UNIT 12 DESIGN OF SIMPLE RECTANGULAR SLABS

Structure

12.1 INTRODUCTION

A slab is a two dimensional structural element carrying loads. Floor and roof slab can be of any shape- rectangular, triangular, trapezoidal, circular etc.. They may be simply supported fixed, hinged or continuous over the supports. Deciding degree of fixity and ideal support conditions for most of the slabs is a bit difficult task in practice. However, for a slab supported on sides, may be taken either simply supported, discontinuous or continuous on a supported edge for design by Working Stress Method. Partial fixity is taken care of by providing appropriate amount of negative reinforcement near a discontinuous edge. In this Unit, simple rectangular slabs with uniformly distributed loads only shall be dealt with.

Objectives

After going through this Unit a student will be able to design the following types of slabs with uniformly distributed loads :

(a) One way slabs

(b) Two way slabs, and

(c) Cantilever slabs.

12.2 DESIGN OF ONE WAY SLABS

A slab is designed as one way slab under the following conditions :

(i) When it is supported on two opposite sides and free on the other two (Figure 12.1)

In this case the bending of the slab due to design load takes place only along supported span. Each strip of the slab parallel to supports and spanning perpendicular to the supported span, remain straight (i.e. it does not share the design load) after load application.

(ii) When it is suppoted on all the edges but $l_x \leq l_y/2$ (Figure 12.2). where l_x and l_y are shorter and longer spans respectively.

In this case, though small portions of the slab bend in longitudinal direction near the shorter edge; for design load carrying purposes, it is assumed to bend *only* along the shorter span (Figure 12.2)

For design purposes, a strip (beam) of *unit width* along the supported edge in (i) or along the shorter span in (ii) is considered for analysis of bending moment and shear forces. The safety and serviceability criteria are the same as those for beams - except as mentioned below :

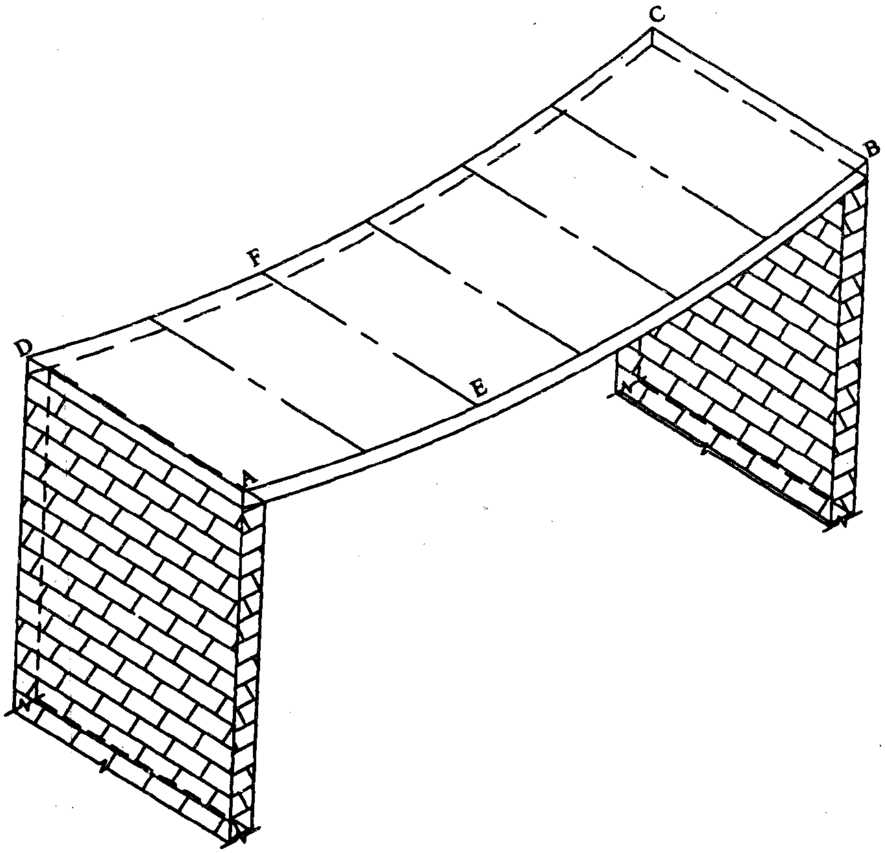


Figure 12.1 : Bending of One Way Slab Supported on Two Opposite Edges

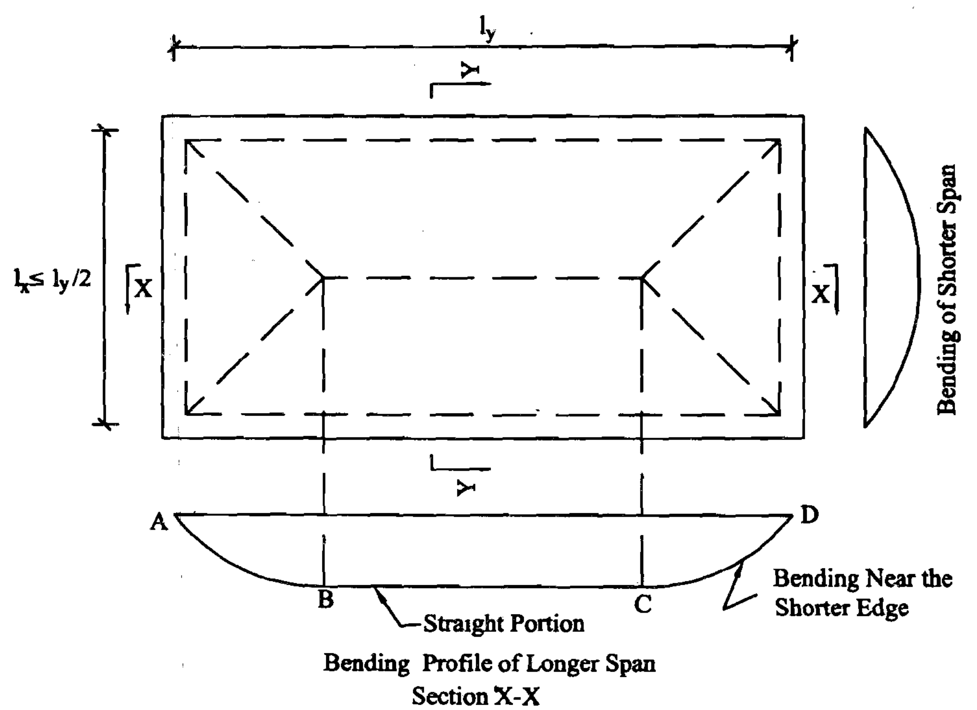


Figure 12.2 : Bending Profile of an One Way Slab Supported on all Edges

(a) For solid slabs the permissible *shear stress* in concrete shall be $k\tau_c$ where k has the value given in Table 12.1.

Table 12.1 : Modification Factor (*k*) for τ_c (vide Table 8.3)

Depth of Slab (mm)	300 or more	275	250	225	200	175	150 or less
**k*	1.00	1.05	1.10	1.15	1.20	1.25	1.30

(b) The reinforcement in either direction in slab shall not be less than 0.15 percent of the total corss sectional area. However, this value can be reduced to 0.12 percent when high strength deformed bars are used.

(c) The diameter of reinforceing bars shall not exceed one eighth of the total thickness of the slab.

(d) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of a solid slab or 450 mm whichever is smaller.

(e) The horizontal distance between parallel reinforcing bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 whichever is smaller.

(f) A clear concrete cover of at least 15 or equal to dia. of bar shall be provided to tensile, compressive, shear or other reinforcement of a slab.

Example 12.1

Design a R.C.C. simply supported roof slab for a room having inside dimension 3.5m x 7.6m and L.L. of 1.5 kN/m. The roof covering is 150 mm thick lime concrete and the slab has a ceiling plaster of 10 mm. The slab is supported on 300 mm thickwall on all its four sides. Use M 15 concrete and Fe 415 steel.

Solution

Since the effective lengths are not known at the outset it may be taken c/c distance of supports in each direction. i.e.

$l_x \approx 3.5\text{m} + 0.3\text{m} = 3.8\text{m}$ and

$l_y \approx 7.6\text{m} + 0.3\text{m} = 7.9\text{m}$

$$\therefore \frac{l_y}{l_x} = \frac{7.9}{3.8} = 2.08 > 2$$

Hence the slab will be designed as one way simply supported slab. Taking effective span,

$l_{ef} \approx 3.8\text{m}$

Depth (*D*)

(i) *Thumb Rule*

$$D \approx \frac{l_{ef}}{10} \text{ to } \frac{l_{ef}}{20}$$

$$\text{Taking } D = \frac{l_{ef}}{20} = \frac{3.8}{20} = 0.19\text{m} = 190$$

* This does not apply to flat slabs.

(ii) *From Deflection Control Criteria*

$$\frac{l_{ef}}{d} < K_B K_1 K_2 K_3$$

where $K_B = 20$

$K_1 = 1.4$ for $A_{stB} = 0.317\%$ (for $\sigma_{cbc} = 5$ MPa & $\sigma_{st} = 230$ MPa)

$K_2 = K_3 = 1$

Substituting these value

$$\frac{l_{ef}}{d} < 20 \times 1.4 \times 1 \times 1$$

or $$\frac{l_{ef}}{d} < 28$$

or $$d > \frac{l_{ef}}{28}\left(= \frac{3.8 \times 10^3}{28} = 125.7\right)$$

$\therefore D > 135.7 + 15 + 5 = 155.7$

Assuming preliminarily $D = 190$

Loads

Self	$= 1 \times 1 \times 0.19 \times 25$	=	4.750 kN/m
LC	$= 1 \times 1 \times 0.15 \times 18.8$	=	2.800 kN/m
Ceiling plaster	$= 1 \times 1 \times 0.01 \times 20.4$	=	0.204 kN/m
Total *DL*		=	7.774 kN/m
LL		=	1.500 kN/m
Total Load, *w*		=	9.274 kN/m

Maximum, B.M., $M = \frac{wl^2_{ef}}{8} = \frac{9.274 \times 3.8^2}{8}$ = 16.740 kN/m

$$\therefore d = \sqrt{\frac{M}{R_B b}} = \sqrt{\frac{16.74 \times 10^6}{0.658 \times 1000}} = 159.5$$

Hence, $D = 159.5 + 15 + 5 = 179.5$

Hence provided $D = 190$

$\therefore d = 190 - 15 - 5 = 170$

(i) Effective span l_{ef} is greater of c/c distance of supports $= 3.5 + 0.3 = 3.8$m

(ii) clear span $+ d = 3.5 + 0.17 = 3.67$m

Hence, $l_{ef} = 3.67$m

A_{st}

Maximum B.M., $M = \frac{wl^2_{ef}}{8} = \frac{9.274 \times 3.67^2}{8} = 15.61$ kNm

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{15.61 \times 10^6}{230 \times 0.902 \times 170} = 442.61 \text{ mm}^2$$

$$A_{st,min} = \frac{0.12}{100} \times 1000 \times 190 = 216 \text{ mm}^2 < 442.61 \text{ mm}^2$$

Using #10 bars (A_s= 78.5 mm^2)

$$\text{spacing} = \frac{1000 \times 78.5}{442.61} = 177.36$$

Hence provided #10 @175 (A_{st} = 448.6 mm^2/m)

Maximum spacing

(i) $3d = 3 \times 170$ = 510 c/c

(ii) = 450 c/c

Check for shear

$$\text{Maximum S.F., } V_{max} = \frac{wl_{ef}}{2} = \frac{9.274 \times 3.67}{2} = 17\text{kN}$$

S.F. at critical section at face of a support,

$$V = \frac{17}{\frac{l_{ef}}{2}} \times \left(\frac{l_{ef}}{2} - 0.5d\right) = \frac{17}{1.835} \times (1.835 - 0.085) = 16.2\text{kN}$$

$$\tau_v = \frac{V}{bd} = \frac{16.2 \times 10^3}{1000 \times 170} = 0.095 \text{ MPa} < \tau_{c,\,max}(1.6 \text{ MPa})$$

$$\frac{100A_s}{bd} = \frac{100 \times 1000 \times 78.5}{1000 \times 170 \times 175} = 0.26$$

$$\therefore \tau_c = k\left[0.22 + \frac{(0.29 - 0.22)}{(0.50 - 0.25)} \times (0.26 - 0.25)\right] = 0.223k \text{ MPa}$$

$$\text{where } k = 1.2 + \frac{(1.25 - 1.2)}{(200 - 175)} \times (200 - 190) = 1.22$$

Hence $\tau_c = 1.22 \times 0.223 = 0.272$ MPa > 0.095 MPa Hence O.K.

Development Length

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{10 \times 230}{4 \times 0.84} = 684.5$$

The positive main reinforcement shall extend into the support

a distance of 230 $\left(> \frac{L_d}{3}\right)$

$$L_d < \frac{1.3M_1}{V} + L_0$$

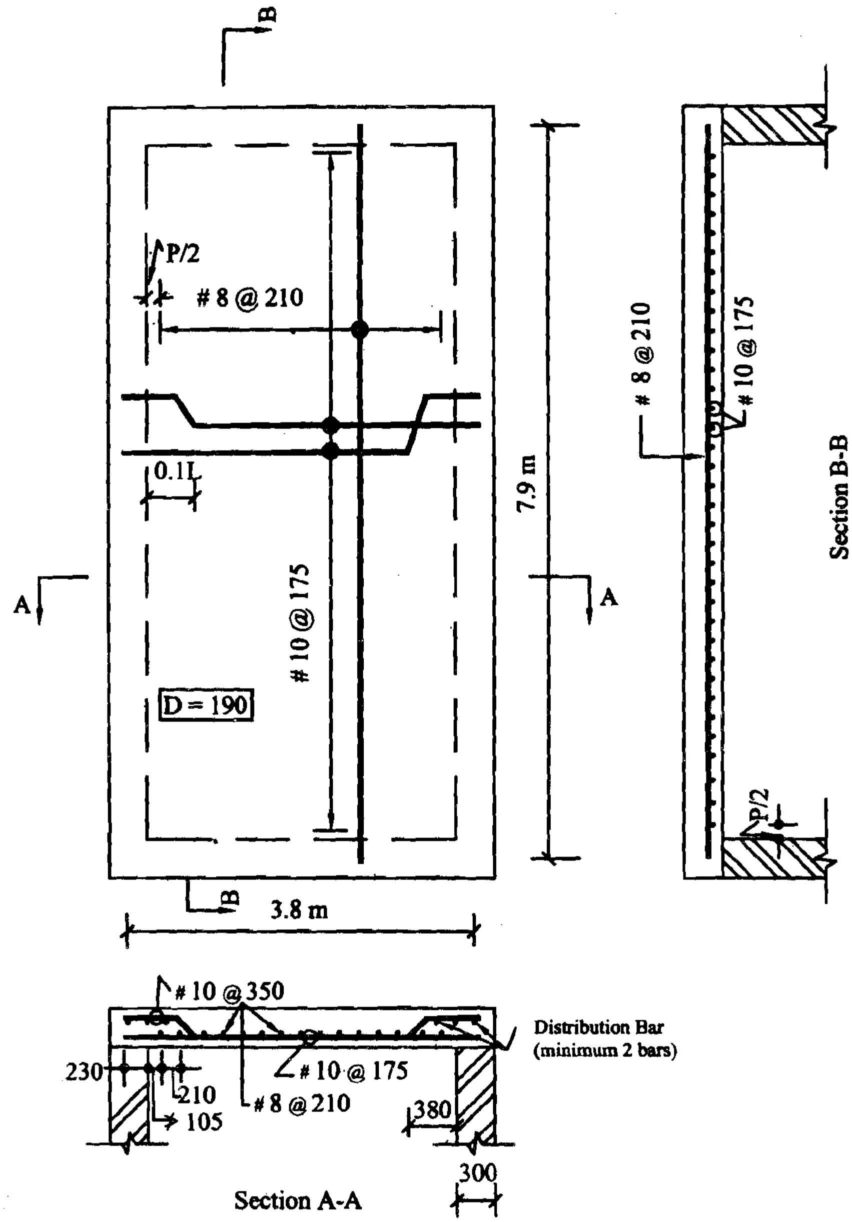


Figure 12.3 : Reinforcement Detailing of the Designed Slab

where $M_1 = f_d A'_{st} j_B d = 230 \times \frac{448.6}{2} \times 0.90 \times 170 \times 10^{-6} = 7.91$ kNm

where A'_{st} = Area of positive tensile reinforcement extending into the support. i.e. 50% of the maximum positive reinforcement at mid span

V = 17 kN (i.e. shear force at simple support)

L_0 = Length of positive tensile reinforcement into the support $- \frac{\text{support width}}{2}$

$$= 230 - \frac{230}{2} = 80$$

Substituting these values in the above formula for L_d

$$L_d < \frac{1.3 \times 7.91 \times 10^3}{17} + 80 = 684.88 \text{ O.K. as } L_d = 684.5$$

Dsitribution Steel

$$A_{std} = \frac{0.12}{100} bd = \frac{0.12}{100} \times 1000 \times 190 = 228 \text{ mm}^2$$

Using #8 bars (A_s = 50 mm²)

$$\text{spacing} = \frac{1000 \times 50}{228} = 219.3$$

Hence provided #8 @210

The reinforcement detailing has been shown in Figure 12.3.

Example 12.2

Design a continuous one way slab for the roof system shown in the Figure 12.4. The ceiling plaster provided is 10 mm thick and the roof covering provided of 150 mm thick lime concrete. The live load on the roof will be taken as 0.75 kN/m². Use M 15 concrete and Fe 415 steel.

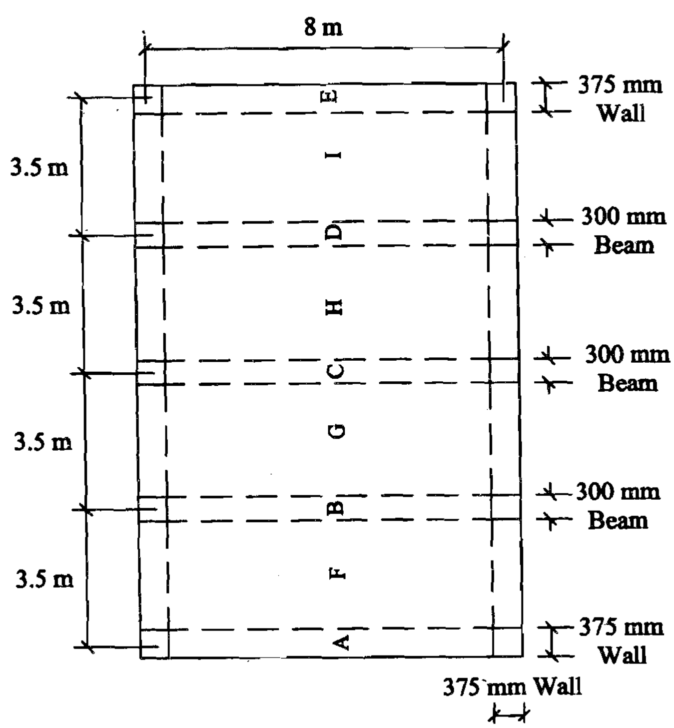


Figure 12.4 : A Continuous One Way Slab

Solution

Depth (*D*)

(i) *Control of Deflection*

$$\frac{l_{ef}}{d} < K_B K_1 K_2 K_3$$

where $K_B = \frac{20+26}{2} = 23$ (since the maxiumum depth will be for end span for which one end is discontinuous and the other continuous)

$K_1 = 1.4$

$K_2 = K_3 = 1$

Substituting these values

$$\frac{l_{ef}}{d} < 23 \times 1.4 \times 1 \times 1$$

or $d > \frac{l_{ef}}{32.2}\left(= \frac{3.5 \times 10^3}{32.2} = 108.7 \right)$ (where l_{ef} taken c/c distance to start with)

$\therefore D > 108.7 + 15 + 5 = 128.7$

Assuming preliminarily $D = 225$

$\therefore d = 225 - 15 - 5 = 205$

Loads

Self	$= 1 \times 1 \times 0.225 \times 25$	= 5.625 kN/m
LC	$= 1 \times 1 \times 0.15 \times 18.8$	= 2.820 kN/m
Ceiling plaster	$= 1 \times 1 \times 0.01 \times 20.4$	= 0.204 kN/m
	Total *DL*	= 8.650 kN/m
LL		= 0.750 kN/m
	Total Load *w*	= 9.400 kN/m

l_{ef} **for End Span** (vide cl. 21.2 b of the Code)

$\because$ Clear Span $= 3.5 - \frac{0.375}{2} - \frac{0.3}{2} = 3.163$

$\frac{1}{12}$ of clear span = 0.264 m

and the width of support, 0.375 m > 0.264 m

l_{ef} of *end span shall be least of*

Clear span $+ \frac{d}{2} = 3.163 + 0.103 = 3.266$ m

or

Clear span + $\frac{\text{width of support}}{2} = 3.163 + 0.1875 = 3.351$ m

Hence, l_{ef} = 3.266 m

l_{ef} of *intermediate span* (vide cl. 21.2 b of the Code)

l_{ef} = clear span between the support = 3.5 − 0.3 = 3.2 m

Since the roof system has more than 3 spans which donot differ by more than 15%

$$\left(\frac{3.266}{3.2} = 1.023\right),$$

design bending moments and shears shall be as per cl. 21.5.1 of the Code

BENDING MOMENT CALCULATION

TYPE OF LOAD	SPAN MOMENTS		SUPPORT MOMENTS	
	Near Middle of End Span (F)	At Middle of Interior Span (G)	At support Next to the end support (B)	At other Interior supports (C)
DL	$\frac{1}{12}\times 9.4\times 3.266^2$ = 8.36 kNm	$\frac{1}{24}\times 9.4\times 3.2^2$ = 4.01 kNm	$-\frac{1}{10}\times 9.4\times 3.266^2$ =–10.03 kNm	$-\frac{1}{12}\times 9.4\times 3.2^2$ =–8.02 kNm
LL	$\frac{1}{10}\times 9.4\times 3.266^2$ = 10.03 kNm	$\frac{1}{12}\times 9.4\times 3.2^2$ = 8.02 kNm	$\frac{1}{-9}\times 9.4\times 3.266^2$ =–11.14 kNm	$\frac{1}{-9}\times 9.4\times 3.2^2$ =–10.7 kNm
(DL+LL)	**18.39 kNm**	**12.03 kNm**	**–21.17 kNm**	**–18.72 kNm**

SHEAR FORCE CALCULATION

TYPE OF LOAD	AT END SUPPORT (A)	AT SUPPORT NEXT TO THE END SUPPORT (B)		AT ALL OTHER INTERIOR SUPPORT (C)
		Outer Side	Inner Side	
DL	0.45×9.4×3.266 = 13.8 kNm	0.6×9.4×3.266 = 18.42 kNm	0.55×9.4 ×3.2 =16.54 kNm	0.5×9.4×3.2 = 15.04 kNm
LL	0.5×9.4×3.266 = 15.35 kNm	0.6×9.4×3.266 = 18.42 kNm	0.6×9.4×3.2 = 18.05 kNm	0.6×9.4×3.2 = 18.05 kNm
(DL+LL)	**29.15 kNm**	**36.84 kNm**	**34.59 kNm**	**33.09 kNm**

Maxmimum B.M. , M = 21.17 kNm

$$\therefore d = \sqrt{\frac{21.17\times 10^6}{0.658\times 1000}} = 179.34$$

$$D = 179.34 + 15 + 5 = 199.4$$

Hence **Adopted D = 225**

and **effective depth, d = 225– 15– 5 = 205**

At B,

$$A_{st} = \frac{M_B}{\sigma_{st} j_B d} = \frac{21.17\times 10^6}{230\times 0.904\times 205} = 496.7\ \text{mm}^2$$

$A_{st,min} = 0.12 \times bd = 0.12 \times 1000 \times 225 = 270\ \text{mm}^2 < 496.7\ \text{mm}^2$

Using # 10 bars ($A_{st} = 78.5\ \text{mm}^2$)

$$\text{Spacing} = \frac{1000\times 78.5}{496.7} = 158.05$$

Maximum spacing (i) $3d = 3 \times 205 = 615$

(ii) 450

Hence Maximum Spacing = 450 > 155

Provided # 10 @ 155

At F (Near middle of end span),

$$A_{st} = \frac{M_F}{\sigma_{st} j_B d} = \frac{18.39 \times 10^6}{230 \times 0.904 \times 205} = 431.45 \text{ mm}^2$$

Using # 10 bars ($A_s = 78.5$ mm²)

$$\text{Spacing} = \frac{1000 \times 78.5}{431.45} = 182$$

Provided #10 @180 (A_{st} = 436.11 mm²)

At G (Middle of interior support)

$$A_{stG} = \frac{M_G}{\sigma_{st} j_B d} = \frac{12.03 \times 10^6}{230 \times 0.904 \times 205} = 282.24 \text{ mm}^2$$

Using # 10 bars ($A_s = 78.5$ mm²)

$$\text{Spacing} = \frac{1000 \times 78.5}{282.24} = 278.13$$

Provided #10 @275

At C (other interior support)

$$A_{st} = \frac{M_C}{\sigma_{st} j_B d} = \frac{18.72 \times 10^6}{230 \times 0.904 \times 205} = 439.19 \text{ mm}^2$$

Using # 10 bars ($A_s = 78.5$ mm²)

$$\text{Spacing} = \frac{1000 \times 78.5}{439.19} = 178.74$$

Provided #10 @175

Temperature reinforcement equal to 0.12% of the gross concrete area will be provided in the longitudinal direction.

$$= \frac{0.12}{100} \times 1000 \times 225 = 270 \text{ mm}^2/\text{m}$$

Using 8 mm ϕ (A_{st}= 50 mm²)

$$\text{Spacing} = \frac{1000 \times 50}{270} = 185.2$$

Maximum spacing for temp. reinforcement, the least of
(i) $5\,d = 5 \times 205 = 1025$, and

(ii) 450

Provided #8 @185

Check for shear

The maximum shear force occurs at the outer side of the support next to end support (B)

Shear Force, S = 36.84 kN

Shear Force at distance d from face of support = 36.84− 9.4 x 0.205= 34.91 kN

$$\text{Shear stress } \tau_v = \frac{S}{bd} = \frac{34.91 \times 10^3}{1000 \times 205} = 0.17 \text{ MPa}$$

$$\text{At B, \% of tension steel } = \frac{100 A_s}{bd} = \frac{100 \times \left(\frac{1000 \times 78.5}{155}\right)}{1000 \times 205} = 0.25$$

$\therefore \tau_c = 1.15 \times 0.22 = 0.253$ MPa > 0.17 MPa Hence O.K.

Check for Development Length

Lets take the face of end support A

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} = \frac{10 \times 230}{4 \times 0.6 \times 1.4} = 684.52$$

$$M_1 = \frac{A_{st(atF)}}{2} \times \sigma_{st} j_b d$$

$$= \frac{436.11}{2} \times 230 \times 0.904 \times 205 = 9.294 \text{ kNm}$$

V = 29.15 kN (at end support)

$$L_0 = \frac{L_s}{2} x' + (8\phi - 5\phi)$$

$$= \frac{375}{2} - 25 + 3 \times 10 = 192.5$$

$$\text{Hence, } \frac{1.3 M_1}{V} + L_0 = \frac{1.3 \times 9.294 \times 10^6}{29.15 \times 10^3} + 192.5 = 607$$

Hence **extra length above the standard 90^0 bend, to be provided = 685− 607 = 78**

$\therefore L_0 = 192.5 + 78 = 270.5 \approx 275$

$$\frac{L_d}{3} = \frac{684.52}{3} = 228.17 \approx 230$$

The reinforcement detailing has been shown in Figure 12.5

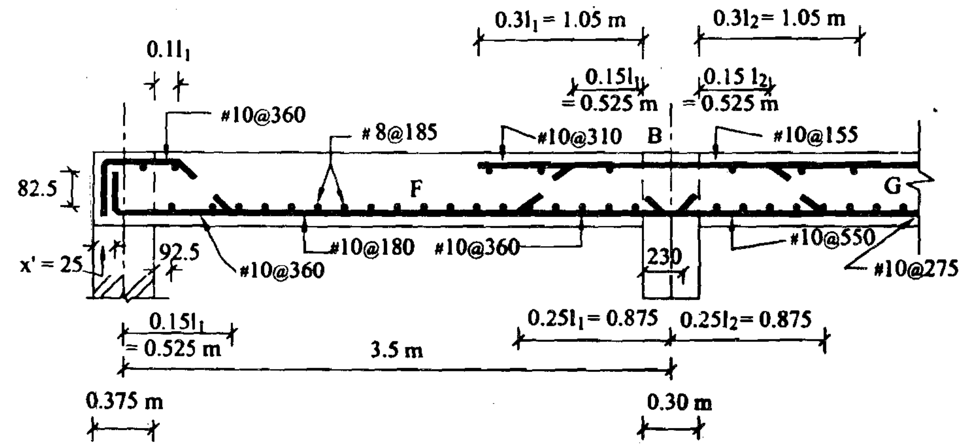


Figure 12.5 : Reinforcement Details of One Way Continuous Slab

12.3 DESIGN OF TWO WAY SLABS

When a slab is supported on all the sides and the ratio of longer to shorter span is less than 2 (i.e. $\frac{l_y}{l_x} < 2$) the slab is designed as two way slab. To be more clear, if $\frac{l_y}{l_x} < 2$, due to bending of slab along both spans, the design load on any point (i.e. on any common unit area crossed by two mutually perpendicular strips of unit width) is shared by both the spans (Figure 12.6). A two way slab depending on the support conditions, may be either of the following types :

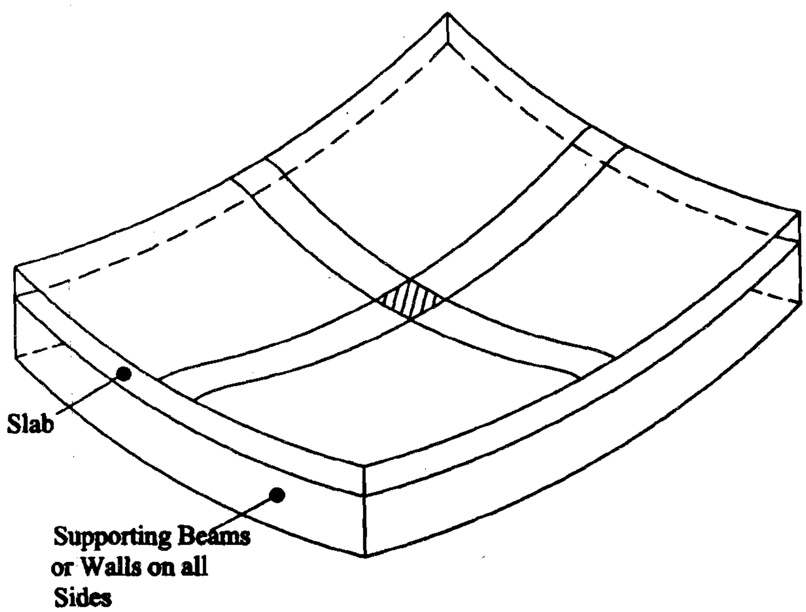


Figure 12.6 : Bending of Two Way Slab

(a) *simply supported slab having no adequate provision to resist torsion at corners and to prevent corners from lifting.* In this case, the maximum bending moments per unit width are given by the following equations :

$$M_x = \alpha_x w l_x^2 \qquad \ldots (12.1)$$

$$M_y = \alpha_y w l_x^2 \qquad \ldots (12.2)$$

where α_x and α_y are the coefficients given in Table 12.2

w = design load per unit area

M_x, M_y = bending moments on strips of unit width spaning l_x and l_y respectively, and

l_x and l_y = lengths of the shorter span and longer span respectively.

Table 12.2 : Bending Moment Coefficients for Slabs Spanning in Two Directions at Right Angles, Simply Supported on Four Sides

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

At least 50% of the tension reinforcement provided at mid span should extend to the supports. The remaining 50% should extend to within $0.1l_x$ and $0.1l_y$ of the support, as appropriate.

(b) *When the corners of a slab are prevented from lifting, the slab may be designed as explained below :*

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

$$M_x = \alpha_x w l_x^2 \qquad \ldots (12.3)$$

$$M_y = \alpha_y w l_x^2 \qquad \ldots (12.4)$$

where α_x and α_y are as given in Table 12.3

Table 12.3 : Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

Case No.	Type of Panel and Moments Considered	Short Span Coefficients (Values of l_y/l_x) 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long Span Coefficients For All Values of l_y/l_x
1.	*Interior Panels:*									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.24
2.	*One Shot Edge Discontinuous:*									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3.	*One Long Edge Discontinuous :*									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4.	*Two Adjacent Edges Discontinuous*									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive mement at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5.	*Two Short Edges Discontinuous*									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	---
	Positive mement at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6.	*Two Long Edges Discontinuous*									
	Negative moment at continuous edge	---	---	---	---	---	---	---	---	0.045
	Positive mement at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7.	*Three Edges Discontinuous (One Long Edge Continuous):*									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	---
	Positive mement at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8.	*Three Edges Discontinuous (One Short Edge Continuous):*									
	Negative moment at continuous edge	---	---	---	---	---	---	---	---	0.057
	Positive mement at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9.	*Four Edges Discontinuous*									
	Positive mement at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

The rules for detailing of reinforcements in this type of slab are :

A slab is divided in each direction into middle strip and edge strip as shown in Figure 12.7

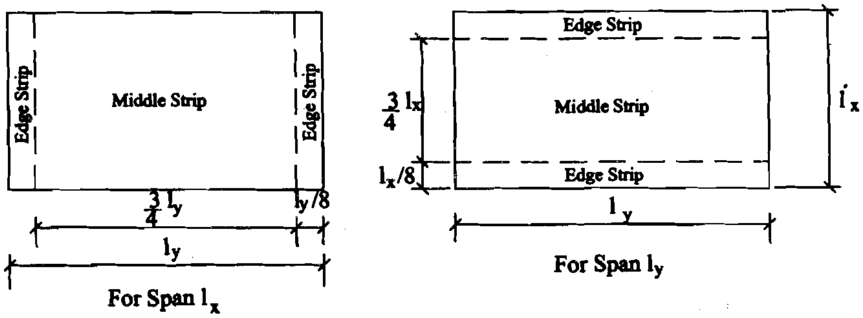


Figure 12.7 : Division of Slab into Middle and Edge Strips

The reinforcement calculated from Eqs. 12.3 & 12.4 shall be provided *only* in middle strips.

The horizontal distance between parallel *main* reinforcing bars shall not be more than three times the effective depth of a slab or 450 whichever in smaller.

In the edge strips, reinforcement shall be provided against temperature, shrinkage, and the spacing of these bars shall not be more than 5 times the effective depth of a solid slab or 450 whichever is smaller.

The detailing of reinforcement have been shown in Figure 12.8 below.

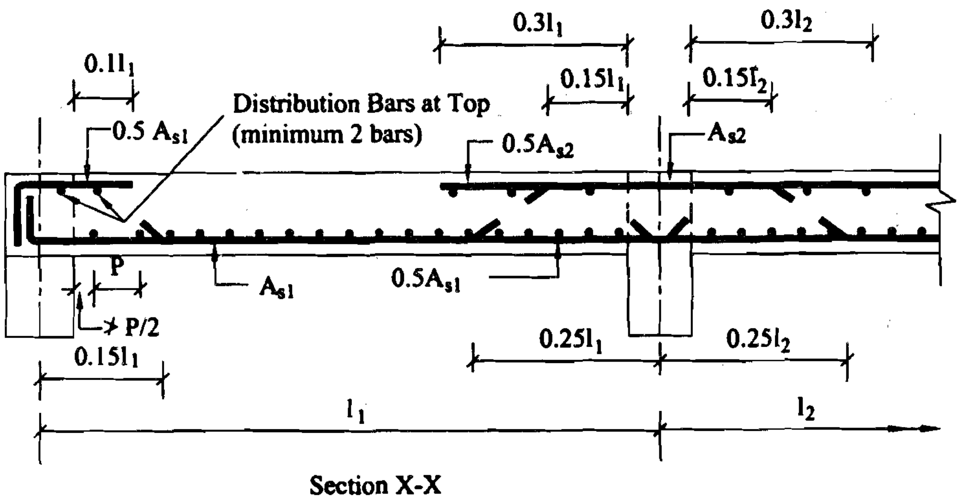


Figure 12.8 : Curtailment Rules for Two Way Slab

In addition to above, torsion reinforcement at corners are provided to prevent cracking due to torsion. If a corner is bounded by two adjacent simple supports, top and bottom reinforcements, *each* with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of $\frac{1}{5}$th of the shorter span shall be provided. 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 Figure 12.12(i).

Where a corner is bounded by one continous and other by discontinuous edge the torsion reinforcement equal to half of that decribed above shall be provided Figure 12.12(ii).

No torsion reinforcement need to be provided at corners bounded by continuous edges only.

Example 12.3

Figure 12.9 shows the structural plan of an office floor of a multistoreyed building of the following specifications :

(i) Live load on floor = 3 kN/m^2

(ii) Floor finish & ceiling plaster = 1.08 kN/m^2

(iii) Provision for removable light partition wall = 1 kN/m^2, and

(iv) Grade of concrete and reinforcement used are M 15 and Fe 415 respectively.

Design and draw the slabs S1 & S2.

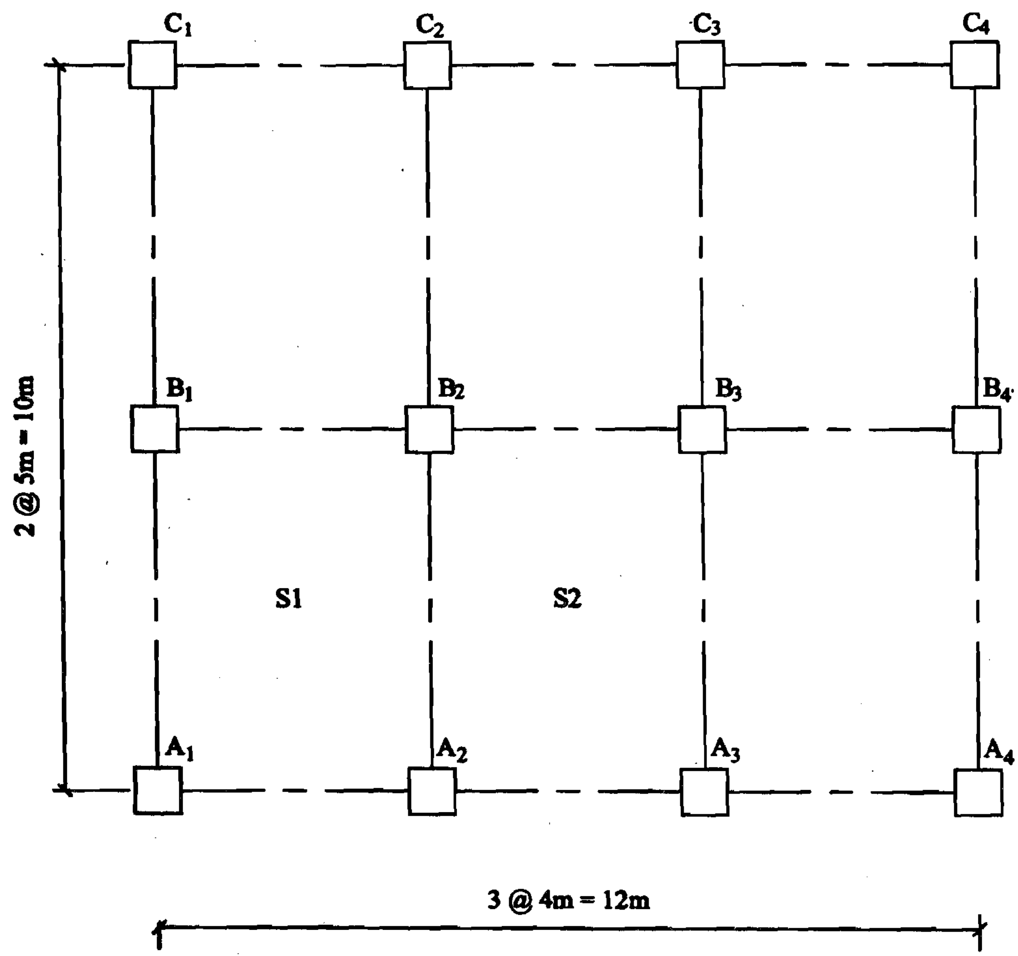


Figure 12.9 : Showing Structural Plan

Solution

Design Constants

$$k_B = \frac{93.33}{93.33 + \sigma_{st}} = \frac{93.33}{93.33 + 230} = 0.289$$

$$j_B = \left(1 - \frac{j_B}{3}\right) = 0.9$$

$$R_B = \frac{1}{2} k_B j_B \sigma_{cbc} = \frac{1}{2} \times 0.289 \times 0.9 \times 5 = 0.65$$

Depth (D)

(i) *From Deflection Control*

$$\frac{l_{ef}}{d} < K_B K_1 K_1 K_3$$

where $K_B = (20 + 26)/2 = 23$ (for one edge continuous and other discontinuous)

For M 15 concrete and Fe 415 steel,

$p_B\% = 0.314\%$

$\therefore K_1 = 1.4$

$K_2 = K_3 = 1$

Substituting these values in the above equation

$$\frac{l_{ef}}{d} < 23 \times 1.4 \times 1 \times 1 (= 32.2)$$

or $\quad \dfrac{4 \times 1000}{d} < 32.2$ or $d = 124.22$

or $\quad D = 124.22 + 15 + 8/2 = 143.22$ (assuming bar dia = 8 mm)

Taking $D = 150$

$d = 150 - 15 - 8/2 = 131$

(ii) *From Moment of Resistance Consideration*

Loads

Self = 0.15 × 1 × 1 × 25	= 3.75 kN/m²
Floor finish & ceiling plaster	= 1.08 kN/m²
Removable Partition	= 1.00 kN/m²
Total *DL*	= 5.83 kN/m²
LL	= 3.00 kN/m²
Total *DL* + *LL*	= 8.83 kN/m²

Values of * Coefficients α_x & α_y and Bending Moments M_x & M_y are given in Figure 12.10

Check for D

$$d = \sqrt{\frac{M_{max}}{R_B b}} = \sqrt{\frac{7.91 \times 10^6}{0.65 \times 1000}} = 110.31 < 131 \text{ Hence O.K.}$$

A_{st}

$$\text{At support, } A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{79.1 \times 10^6}{230 \times 0.9 \times 131} = 291 \text{ mm}^2$$

* 1. Values of α_x are written without perenthesis
2. Values of M_s are written in parenthesis (kNm/m)
3. Values of Average Bending Moments are written in rectangular boxes (kNm.m)

Spacing for # 8 bars, $s = \dfrac{1000 \times 50}{291} = 171.3$

$A_{st,min} = 0.12\ bd\% = \dfrac{0.12 \times 1000 \times 131}{100} = 157\ mm^2$

$\therefore s_{max} = \dfrac{1000 \times 50}{157} = 318.47$

spacing shall *also* not exceed the followings :

(i) $3d = 3 \times 131 = 393$, and

(ii) 450

Hence provided #8@170

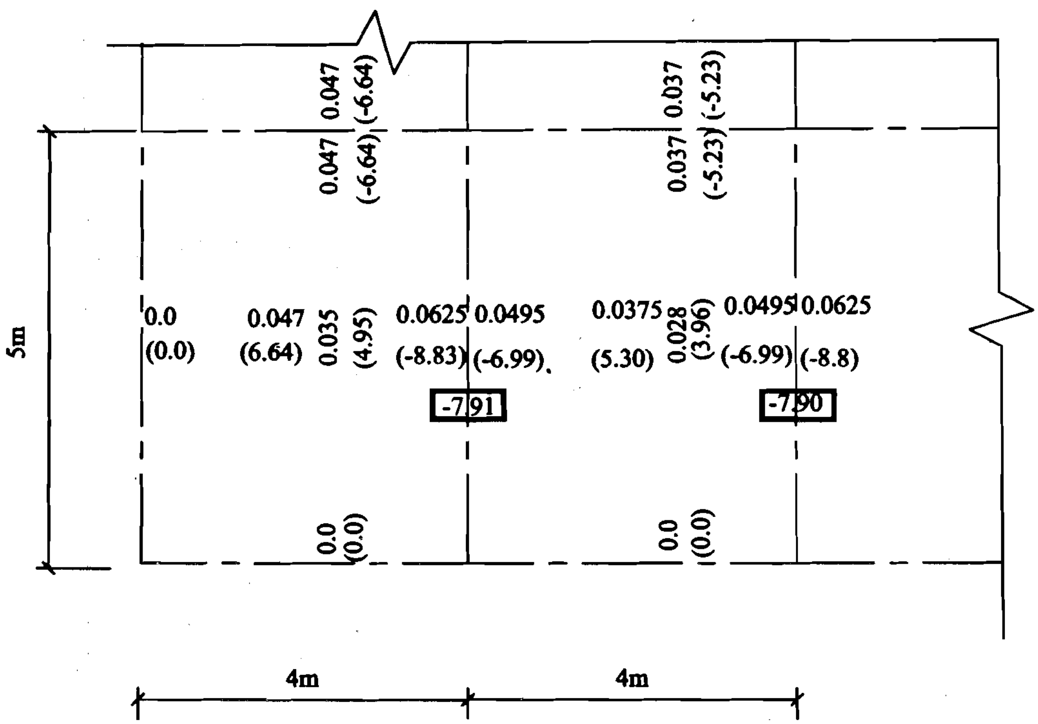


Figure 12.10 : Showing Values of α_x and M_s

Similarly the spacing of reinforcing bars at supports and mid spans in both directions have been evaluated and shown in Figure 12.11. It may be noted that $d = 131 - 8 = 123$ in the upper layer of reinforcement

Torsional Reinforcement at Corners

Maximum area of steel at mid span, $A_{st} = \dfrac{M}{\sigma_{st} j_B d} = \dfrac{6.64 \times 10^6}{230 \times 0.9 \times 131} = 244.86\ mm^2$

At corner A_1, the slab is simply supported on both edges, the area of reinforcement in each of the four layers shall be 3/4 th of the area required for the maximum mid span moment in the slab.

$\therefore$ Reinforced area required, $A_s = 244.8 \times \dfrac{3}{4} = 183.6\ mm^2$, and

spacing of #8 bars in each layer at corner $A_1 = \dfrac{1000 \times 50}{183.6} = 272.33$

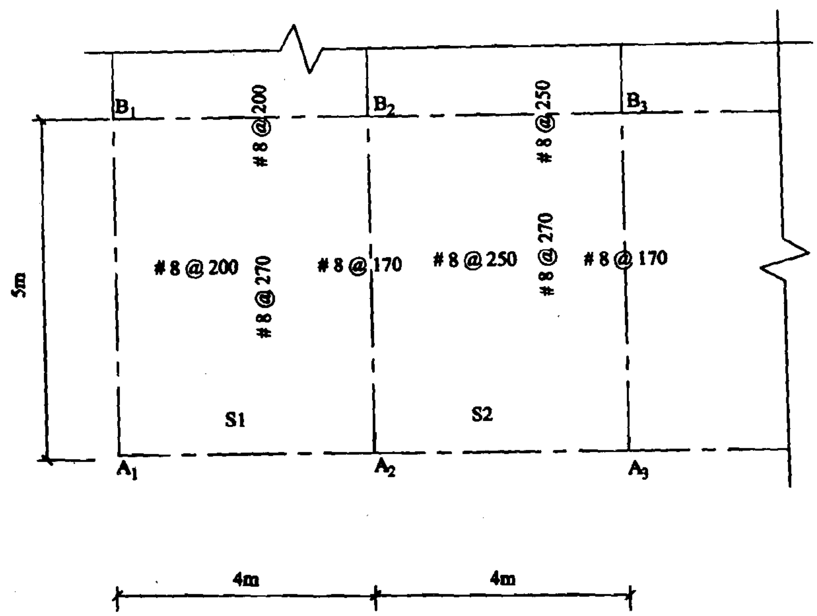
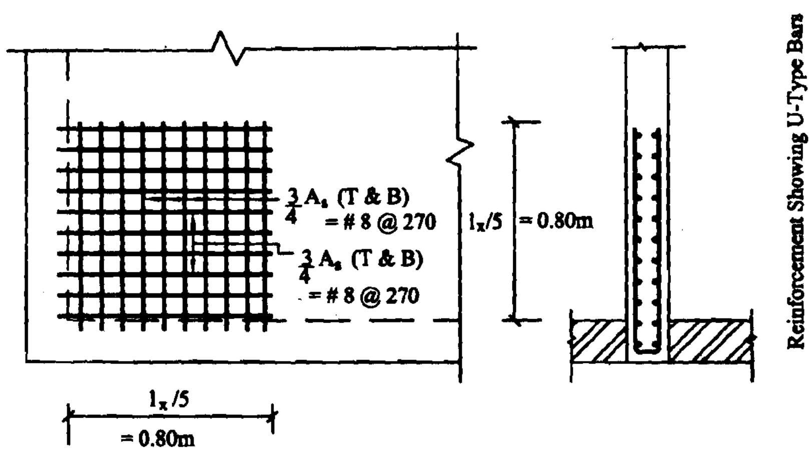
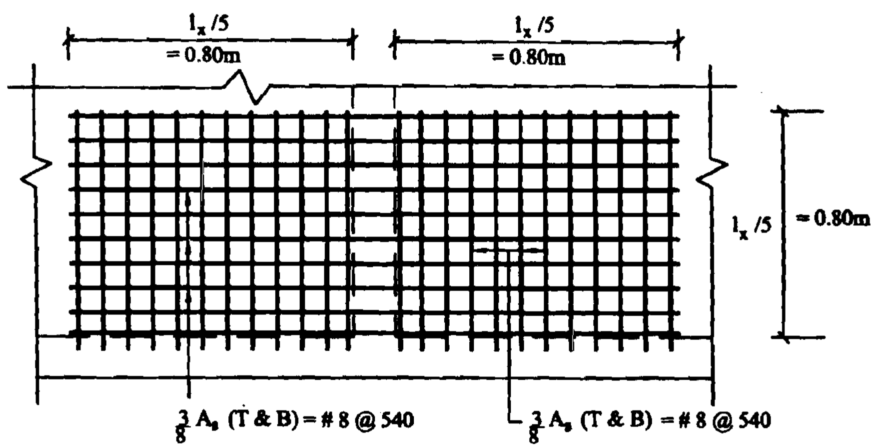


Figure 12.11 : Showing Reinforcement of the Designed Slab



(i) Corner with Two Discontinuous Ends



(ii) Corner with One Discontinuous End

Figure 12.12 : Torsional Reinforcement in Slab

Spacing of #8 bars in each layer at both sides of corner A_2, A_3 and B_1 = twice of that for A_1 (since area of reinforcement required is half) = 272.33 x 2 = 544.66 Detailing of reinforcements are shown in Figure 12.12

12.4 DESIGN OF CANTILEVER SLABS

A Cantilever slab is an one way slab, bending due to hogging bending moment necessiating main reinforcement to be placed near the top face. Its effective span is measured from face of its support to free edge. Figure 12.13 illustrates its configuration.

Example 12.4

Design a R.C.C. cantilever slab for a bus stop shade of 2.5m span as shwon in Figure 12.13 for a live load of 0.75 kN/m^2 and having a roof covering of bituminous felt for water proofing (Unit weight 0.744 kN/m^2) and ceiling plaster 10mm thick. Use M 15 concrete and Fe 415 steel.

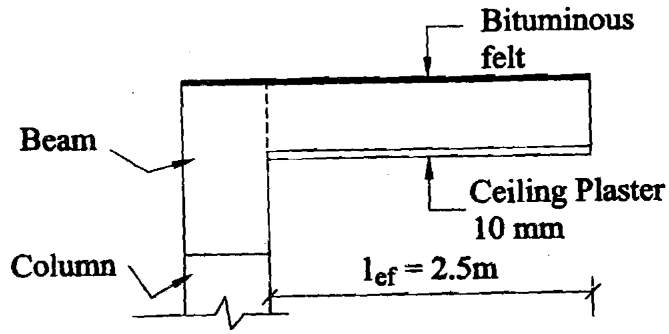


Figure 12.13 : A Cantilever Slab

Solution

Effective span

l_{ef} = 2.5m

Estimation of the Total Depth

(i) *From Deflection control*

$$\frac{l_{ef}}{d} < K_B K_1 K_2 K_3$$

where $K_B = 7$

$K_1 = 1.4$ for $A_{stB} = 0.317\%$

(for $\sigma_{cbc} = 5$MPa & $\sigma_{st} = 230$ MPa)

$K_3 = K_4 = 1$

Substituting these values

$$\frac{l_{ef}}{d} < 7 \times 1.4 \times 1 \times 1$$

or $$\frac{l_{ef}}{d} < 9.8$$

or $$d > \frac{l_{ef}}{9.80}\left(= \frac{2.5 \times 10^3}{9.8} = 255.1\right)$$

$\therefore$ $D = 255.1 + 15 + 5 = 275.1$

Assuming preliminarily $D = 280$

(ii) *From Moment of Resistance Criteria*

Loads

Self = $1 \times 1 \times 0.28 \times 25$	= 7.000 kN/m²
Bituminous felt = $1 \times 1 \times 0.714$	= 0.714 kN/m²
Ceiling plaster = $1 \times 1 \times 0.01 \times 20.4$	= 0.204 kN/m²
Total *DL*	= 7.275 kN/m²
LL	= 0.750 kN/m²
Total Load *w*	= 8.025 kN/m²

$$\text{Maximum BM, } M = \frac{wl_{ef}^2}{2} = \frac{8.025 \times (2.5)^2}{2} = 25.08 \text{ kNm/m}$$

$$\therefore d = \sqrt{\frac{M}{R_B b}} = \sqrt{\frac{25.08 \times 10^6}{0.658 \times 1000}} = 195.22$$

Hence $D' = 195.22 + 15 + 5 = 215.22$

Hence, **provided $D = 280$**

$\therefore$ $d = 280 - 15 - 5 = 260$

A_{st}

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{25.08 \times 10^6}{230 \times 260 \times 0.902} = 464.96 \text{ mm}^2$$

$$A_{st,min} = \frac{0.12}{100} \times 1000 \times 280 = 336 \text{ mm}^2 < 464.96 \text{ mm}^2$$

Using #10 bars, ($A_s = 78.5$ mm²)

$$\text{Spacing} = \frac{1000 \times 78.5}{464.96} = 168.83 \text{ c/c}$$

Maximum spacing

(i) $3d = 3 \times 260 = 780$ c/c

(ii) 450 c/c

Hence **provided #10 @165** ($A_{st} = \frac{1000 \times 78.5}{165} \times 1 = 475.75$ mm²/m)

Check for Shear

S.F. at critical section, $V = 8.025 \times (2.5 - 0.26) = 17.976$ kN

$$\tau_v = \frac{V}{bd} = \frac{17.976 \times 10^3}{1000 \times 260} = 0.069 \text{ MPa} < \tau_{c,\max} = 1.6$$

$$\frac{100A_s}{bd} = \frac{1000 \times 78.5}{165 \times 260 \times 1000} \times 100 = 0.183$$

$$\therefore \tau_c \approx 0.22\text{MPa} >> 0.069\text{MPa} \quad \text{Hence O.K.}$$

Curtailment of steel

As A_{stmin} (336 mm²) is more than 50% of A_{st} (464.96) provided. Hence all the bars shall be extended upto free end.

Development Length

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{10 \times 230}{4 \times 0.6 \times 1.4} = 484.52$$

Distribution Reinforcement

$$A_{sd} = \frac{0.12}{100} \times 1000 \times 280 = 336 \text{ mm}^2$$

Using #8 bars, (A_s= 50 mm²)

$$\text{Spacing} = \frac{1000 \times 50}{336} = 148.81$$

Hence provided #8@145

The detailing of reinforcement has been shown in Figure 12.14.

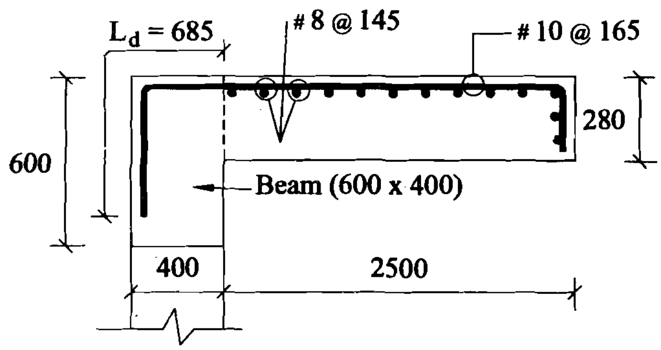


Figure 12.14 : Details of Reinforcement of Cantilever Slab Designed

SAQ 1

(i) Design and detail the slab of a room of size 3m x 6.5m supported on all sides onwalls of 230 mm thickness for a live load of 3 kN/m². The slab has a ceiling plaster and patent store flooring of 10 mm and 30 mm respectively. Use M 15 concrete and Fe 415 steel.

(ii) Design and detail the slab of the specification of Example (i) above excepth that size of room is 4 m x 5.5 m.

12.5 SUMMARY

A plate transferring transverse load of floor/roof to its supports is called a slab. A simple floor or roof slab supported on one side (a cantilever slab), or two opposite sides (a simply supported one way slab) and on all the four sides (an one way simply supported slab, a continuous one way slab and two way slab) have been designed and detailed in this Unit.

12.6 ANSWERS TO SAQs

SAQ 1

(i) Refer Example 12.1

(ii) Refer Section 12.3 and Table 12.2 for α_x