2.41	0.51
2.42	0.51
2.43	0.51
2.44	0.51
2.45	0.51
2.46	0.51
2.47	0.51
2.48	0.51
2.49	0.51
2.50	0.51
2.51	0.51
2.52	0.51
2.53	0.51
2.54	0.51
2.55	0.51
2.56	0.51
2.57	0.51
2.58	0.51
2.59	0.51
2.60	0.51
2.61	0.51
2.62	0.51
2.63	0.51

2.64	0.51
2.65	0.51
2.66	0.51
2.67	0.51
2.68	0.51
2.69	0.51
2.70	0.51
2.71	0.51
2.72	0.51
2.73	0.51
2.74	0.51
2.75	0.51
2.76	0.51
2.77	0.51
2.78	0.51
2.79	0.51
2.80	0.51
2.81	0.51
2.82	0.51
2.83	0.51
2.84	0.51
2.85	0.51
2.86	0.51
2.87	0.51
2.88	0.51
2.89	0.51
2.90	0.51
2.91	0.51
2.92	0.51
2.93	0.51
2.94	0.51
2.95	0.51
2.96	0.51
2.97	0.51
2.98	0.51
2.99	0.51
3.00	0.51
3.01	0.51
3.02	0.51
3.03	0.51
3.04	0.51
3.05	0.51
3.06	0.51
3.07	0.51
3.08	0.51
3.09	0.51
3.10	0.51
3.11	0.51
3.12	0.51
3.13	0.51
3.14	0.51

3.15	0.51

Permissible Bond stress Table τ_{bd} in concrete (IS : 456-2000)								
M-10	M-15	M-20	M-25	M-30	M-35	M-40	M-45	M-50
--	0.6	0.8	0.9	1	1.1	1.2	1.3	1.4

Development Length in tension			
Plain M.S. Bars		H.Y.S.D. Bars	
τ_{bd} (N / mm ²)	$k_d = L_d \Phi$	τ_{bd} (N / mm ²)	$k_d = L_d \Phi$
0.6	58	0.96	60
0.8	44	1.28	45
0.9	39	1.44	40
1	35	1.6	36
1.1	32	1.76	33
1.2	29	1.92	30
1.3	27	2.08	28
1.4	25	2.24	26



Permissible stress in concrete (IS : 456-2000)					
Permission stress in compression (N/mm ²)				Permissible stress in bond (Average) for plain bars in tention (N/mm ²)	
Bending α_{cbc}		Direct (α_{cc})			
(N/mm ²)	Kg/m ²	(N/mm ²)	Kg/m ²	(N/mm ²)	in kg/m ²
3.0	300	2.5	250	--	--
5.0	500	4.0	400	0.6	60
7.0	700	5.0	500	0.8	80
8.5	850	6.0	600	0.9	90
10.0	1000	8.0	800	1.0	100
11.5	1150	9.0	900	1.1	110
13.0	1300	10.0	1000	1.2	120
14.5	1450	11.0	1100	1.3	130
16.0	1600	12.0	1200	1.4	140

0.40.81.21.62

Percentage of tension reinforcement

2

2.4

2.8