CHAPTER 5

TWO WAY EDGE SUPPORTED SLABS

5.1 Introduction

One-way slab (slab supported on two opposite edges only) has only one plane of bending, and the load is transferred to those two supports. But if a slab is supported on all the four edges, the load is transferred on the four supports and bending takes place along both spans. Also, the bending moments and deflection are considerably reduced as compared to one-way slab. Thus, a thinner slab can carry the same load when supported on all the four edges than one-way slab. When two-way slab is loaded, the corners get lifted-up. If the corners are held down (which is the usual case) by fixity at the wall support, bending moment and deflection are further reduced; but special torsion reinforcement at the corners has to be provided to check the cracking of corners of slab.

In two-way square slab, the two-way action is equal in each direction. In long narrow slabs, where the ratio of long to short span is greater than two, the two-way action effectively reduced to one-way action in the direction of short span although the end beams do carry some load.

The exact analysis of stresses in two-way slab is quite complex and is based on elastic theory; and it requires solution of higher order differential equation. It is usual to neglect poisson's ratio in such calculations. For analysis of homogeneous isotropic plate the basic differential equation used to determine internal forces is given as,

$$\frac{\partial^2 M_x}{\partial x^2} + \frac{\partial^2 M_y}{\partial y^2} - \frac{2\partial^2 M_{xy}}{\partial x \,.\, \partial y} = -q$$

where M_x & M_y are bending moment developed in slab in x- and y-direction

M_{xy} is twisting moment developed in slab

q --intensity of applied load on the plate

For practical design problem, codes provide tables of coefficients for moments and shear-forces obtained from elastic analysis of individual rectangular slab-panel corrected for redistribution of moments. The coefficients in the tables are given depending on aspect ratios, l_y/l_x and support conditions of slab panel. These tables may be used for analysis of any two-way slab system made of a numbers of rectangular slab panels.

Slabs, however, can be analyzed using approximate theories which have proved to be quite satisfactory for some cases of two-way slab. Rankine-Grashoff's method is the most commonly used theory. This method is suitable for analysis of simply supported two-way slabs if corners are not held down. This method neglects torsion at corners.

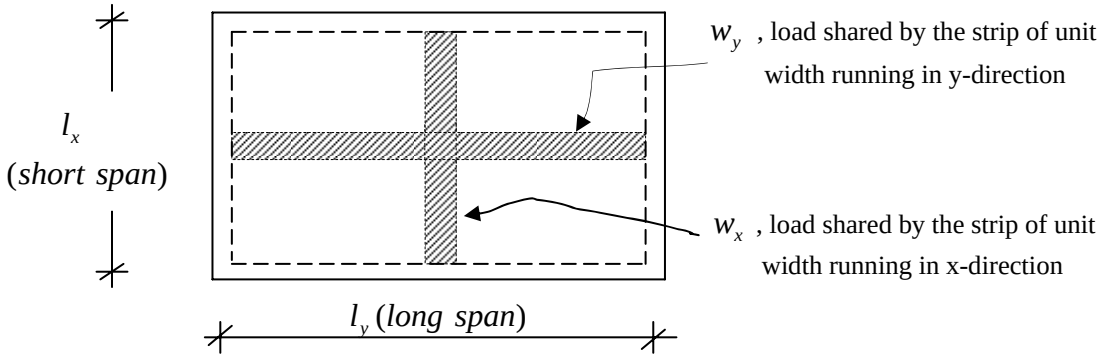
The finite element method (FEM) can be used to analyze slabs of any shape, boundary condition and subjected to any loading. This method can also account for stiffness of the

supporting beams. This method is extremely useful for slabs with openings and those subjected to concentrated load.

Two-way slabs can also be analyzed using the ultimate load theory. Johansen's yield line theory is the most popular. In this theory, the strength of the slab is assumed to be governed by flexure alone. The effects of shear and deflection are to be considered separately. It is assumed that a collapse-mechanism is formed in the slab at failure. The reinforcing steel is assumed to have fully yielded along the yield lines or cracks at failure. Then, analysis of slab is made using either *equilibrium* or *virtual-work* method on assumed yield lines of slab.

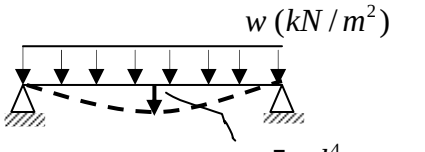
5.2. Rankine-Grashoff's Approximate Method

This method is suitable for analysis of two-way simply supported slab if corners are not held down (truly simply supported slab). The method neglect torsion at the corner of slab. The Rankine-Grashoff's method assume that load on the slab is shared between strips of unit width running in the two directions parallel to the side of the slab as shown below.



These slab strips are not independent in action. At their common intersection point, their deflections should be equal. Considering these slab strips as beams, the values of the share of load in both directions, w_x and w_y are obtained from compatibility of equal deflections of the strips at the center of the slab.

Consider a simply supported two-way slab loaded by uniformly distributed load, $w_d\,(kN/m^2)$. Let w_x and w_y share of w_d in x- and y-directions, respectively. Assuming slab strips as simply supported beam subjected to uniform load, the maximum deflection and maximum bending moment of slab strips are obtained as shown below,



$$\Delta_{\max} = \frac{5w.l^4}{384E.I}$$



$$M_{\max} = \frac{w.l^2}{8}$$

Deflections at the intersection of the two strips are equal, $\Rightarrow \quad \Delta_x = \Delta_y$

$$\frac{5w_x . l_x^4}{384E.I} = \frac{5w_y . l_y^2}{384E.I}$$

$$\Leftrightarrow \quad \frac{w_x}{w_y} = \left(\frac{l_y}{l_x}\right)^4 \qquad (1)$$

and, $\quad w_x + w_y = w_d \qquad (2)$

Let $\alpha = \frac{l_y}{l_x}$, equating Eq.(1) and Eq.(2)

$$w_x = \frac{\alpha^4}{1+\alpha^4} . w_d \qquad \text{and} \qquad w_y = \frac{1}{1+\alpha^4} . w_d$$

Therefore, the bending moment per unit width in both directions are given by substituting w_x and w_y into equations of maximum bending moment of slab strip as,

$$M_x = \frac{w_x . l_x^2}{8} = \frac{\alpha^4}{1+\alpha^4} . \frac{w_d . l_x^2}{8} = \beta_x . w_d . l_x^2$$

and $\quad M_y = \frac{w_y . l_y^2}{8} = \frac{\alpha^2}{1+\alpha^4} . \frac{w_d . l_x^2}{8} = \beta_y . w_d . l_x^2$

where $\quad w_d$ --*service* or *factored* uniform design load depending on design method

$$\beta_x = \frac{\alpha^4}{8\left(1+\alpha^4\right)}$$

and $\quad \beta_y = \frac{\alpha^2}{8\left(1+\alpha^4\right)}$

From these two equations of moment, it can be seen that a larger share of moment goes along the shorter span. Note that bending moment using Rankine-Grashoff's does not consider the effect of torsion at the corners. Values of bending moment coefficients, β_x & β_y are given table below.

Table: Bending moment coefficients for simply supported two-way rectangular slabs

$\alpha = l_y / l_x$	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
β_x	0.0625	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
β_y	0.625	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

5.3. Analysis of Two-way Rectangular Slab using Code's Coefficients (EBCS-2/95)

EBCS-2/95 Code provide moment coefficient table for analysis of rectangular slab panels subjected to uniformly distributed load with provision for torsion at the corners depending on aspect ratios, l_y/l_x and support conditions of slab. Provision for torsion prevents the lifting of the corners of the slab. Code's method can also be used for analysis of slab subjected to concentrated load in addition to a uniform load by treating concentrated load as equivalent-uniform load provided that the sum of the non-uniform load on a panel does not exceed 20% of the total load. Code's method assumes unyielding supports of slab. Unyielding supports of slab may be ensured by proportioning supports of slab with depth larger than or equal to 2.5 times thickness of slab.

Maximum moments for individual slab panels with edges either simply supported (discontinuous) or fully fixed (continuous) are given by,

$$M_i = \alpha_i \,.\, w_d \,.\, l_x^{\,2}$$

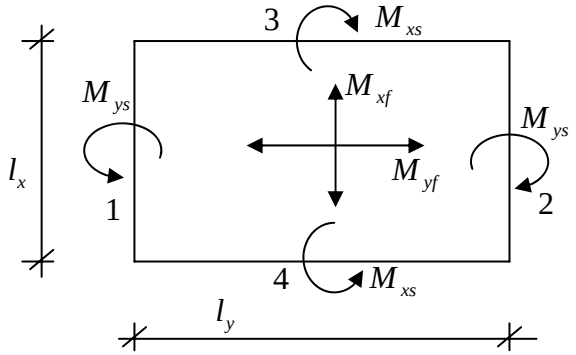
where M_i --is the design moment per unit width at the point of reference

α_i --is moment coefficient at the point of reference given by code as function of aspect ratio, l_y/l_x and support condition (refer table 1)

l_x & l_y are the shorter and longer spans of slab panel, in the respective direction

w_d --*service* or *factored* uniform design load depending on design method

Notation used for different critical moments and edge numbers are as shown below.



Subscripts used for moments and moment coefficient have the following meaning.

s--support [-ve moment]
f--field or span [+ve moment]
x--direction of shorter span
y--direction of longer span

Therefore, the maximum support and span moments per unit width develop at particular critical points of slab panel of two-way system are given by the following equations:

$$M_{xs} = \alpha_{xs} \,.\, w_d \,.\, l_x^{\,2}$$

$$M_{ys} = \alpha_{ys} \,.\, w_d \,.\, l_x^{\,2}$$

$$M_{xf} = \alpha_{xf} \,.\, w_d \,.\, l_x^{\,2}$$

and $$M_{yf} = \alpha_{yf} \,.\, w_d \,.\, l_x^{\,2}$$

Moment coefficient table given by EBCS-2/95 provide moment coefficients for nine separate slab panels with different possible support conditions as shown below.

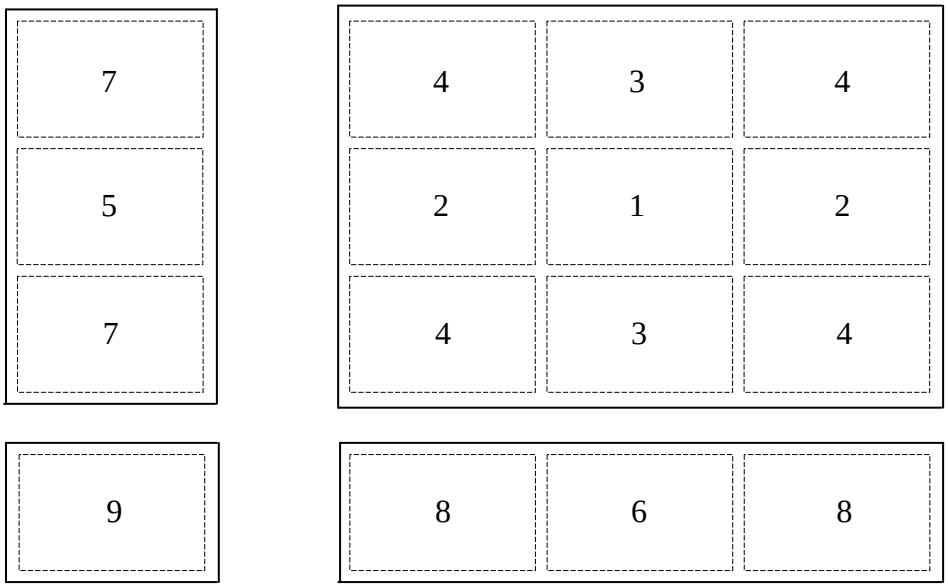
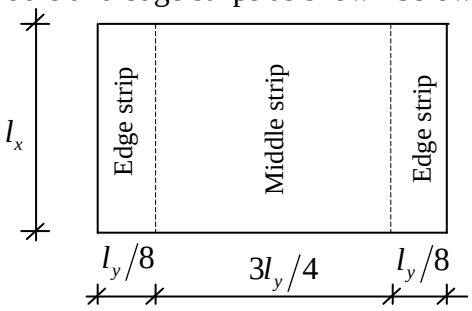


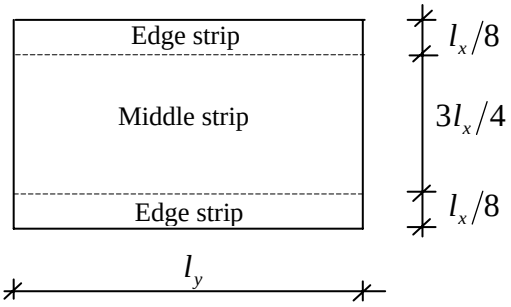
Fig: Possible two-way rectangular slab panel with different support

For slab panel with support condition different from those given above, interpolate linearly between the neighboring supports condition of slab panels. Analysis two-way slab system consist of a number of rectangular slab panels are made based on analysis of individual slab panels simultaneously loaded by the maximum design load of slab, $w_d = DL + LL$ in working stress method or $w_d = 1.3DL + 1.6LL$ in strength limit state method by treating edges of slab panels as either simply supported or fully fixed. External edges are generally considered simply supported, and continuous edges are considered fully fixed.

For purpose of design of slab and provision of reinforcement, the slab panel is divided into middle and edge strips as shown below.



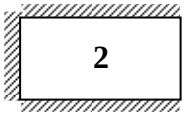
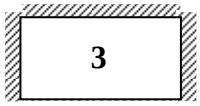
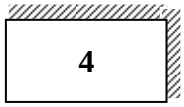
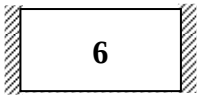
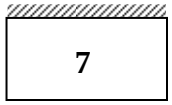
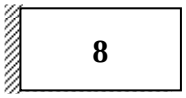
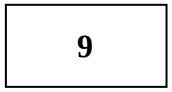
For reinforcement in short span



For reinforcement in long span

The maximum design span moments calculated as above apply to the middle strips and no redistribution shall be made. For intermediate support in continuous slab, there will thus be two different support moments. The difference may be distributed between the slab panels on either side of the support to equalize their moments, as in the moment distribution method for frames.

Table 1: Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corner

Support Condition	Coeff.	Values of l_y/l_x								Long span coefficients, α_{ys} & α_{yf}, for all values of l_y/l_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
 **1**	α_{xs}	0.032	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
	α_{xf}	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
 **2**	α_{xs}	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.039
	α_{xf}	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.029
 **3**	α_{xs}	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.039
	α_{xf}	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.030
 **4**	α_{xs}	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.047
	α_{xf}	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.036
 **5**	α_{xs}	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	
	α_{xf}	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
 **6**	α_{xs}									0.045
	α_{xf}	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
 **7**	α_{xs}	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	
	α_{xf}	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0.044
 **8**	α_{xs}									0.058
	α_{xf}	0.044	0.054	0.063	0.071	0.078	0.084	0.096	0.105	0.044
 **9**	α_{xf}	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

Two methods of differing accuracy are specified by EBCS-2 to distribute the intermediate support moments: *method I* and *method II*.

Method I: - Dimensioning in this method is carried out either for:

a) initial moment directly, or
b) average of initial moments at the support

This method may be used:

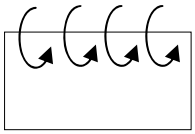
-When the difference between initial support moments are less than 20% of the larger moment, and

-For internal structures where live load does not exceed 2.5 times the dead load or for external structures 0.8 times dead load

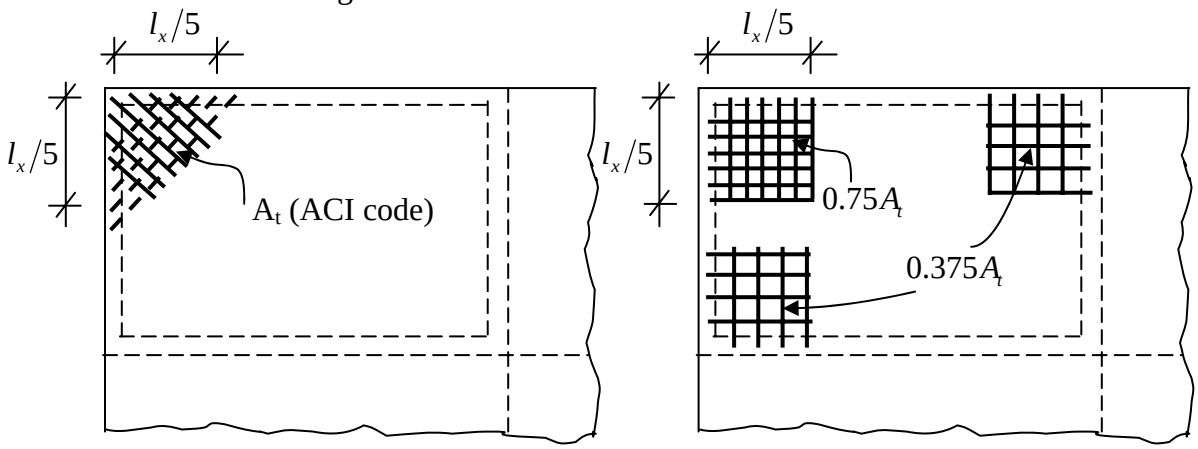
Method II:- The conditions given in method I are not met, method II or other more accurate method shall be used to distribute unbalanced support moment. In method II consideration of change of support moments is limited to the adjacent span. Therefore, the unbalanced support moment is locally distributed at each edge without iteration using the moment distribution method depending on the relative stiffness of the adjacent slab panels. The relative stiffness of each slab panels shall be taken proportional to its gross moment of inertia divided by the smaller span.

If the support moment is decreased while carrying out moment distribution of unbalanced support moment, the span moments M_{xf} & M_{yf} are then increased to allow for the change of support moments. This increase is calculated as being equal to the change of the support moment multiplied by the factor given in table 2. If a support moment is increased, no adjustment shall be made to the span moments.

Table 2: Factors for adjusting span moments, M_{xf} & M_{yf}

l_y/l_x	 l_y		 l_y	
	c_x	c_y	c_x	c_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481	0.251
2.0	0.238	0.055	0.484	0.248

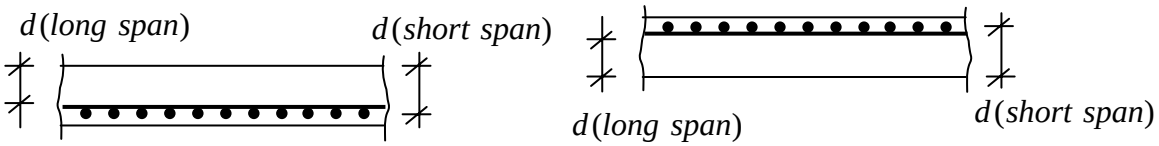
At corners of discontinuous corners of two-way slab, special torsion reinforcement is required at top along diagonal and at bottom perpendicular to the diagonal of rectangle extends for the length about $l_x/5$ from corner as shown below. Alternatively, mesh reinforcement may be provided at top and bottom of the corner of the rectangle. This mesh reinforcement according to BS:8110 is as shown below.



Corner reinforcement both along diagonal & perpendicular to diagonal for torsion

Alternate corner reinforcement at top & bottom for torsion (BS:8110)

Placement of reinforcement of edge supported two-way slab in the short and long direction are as shown in the below.

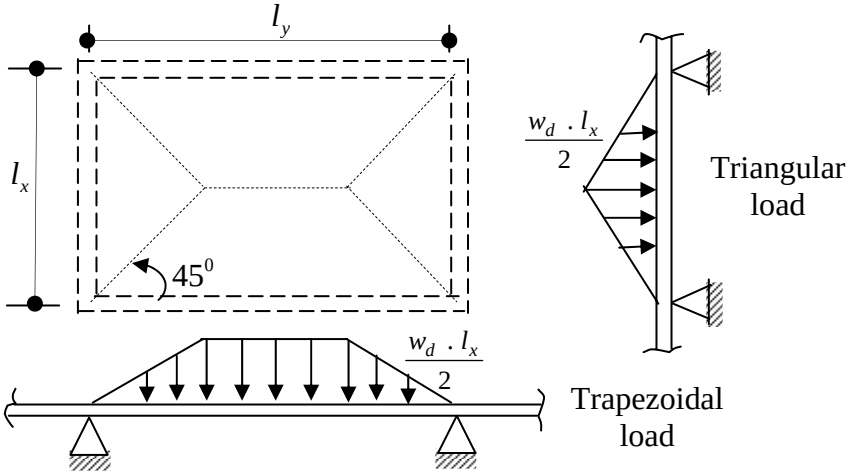


For positive reinforcement

For negative reinforcement

5.4. Loads on Supporting Beams and Maximum Shear-force of Two-way slab

The load on two-way slab transferred to the supporting beams may be assumed as the load within tributary area of slab bounded by the intersection of 45^0 line from the corners with the median line of the panel parallel to the long side as shown below. The lines that divide the load on slab to the supporting beam correspond to the assumed crack-lines of yield-line theory of slab.



According to EBCS-2/95, the design loads on supporting beam and the design shear-force of two-way slab subjected to a uniformly distributed load considering torsion at corners may be determined using the following equation.

$$V_x = \beta_{vx} \,.\, w_d \,.\, l_x$$

and $$V_y = \beta_{vy} \,.\, w_d \,.\, l_x$$

where β_{vi} --are shear-force coefficient given by the code as a function of aspect ratio, l_y / l_x and supporting condition of slab panel (refer table 3)

w_d --*service* or *factored* uniform design load depending on method of design

The design load on supporting beam is assumed to be distributed over a length of 0.75 times the span length of beam as shown below.

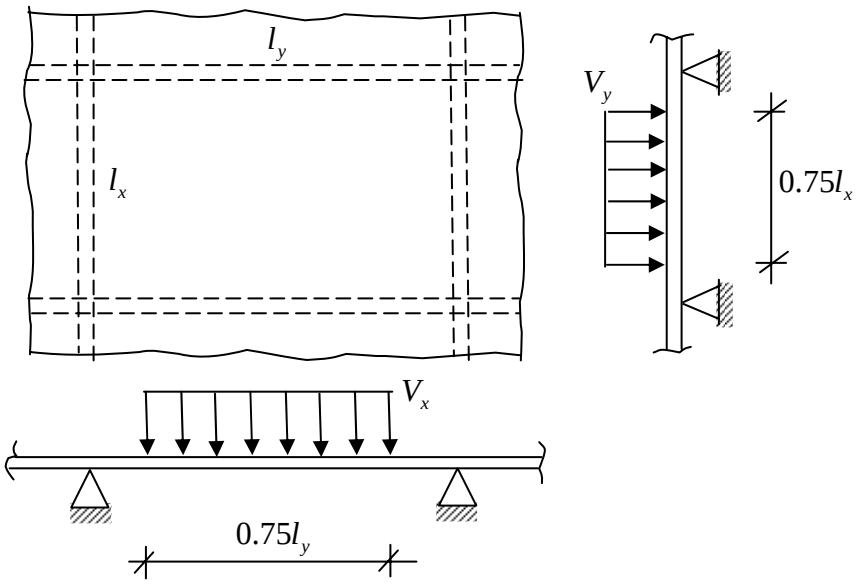
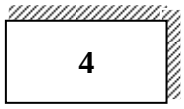
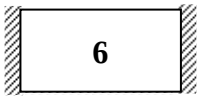
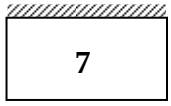
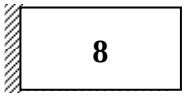
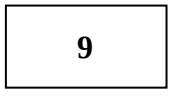
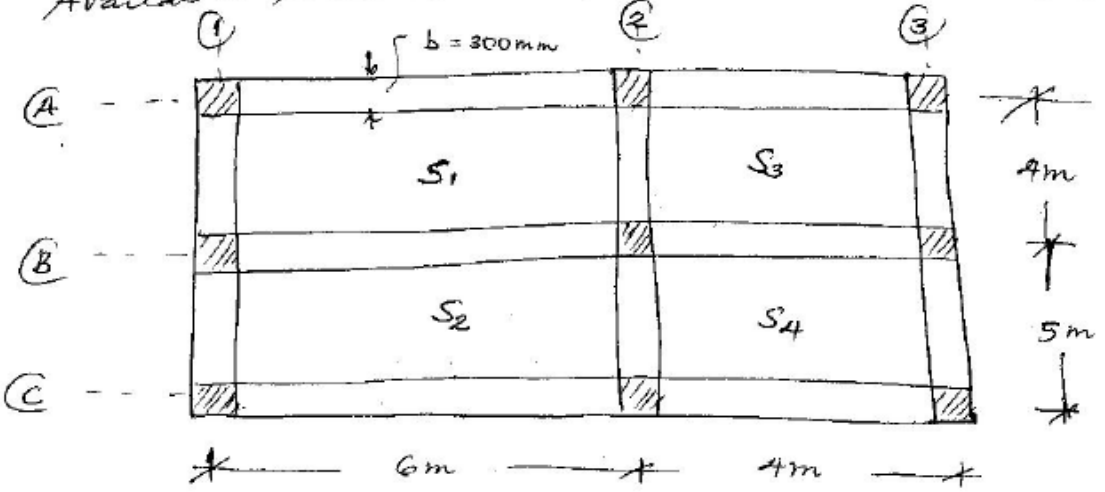


Table 3: Shear-force coefficients for uniformly loaded rectangular panels supported on four sides with provision for torsion at corner

Types of panel and location	Edge	β_{vx} for values of l_y/l_x								β_{vy}
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
 **1**	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
 **2**	Continuous	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinuous	--	--	--	--	--	--	--	--	0.24
 **3**	Continuous	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
	Discontinuous	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	--
 **4**	Continuous	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
	Discontinuous	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
 **5**	Continuous	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	--
	Discontinuous	--	--	--	--	--	--	--	--	0.26
 **6**	Continuous	--	--	--	--	--	--	--	--	0.40
	Discontinuous	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	--
 **7**	Continuous	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	--
	Discontinuous	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.30
 **8**	Continuous	--	--	--	--	--	--	--	--	0.45
	Discontinuous	0.30	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
 **9**	Discontinuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

Examples on Load Transfer to Supporting Beams

1. Design the beam on axis - 'B' for a floor system shown below, which supports live load, of 4 kN/m² and dead load from finishing and partition of 3 kN/m² in addition to self-weight. Materials used are: C-25, S-400 and class I work. Available bars are: ϕ8mm and ϕ6mm, ϕ12mm



Solⁿ :-

Step 1: Check for ~~One~~ - Way or two way slab.

S_1 : $6/4 = 1.5 < 2$

S_2 : $6/5 = 1.2 < 2$

S_3 : $4/4 = 1.0 < 2$

S_4 : $5/4 = 1.25 < 2$

} All are two way panels.

Step 2: Depth of slab:

$$d \geq \left(0.4 + \frac{0.6 f_{yk}}{400}\right) \frac{L_x}{\beta_a}$$

$$= \left(0.4 + \frac{0.6 * 400}{400}\right) \frac{L_x}{\beta_a}$$

$$d \geq \frac{L_x}{\beta_a}$$

$$\frac{l_x}{\beta_a} = \begin{cases} \frac{4000}{35} = 114.29\ mm & \text{for } S_1 \\ \frac{4000}{40} = 100\ mm & \text{for } S_3 \\ \frac{5000}{38} = 131.58\ mm & \text{for } S_2 \\ \frac{4000}{37} = 108.11\ mm & \text{for } S_4 \end{cases}$$

$\Rightarrow$ Governing depth, $d = 131.58\ mm$

Assuming Φ 12 mm is used:

$$D = 131.58 + 15 + 6 = 152.58\ mm$$

Take $D = 160\ mm$.

Step 3:- Loading:

Self-weight = $0.16 * 25 = 4\ kN/m/m$

partition & finishing = $3\ kN/m/m$

$\Rightarrow$ $G_k = 7\ kN/m/m$

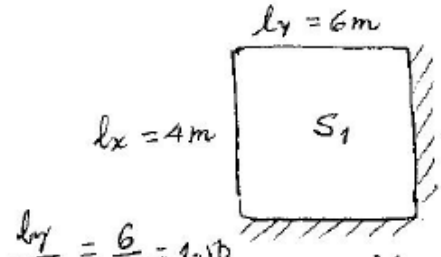
$Q_k = 4\ kN/m/m$.

Design load, $P_d = 1.3\ G_k + 1.6\ Q_k$

$= 1.3\ (7) + 1.6\ (4)$

$= 15.5\ kN/m/m$

Step 4:- Load transfer to Beams:



$\frac{l_y}{l_x} = \frac{6}{4} = 1.50$

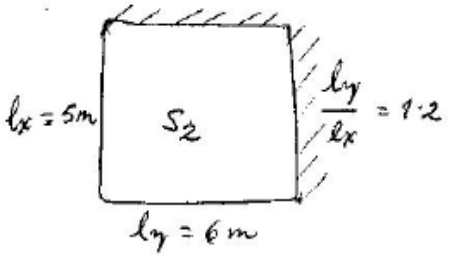
V_x (Cont.) $= \beta_{vx} P_d l_x = 0.54 * 15.5 * 4$

$= 33.48\ kN$

V_x (dis.) $= \beta_{vx} P_d l_x = 0.85 * 15.5 * 4 = 21.7\ kN$

V_y (cont.) $= \beta_{vy} P_d l_x = 0.4 * 15.5 * 4 = 24.80\ kN$

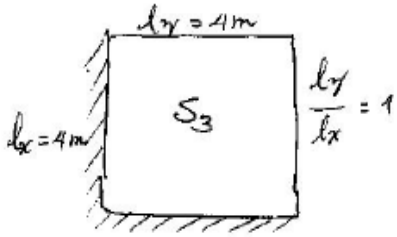
V_y (dis) $= \beta_{vy} P_d l_x = 0.26 * 15.5 * 4 = 16.12\ kN$



V_x (cont.) $= 0.47$
V_x (dis.) $= 0.31$
V_y (cont.) $= 0.40$
V_y (dis.) $= 0.26$

$* 15.5 * 5 =$

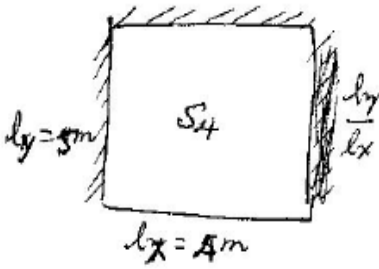
36.43 kN
24.03 kN
31.00 kN
20.15 kN



V_x (cont.) $= 0.40$
V_x (dis.) $= 0.26$
V_y (cont.) $= 0.40$
V_y (disc) $= 0.26$

$* 15.5 * 4 =$

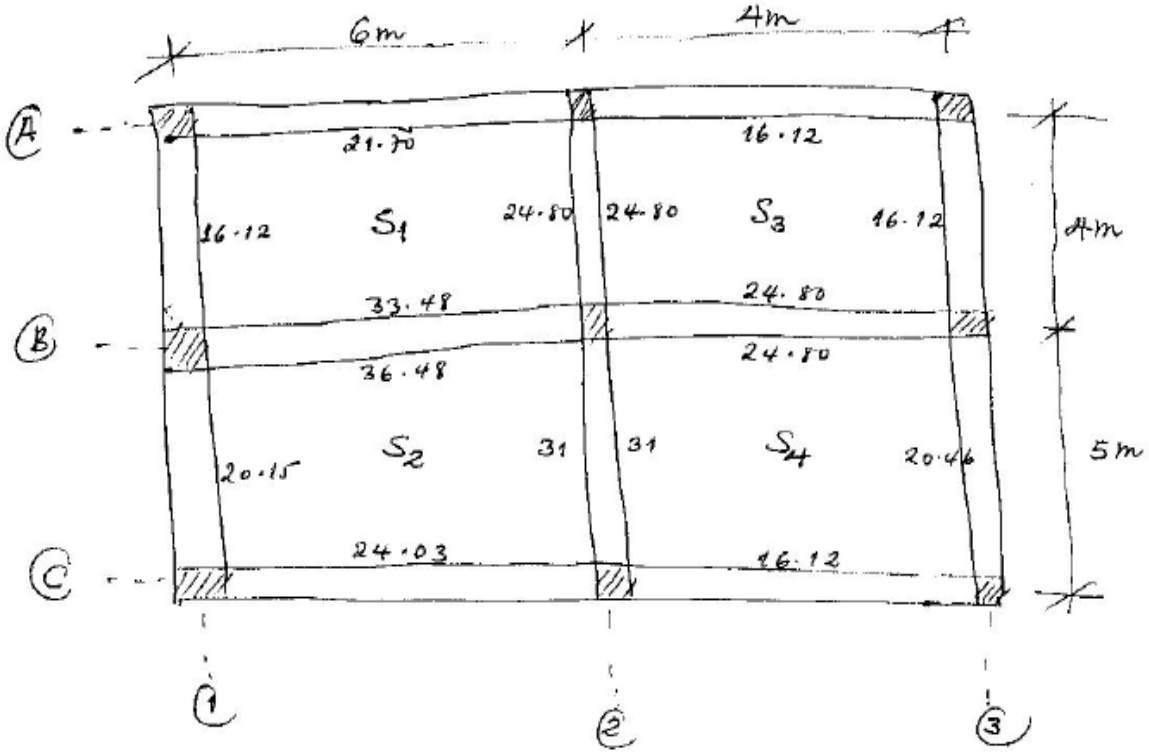
24.80 kN
16.12 kN
24.80 kN
16.12 kN



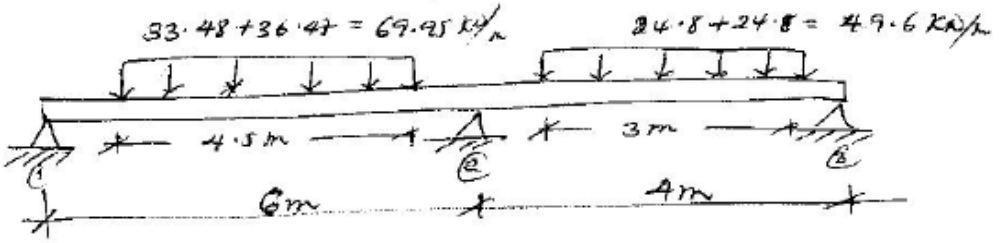
V_x (cont.) $= 0.50$
V_x (dis.) $= 0.33$
V_y (cont) $= 0.40$
V_y (dis.) $= 0.26$

$* 15.5 * 4$

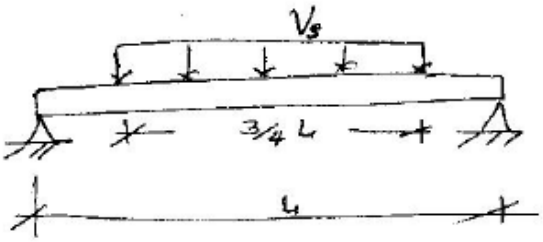
31 kN
20.46 kN
24.80 kN
16.12 kN



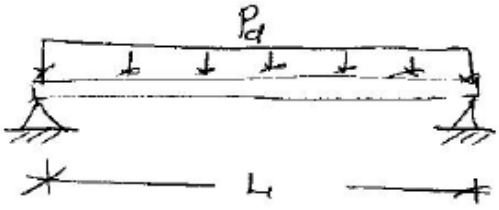
On Axis - B; the loading are shown as follow:



Step 5: Change the load into Uniformly distributed load over the whole span length.



Case I



Case II.

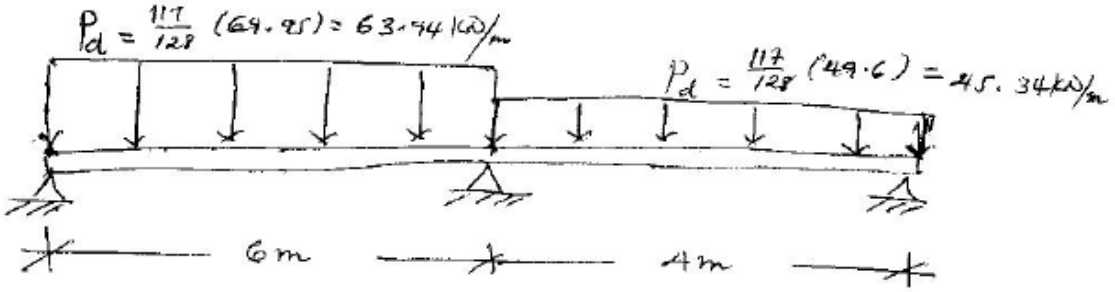
Case I : $FEM_1 = \frac{3}{4}\frac{V_s L}{24}(3L^2 - \frac{9}{16}L^2)$

Case II : $FEM_2 = \frac{P_d l^2}{12}$

Equating the two fixed end Moments;

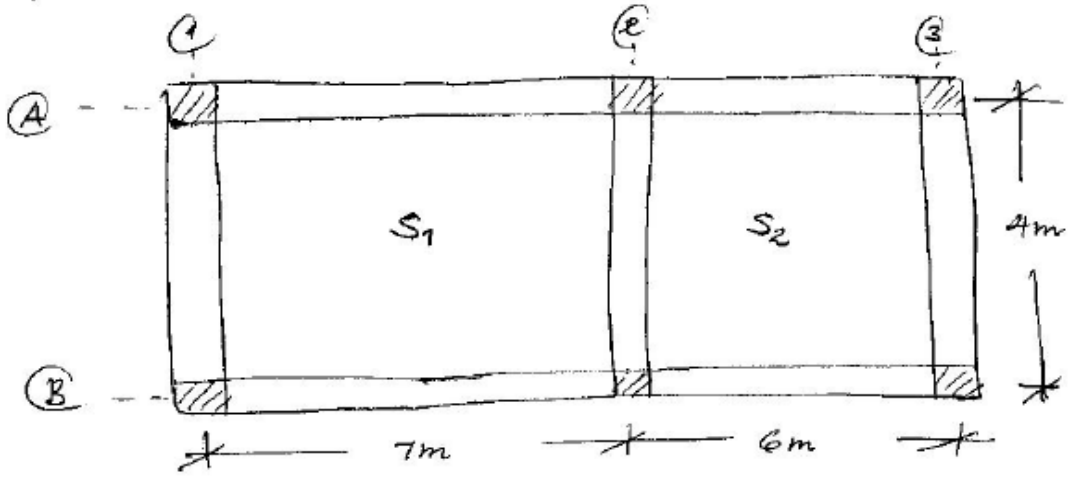
$\rightarrow \frac{3}{4}\frac{V_s L}{24}(3L^2 - 9L^2/16) = \frac{P_d l^2}{12}$

$\rightarrow \boxed{P_d = \frac{117}{128} V_s} \Leftrightarrow \boxed{P_d = 0.914 V_s}$



Step 6: Design the beam as a Continuous beam.

2. Design a beam On Axis - B for the Floor System shown below which is subjected to a live load of 5KN/m² and load from partion & Finishing of 3 KN/m². Use C-25, S-400 and Class I Work. Take The Width of beam = 300mm.



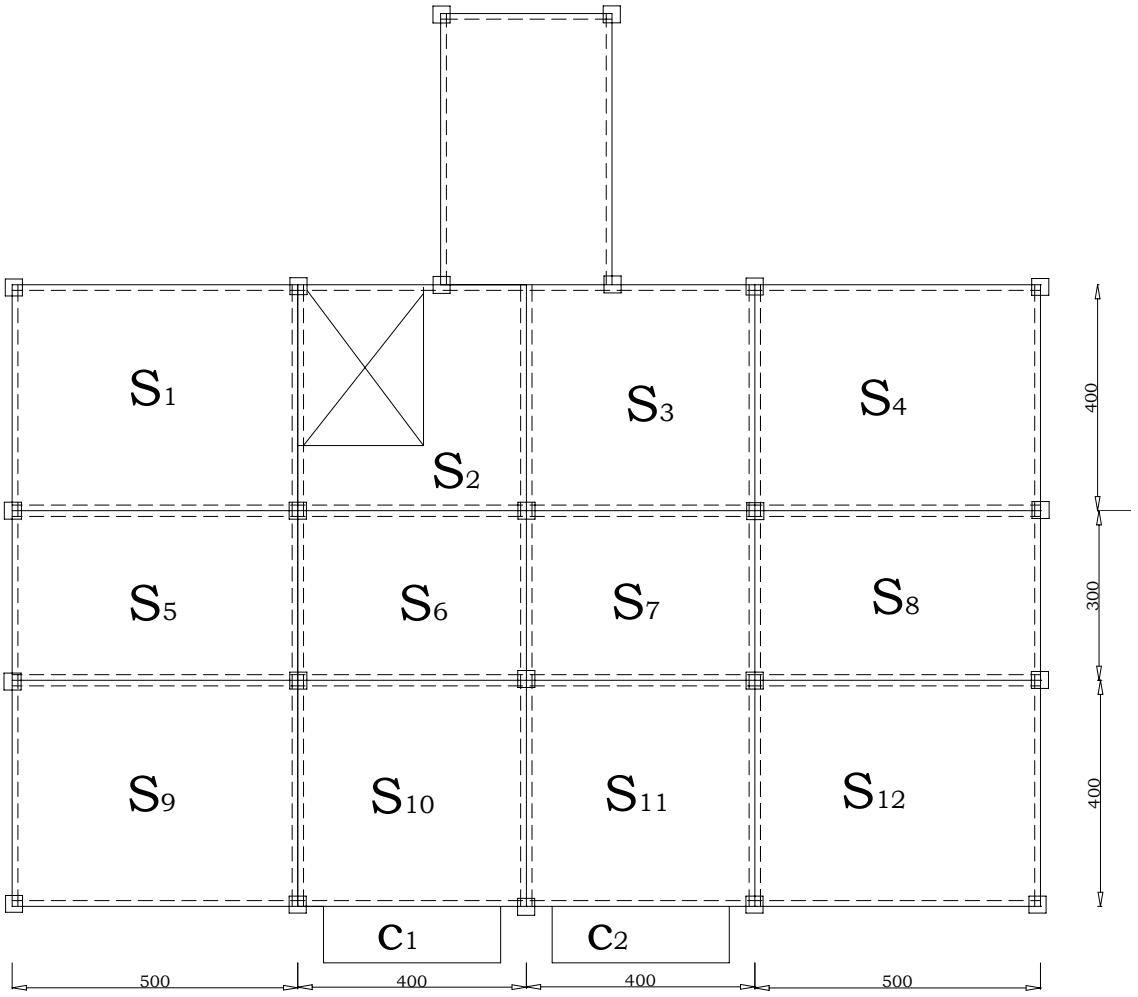
Assignment-5

Question No. 1-final design

Question No. 2

TYPICAL FLOOR SLAB DESIGN

Lay out of slabs



1.1 Depth determination

The minimum effective depth from serviceability

$$d \geq \left[0.4 + 0.6\left(\frac{f_{yk}}{400}\right)\frac{Le}{\beta_a} \right]$$ [EBCS-2,1995. Art5.2.3]

f_{yk}=300Mpa

Where,

Le = Effective span length, for two way slabs the shorter span

β_a = Constant, a function of restraint, [EBCS-2, 1995, Table 5.1]

$$d \geq \left[0.4 + 0.6\left(\frac{300}{400}\right)\frac{Le}{\beta a} \right]$$

$$\geq 0.85\frac{Le}{\beta a}$$

Slab No.	Le [mm]	Span Ratio	Span type	β_a	Effective depth
S1,S4	4000	1.25	END SPAN	37.50	90.67
S2,S3	4000	1.0	EDGE SPAN	40.00	85.00
S5,S8	3000	1.67	EDGE SPAN	33.33	76.50
S6,S7	3000	1.33	INTERIOR SPAN	41.67	61.20
S9,S12	4000	1.25	END SPAN	37.50	90.67
S10,S11	4000	1.0	EDGE SPAN	40.00	85.00
C1,C2	1000	3.10	CANTELEVER	10.00	85.00

From the serviceability results from the above table, the effective depth in S_1, S_4, S_9, S_{12} governs the design.

d = 90.67 *mm*, using Ø10mm.Reinforcement bar, 15*mm* concrete cover, Hence

D = 90.67 + 15 +1.5(10) =120.67*mm.*

Provide D = 150*mm.* thick Slab for design.

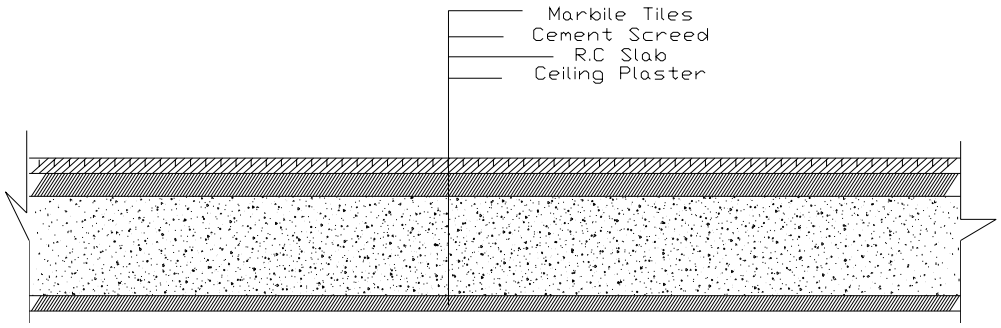
1.2 Load, shear & moment on slab.

(i) Live load

Since the building is designed for Shops and Offices Complex, Live Load of 5KN/m^2 is considered. [EBCS-1, 1995, Art 2.6.3]

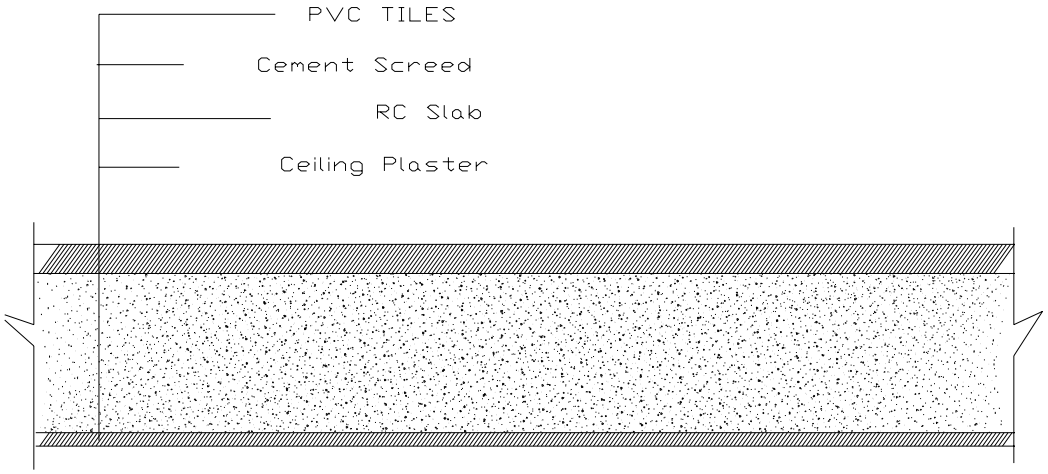
(ii) Dead Load

Slab Own weight



a) Slab S2, S3, C1, C2

150 mm RC Slab	= 0.15*25=3.75KN/m^2
20 mm Ceiling Plaster	= 0.02*23=0.46KN/m^2
30 mm Cement Screed	= 0.03*23=0.69KN/m^2
20 mm Marble Tiling	= 0.02*27=0.54KN/m^2
Total	=5.44KN/m^2



b) Slab S1, S4, S5, S6, S7, S8, S9, S10, S11, S12

150 *mm* RC Slab	= 0.15*25=3.75KN/m^2
20*mm* Ceiling Plaster	= 0.02*23=0.46KN/m^2
48*mm* Cement Screed	= 0.048*23=1.104KN/m^2
2*mm* PVC Tiles	= 0.02*16=0.32KN/m^2
Total	=5.634KN/m^2

Due to partion wall

The unit specific weight of 15*cm* thickness Pumice Hollow Block is taken the average value of 12KN/m^3. And plastering of 25mm both sides

[EBCS 1, 1995, Art 2.4.2]

Unit weight of the Glazing partition, is 27 KN/m^3 considered.
[EBCS 1, 1995, Art 2.4.2]

Weight due to partition wall = (0.15*12*2.85) + (0.025*23*2.85*2)
= 8.41KN/m
Weight due to Glazing partition = (0.006*27*2.85)
= 0.46KN/m

Slab, (S1)

P1=Weight of wall partition = 6.5*8.41
= 54.665KN
P2=Weight of Glazing partition = 2.55*0.46
= 1.15KN
P = P1 + P2
= 54.665 + 1.15
= 55.815KN

Dead Load Distribution on Slab No (S1) = $\frac{55.815}{5*4} = 2.791 KN/m2$

Unit weight of glazing per meter linear = 0.46 KN/m
Unit weight of HCB wall per meter linear = 8.41 KN/m

Slab No	Length of HCB wall (m)	Length of glazed wall (m)	P_1 (KN)	P_2 (KN)	P (KN)	Distributed dead load. (KN/m^2)
S_1	6.50	2.55	54.66	1.17	55.83	2.791
S_2	-	-	-	-	-	-
S_3	4.70	-	39.53	-	39.53	2.47
S_4	6.00	2.50	50.46	1.15	51.61	2.58
S_5	5.20	1.70	43.73	0.78	44.51	2.97
S_6	-	-	-	-	-	-
S_7	-	-	-	-	-	-
S_8	5.20	1.70	43.73	0.78	44.51	2.97
S_9	4.50	-	37.85	-	37.85	1.89
S_{10}	-	3.00	-	1.38	1.38	0.86
S_{11}	-	3.00	-	1.38	1.38	0.86
S_{12}	4.50	-	37.85	-	37.85	1.89
C_1	1.00	-	8.41	-	8.41	2.71
C_2	1.00	-	8.41	-	8.41	2.71

Factored Load

The Factored Design Load is ⇨ $P_d = 1.3\ G_k + 1.6\ Q_k$ [EBCS-2, 1995]

Slab (S1)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+2.791) + 1.6(5)$
$= 18.95\ KN/m^2$

Slab number 2 treated separately in section

Slab (S3)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.44+2.46) + 1.6(5)$
$= 18.27 KN/m^2$

Slab (S4)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+2.579) + 1.6(5)$
$= 18.68\ KN/m^2$

Slab (S5)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+2.97) + 1.6(5)$
$= 19.18$ KN/m2

Slab (S6)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634) +1.6(5)$
$= 15.32\ KN/m^2$

Slab (S7)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634) +1.6(5)$
1.6(5)
$= 15.32\ KN/m^2$

Slab (S8)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+2.97) +1.6(5)$
1.6(5)
$= 19.18\ KN/m^2$

Slab (S9)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+1.89) + 1.6(5)$
$= 17.78\ KN/m^2$

Slab (S10)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+1.38) + 1.6(5)$
$= 17.11\ KN/m^2$

Slab (S11)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+1.38) +$
$= 17.11\ KN/m^2$

Slab (S12)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.634+1.89) +$
$= 17.78\ KN/m^2$

Slab (Cantilever C1)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.44+2.71) + 1.6(5)$
$= 18.59\ KN/m^2$

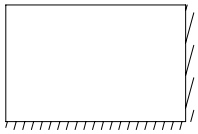
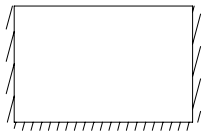
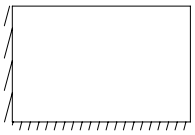
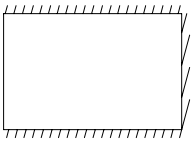
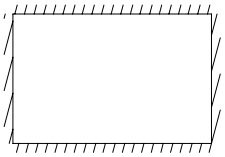
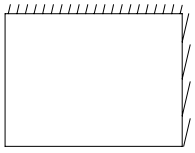
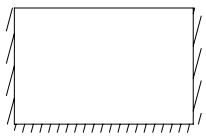
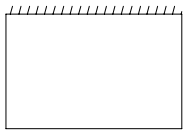
Slab (Cantilever C2)

$P_d = 1.3\ G_k + 1.6\ Q_k$
$= 1.3(5.44+2.71) + 1.6(5)$
$= 18.59\ KN/m^2$

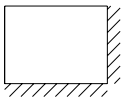
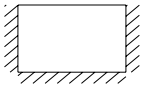
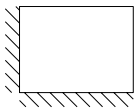
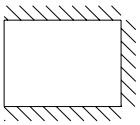
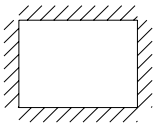
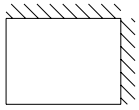
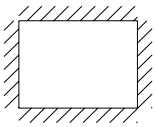
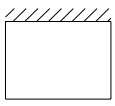
Moment distribution for each Panel

The following Bending moment coefficient for regular Panel supports on the side with provision for torsion at corner.

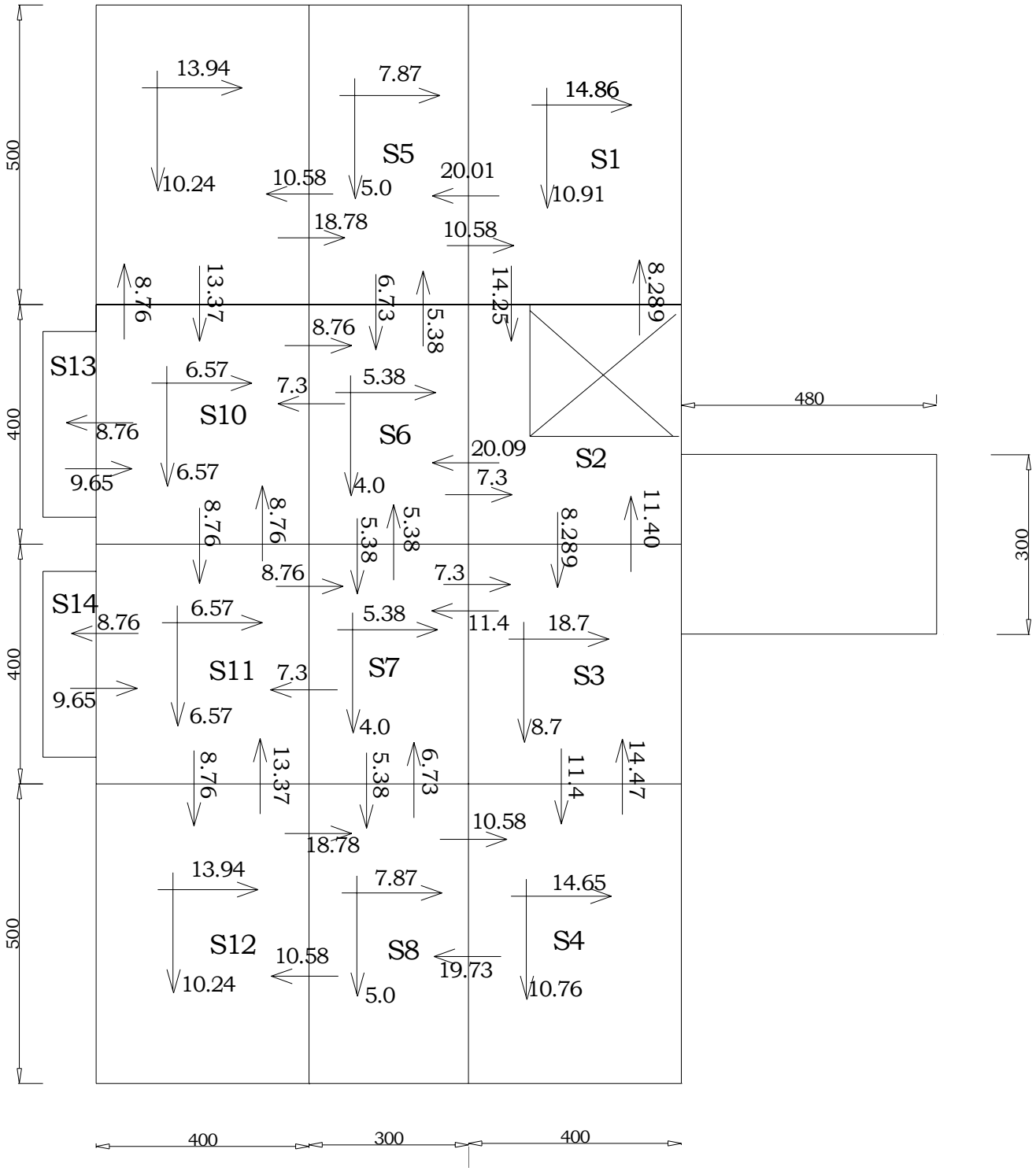
[EBCS 1, 1995 Table A-1]

Panel No	Support condition	$\frac{Ly}{Lx}$	Moment coefficients	Bending moment $M_i = \alpha_i P_b L_x^2 [KN-m/m]$
S_1		1.25	$\alpha_{xs}=0.066$ $\alpha_{ys}=0.047$ $\alpha_{xf}=0.049$ $\alpha_{yf}=0.036$	$M_{xs}=0.066*18.95*4^2=20.01$ $M_{ys}=0.047*18.95*4^2=14.25$ $M_{xf}=0.049*18.95*4^2=14.86$ $M_{yf}=0.036*18.95*4^2=10.91$
S_3		1.00	$\alpha_{xs}=0.039$ $\alpha_{ys}=0.039$ $\alpha_{xf}=0.030$ $\alpha_{yf}=0.030$	$M_{xs}=0.039*18.27*4^2=11.40$ $M_{ys}=0.039*18.27*4^2=11.40$ $M_{xf}=0.03*18.27*4^2=8.77$ $M_{yf}=0.03*18.27*4^2=8.77$
S_4		1.25	$\alpha_{xs}=0.066$ $\alpha_{ys}=0.047$ $\alpha_{xf}=0.049$ $\alpha_{yf}=0.036$	$M_{xs}=0.066*18.68*4^2=19.73$ $M_{ys}=0.047*18.68*4^2=14.47$ $M_{xf}=0.049*18.68*4^2=14.65$ $M_{yf}=0.036*18.68*4^2=10.76$
S_5,S_8		1.67	$\alpha_{xs}=0.0613$ $\alpha_{ys}=0.039$ $\alpha_{xf}=0.0456$ $\alpha_{yf}=0.029$	$M_{xs}=0.0613*19.18*3^2=10.58$ $M_{ys}=0.039*19.18*3^2=6.73$ $M_{xf}=0.0456*19.18*3^2=7.87$ $M_{yf}=0.029*19.18*3^2=5.00$
S_6, S_7		1.33	$\alpha_{xs}=0.053$ $\alpha_{ys}=0.039$ $\alpha_{xf}=0.039$ $\alpha_{yf}=0.029$	$M_{xs}=0.053*15.32*3^2=7.30$ $M_{ys}=0.039*15.32*3^2=5.38$ $M_{xf}=0.039*15.32*3^2=5.38$ $M_{yf}=0.029*15.32*3^2=4.00$
S_9,S_{12}		1.25	$\alpha_{xs}=0.066$ $\alpha_{ys}=0.047$ $\alpha_{xf}=0.049$ $\alpha_{yf}=0.036$	$M_{xs}=0.066*17.78*4^2=18.78$ $M_{ys}=0.047*17.78*4^2=13.37$ $M_{xf}=0.049*17.78*4^2=13.94$ $M_{yf}=0.036*17.78*4^2=10.24$
S_{10},S_{11}		1.00	$\alpha_{xs}=0.032$ $\alpha_{ys}=0.032$ $\alpha_{xf}=0.024$ $\alpha_{yf}=0.024$	$M_{xs}=0.032*17.11*4^2=8.76$ $M_{ys}=0.032*17.11*4^2=8.76$ $M_{xf}=0.024*17.11*4^2=6.57$ $M_{yf}=0.024*17.11*4^2=6.57$
C_1,C_2		3.1	-	$M_{ys}=\frac{wl^2}{2}=\frac{19.3*1^2}{2}=9.65KN/m$

Shear force for un-factored live load & Dead load

Panel No	Support condition	Shear Coefficient β_{ij}	Shear for live load $V_i = \beta_{vi} q_k L_{xi}$	Shear For dead load $V_i = \beta_{vi} g_k L_{xi}$
S_1		$\beta_{vxc} = 0.485$ $\beta_{vxd} = 0.315$ $\beta_{vyc} = 0.400$ $\beta_{vyd} = 0.260$	$V_{xc} = 0.485*5*4 = 9.7$ $V_{xd} = 0.315*5*4 = 6.3$ $V_{yc} = 0.4*5*4 = 8.0$ $V_{yd} = 0.26*5*4 = 5.20$	$V_{xc} = 0.485*8.425*4 = 16.34$ $V_{xd} = 0.315*8.425*4 = 10.62$ $V_{yc} = 0.4*8.425*4 = 13.4$ $V_{yd} = 0.26*8.425*4 = 8.76$
S_3		$\beta_{vxc} = 0.360$ $\beta_{vxd} = 0.240$ $\beta_{vyc} = 0.360$ $\beta_{vyd} = 0.000$	$V_{xc} = 0.36*5*4 = 7.2$ $V_{xd} = 0.24*5*4 = 4.8$ $V_{yc} = 0.36*5*4 = 7.2$ $V_{yd} = 0.00$	$V_{xc} = 0.36*7.9*4 = 11.37$ $V_{xd} = 0.24*7.9*4 = 7.58$ $V_{yc} = 0.36*7.9*4 = 11.37$ $V_{yd} = 0.00$
S_4		$\beta_{vxc} = 0.485$ $\beta_{vxd} = 0.315$ $\beta_{vyc} = 0.400$ $\beta_{vyd} = 0.260$	$V_{xc} = 0.485*5*4 = 9.7$ $V_{xd} = 0.315*5*4 = 6.3$ $V_{yc} = 0.4*5*4 = 8.0$ $V_{yd} = 0.26*5*4 = 5.2$	$V_{xc} = 0.485*8.213*4 = 15.93$ $V_{xd} = 0.315*8.213*4 = 10.35$ $V_{yc} = 0.4*8.213*4 = 13.14$ $V_{yd} = 0.26*8.213*4 = 8.54$
S_5,S_8		$\beta_{vxc} = 0.489$ $\beta_{vxd} = 0.000$ $\beta_{vyc} = 0.360$ $\beta_{vyd} = 0.240$	$V_{xc} = 0.489*5*3 = 7.33$ $V_{xd} = 0.00$ $V_{yc} = 0.36*5*3 = 5.4$ $V_{yd} = 0.24*5*3 = 3.6$	$V_{xc} = 0.489*8.604*3 = 12.62$ $V_{xd} = 0.00$ $V_{yc} = 0.36*8.604*3 = 9.29$ $V_{yd} = 0.24*8.604*3 = 6.19$
S_6, S_7		$\beta_{vxc} = 0.416$ $\beta_{vxd} = 0.00$ $\beta_{vyc} = 0.330$ $\beta_{vyd} = 0.00$	$V_{xc} = 0.416*5*3 = 6.24$ $V_{xd} = 0.00$ $V_{yc} = 0.33*5*3 = 4.95$ $V_{yd} = 0.00$	$V_{xc} = 0.416*5.634*3 = 7.03$ $V_{xd} = 0.00$ $V_{yc} = 0.33*5.634*3 = 5.57$ $V_{yd} = 0.00$
S_9,S_{12}		$\beta_{vxc} = 0.485$ $\beta_{vxd} = 0.315$ $\beta_{vyc} = 0.400$ $\beta_{vyd} = 0.260$	$V_{xc} = 0.485*5*4 = 9.7$ $V_{xd} = 0.315*5*4 = 6.3$ $V_{yc} = 0.4*5*4 = 8.00$ $V_{yd} = 0.26*5*4 = 5.2$	$V_{xc} = 0.485*7.524*4 = 14.60$ $V_{xd} = 0.315*7.524*4 = 9.48$ $V_{yc} = 0.4*7.524*4 = 12.0$ $V_{yd} = 0.26*7.524*4 = 7.82$
S_{10},S_{11}		$\beta_{vxc} = 0.330$ $\beta_{vxd} = 0.330$ $\beta_{vyc} = 0.330$ $\beta_{vyd} = 0.000$	$V_{xc} = 0.33*5*4 = 6.60$ $V_{xd} = 0.00$ $V_{yc} = 0.33*5*4 = 6.60$ $V_{yd} = 0.00$	$V_{xc} = 0..*7.014*4 = 9.25$ $V_{xd} = 0.00$ $V_{yc} = 0.33*7.014*4 = 9.25$ $V_{yd} = 0.00$
C_1,C_2		-	$V_{xc} = 5*1 = 5$	$V_{xc} = 8.14*1 = 8.14$

1.2.1 Balancing of support moments



Panel No.	Balanced Support Moment by Moment Distribution method					Adjusted span Moment	
S_1 & S_5	S_1				S_5	M_{xf}	M_{yf}
	K=4*(I/4)=1 DF = 0.428 Unbalanced moment 20.01 KN-m/m			K=4*(I/3)=1.33 DF =0.571 Unbalanced moment 10.58 KN-m/m		S_1=16.31	12.35
	= -0.428 * (20.01-10.58)+ 20.01 = 15.974 KN-m/m			= 0.571 *(20.01 – 10.58)+ 10.58 = 15.974 KN-m/m			
	If the difference Between moments below 20% take average of the two						
	Panel No.	K	DF	Unbalanced Moment	Balanced Moment		
S_3 , S_4	S3	1.00	0.555	11.4	13.104	9.784	9.80
	S4	0.80	0.444	14.47	13.104	16.512	12.37
S_5 , S_6	S5	0.80	0.444	6.73	6.13	8.042	5.05
	S6	1.00	0.555	5.38	6.13	5.38	4.00
S_6 , S_7	S6	-	-	5.38	5.38		
	S7	-	-	5.38	5.38	5.38	4.00
S_7 ,S_8	S7	1.00	0.555	5.38	6.13		
	S8	0.80	0.444	6.73	6.13	8.042	5.05
S_9 ,S_{10}	S9	0.80	0.444	13.37	11.32	15.87	11.81
	S10	1.00	0.555	8.76	11.32	6.77	6.85
S_{10} S_{11}	S10	-	-	8.76	8.76		
	S11	-	-	8.76	8.76	6.77	6.85
S_{11} S_{12}	S11	1.00	0.555	8.76	11.32		
	S12	0.80	0.444	13.37	11.32	15.724	11.65
S5 , S9	S5	1.33	0.571	10.58	15.27		
	S9	1.00	0.428	18.78	15.27		
S6 , S10	S6	-	-	8.03	8.03		
	S10	-	-	8.03	8.03		
S3 , S7	S3	1.00	0.428	11.4	9.64		
	S7	1.33	0.571	7.3	9.64		
S7 , S11	S7	Less than 20% take average		7.3	8.03		
	S11			8.76	8.03		
S4 , S8	S4	1.00	0.428	19.73	15.8		
	S8	1.33	0.571	10.58	15.8		
S8 , S12	S8	1.33	0.571	10.58	15.27		
	S12	1.00	0.428	18.78	15.27		
C1 , S10	C1	Less than 20% take average		9.65	9.205		
	S10			8.76	9.205		
C2 , S11	C2	Less than 20% take average		9.65	9.205		
	S11			8.76	9.205		

1.2.2 Adjustment of span moments

Slab number (S_1)

M_{xs}=20.01-15.974 = 4.036

For $\frac{L_Y}{L_X} = \frac{5}{4} = 1.25$ [EBCS-2, 1995 Table A-2]

C_x = 0.3585

C_y = 0.357

$\Delta M_{xf} = C_x * \Delta M = 0.3585(4.036) = 1.4469$

$\Delta M_{yf} = C_y * \Delta M = 0.357(4.036) = 1.44$

Adjusted moment

M_{xf} = 14.86+1.4469 = 16.306

M_{yf} = 10.91+1.44 = 12.35

Slab number (S_3)

M_{ys} = 11.4-9.64=1.76

For $\frac{L_y}{L_x} = \frac{4}{4} = 1$

C_x = 0.28

C_y =0.38

$\Delta M_{xf} = C_x * \Delta M = 0.28(1.76) = 0.4928$

$\Delta M_{yf} = C_y * \Delta M = 0.38(1.76) = 0.6688$

Adjusted moment

M_{xf} = 8.7 + 0.4928 = 9.192

M_{yf} = 8.7 + 0.6688 = 9.368

Slab number (S_4)

Mxs =19.73-15.8 = 3.93

Mys = 14.47-13.103 = 1.367

For $\frac{L_Y}{Lx} = \frac{5}{4} = 1$

C_{x1}= 0.3585

C_{y1}= 0.357

C_{x2}= 0.332

C_{y2}= 0.154

$\Delta M_{xf1} = C_{x1} * \Delta M = 0.3585 * 3.93 = 1.408$
$\Delta M_{yf1} = C_{y1} * \Delta M = 0.357 * 3.93 = 1.403$
$\Delta M_{xf2} = C_{x2} * \Delta M = 0.332 * 1.369 = 0.4538$
$\Delta M_{yf2} = C_{y2} * \Delta M = 0.154 * 1.369 = 0.2105$

Adjusted moment

$M_{xf} = 14.65 + 1.408 + 0.4538 = 16.511$
$M_{yf} = 10.76 + 1.403 + 0.2105 = 12.373$

Slab number (S_5) & (S_8)

$M_{ys} = 6.73 - 6.13 = 0.6$

For $\frac{L_y}{L_x} = \frac{5}{3} = 1.667$

$C_x = 0.2883$
$C_y = 0.07697$

$\Delta M_{xf} = C_x * \Delta M = 0.2883 * 0.6 = 0.17298$
$\Delta M_{yf} = C_y * \Delta M = 0.07697 * 0.6 = 0.0461$

Adjusted moment

Mxf = 7.87+0.17298=8.042
Myf = 5.00+0.0461=5.0461

Slab number (S_9) and (S_{12})

$M_{xs} = 18.78 - 15.27 = 3.51$
$M_{ys} = 13.37 - 11.32 = 2.05$

For $\frac{L_y}{L_x} = \frac{5}{4} = 1.25$

$C_{x1} = 0.3585$
$C_{y1} = 0.357$
$C_{x2} = 0.332$
$C_{y2} = 0.154$

$\Delta M_{xf1} = C_{x1} * \Delta M = 0.3585 * 3.51 = 1.258$
$\Delta M_{yf1} = C_{y1} * \Delta M = 0.357 * 3.51 = 1.253$
$\Delta M_{xf2} = C_{x2} * \Delta M = 0.332 * 2.05 = 0.6806$
$\Delta M_{xf2} = C_{y2} * \Delta M = 0.154 * 2.05 = 0.3157$

Adjusted moment

M_{xf} = 13.94+1.258+0.6806=15.8706
M_{yf} = 10.24+1.253+0.3157=11.8087

Slab number (S_{10}) and (S_{11})

M_{xs} = 8.76-8.03=0.73

For $\frac{L_y}{L_x} = \frac{4}{4} = 1$

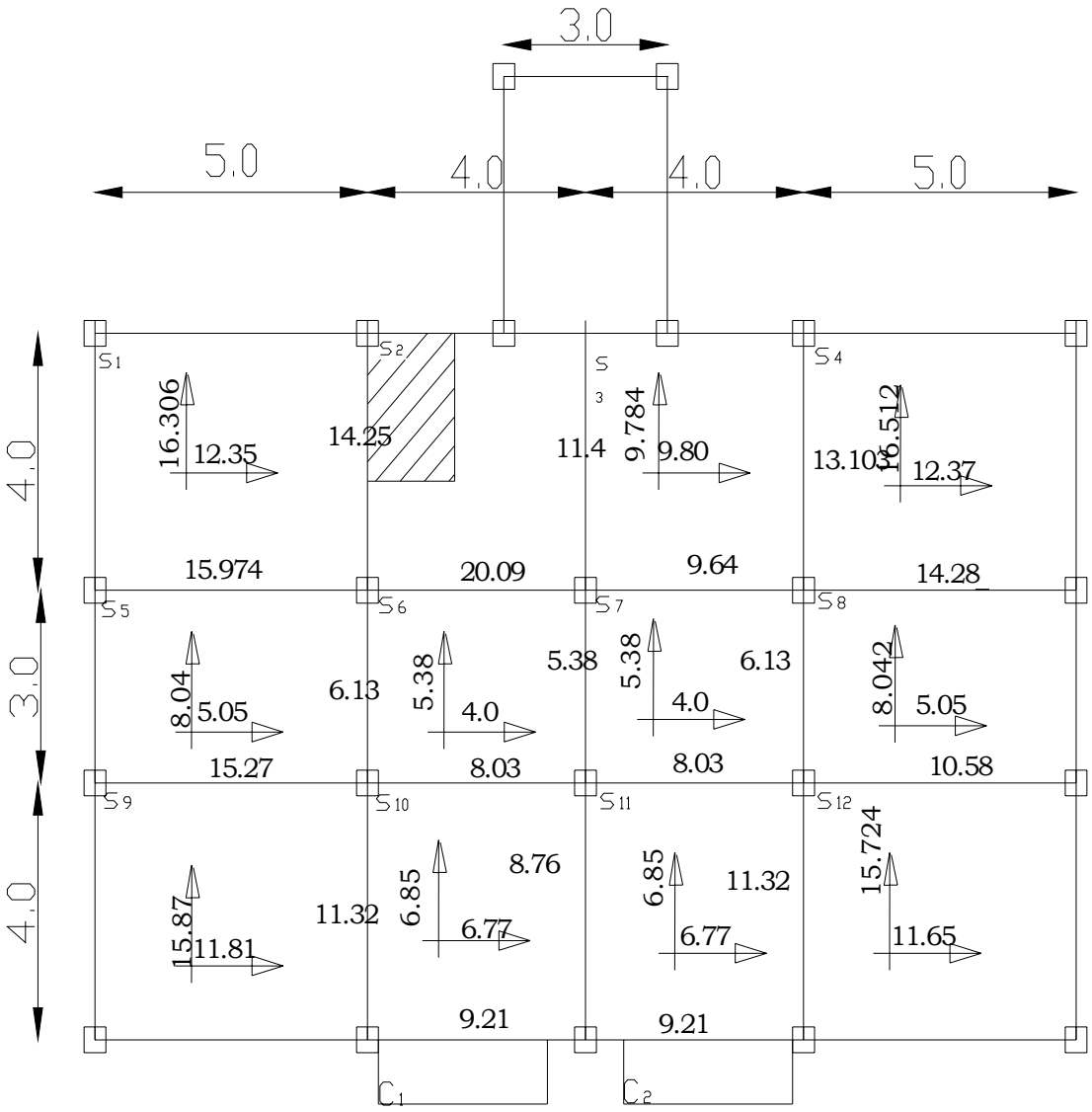
C_x = 0.28
C_y =0.38

$\Delta M_{xf} = C_x * \Delta M = 0.28*0.73=0.2044$
$\Delta M_{yf} = C_y * \Delta M = 0.38*0.73=0.2774$

Adjusted moment

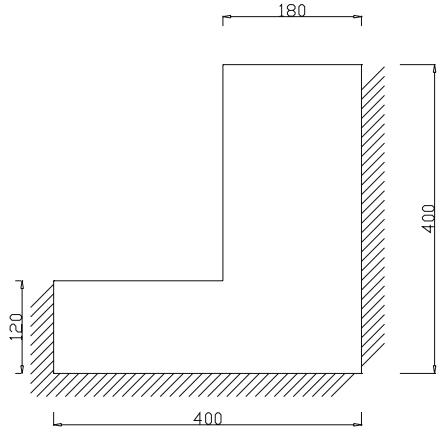
M_{xf} = 6.57+0.2044=6.774
M_{yf} = 6.57+0.2774=6.847

Adjusted design support & span moments



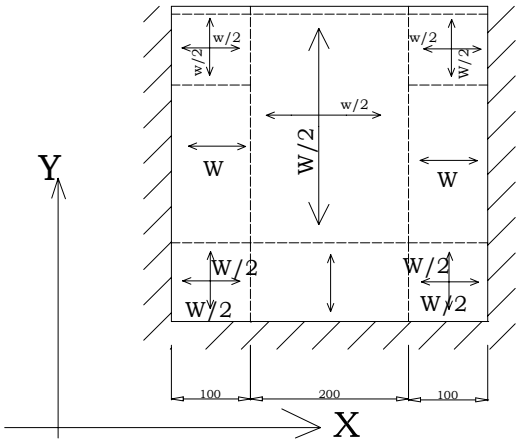
1.3 Analysis and design of slab No 2

Slab with corner opening, panel S_2 [By strip method]



Moment with out opening [basic case]

Dead load = 5.44KN/m²
Live load = 5KN/m²
Design Load = 1.3 * 5.44 + 5 *1.6 = 15.072 KN/m²



αb/2

(1-α)b/2

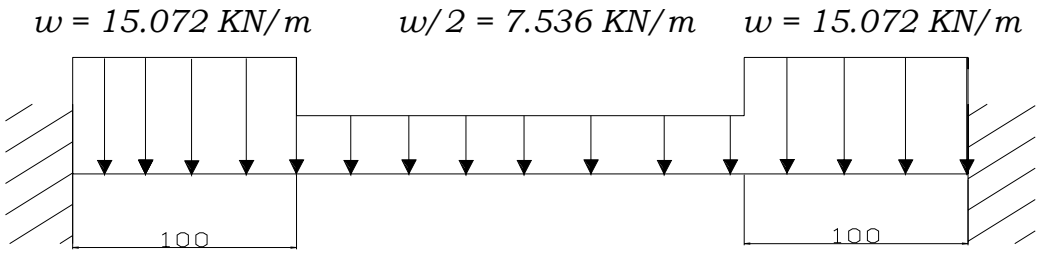
Since the slab is square, then taking the ratio of M_{xs} and M_{xf} = 2 ,⇨ $\alpha = 0.366$

Hence

$$\alpha\frac{b}{2} = 0.366 * \frac{4}{2} = 0.73$$

$$(1-\alpha)\frac{b}{2} = (1-0.366) * \frac{4}{2} = 1.27$$

X- DIRECTION



Cantilever moment

M_x = 7.536 * 2 * 1 * + 7.536 * 1 * 0.5
= 18.84 KN-m/m

Field and support moments will be calculated using the ratio of two

M_{xf} = 1/3 * 18.84 = 6.28KN-m/m

M_{xs} = 2/3 * 18.84 = 12.56 KN-m/m

edge strip to the fixed side

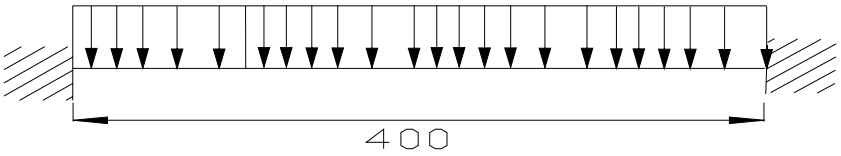


$$M_{xs} = \frac{wb^2}{96} = \frac{15.072 * 4^2}{96} = 2.512 KN - m/m$$

$$M_{xf} = \frac{wb^2}{192} = \frac{15.072 * 4^2}{192} = 1.256 KN - m/m$$

Edge strip to simply supported side

7.536 KN/m



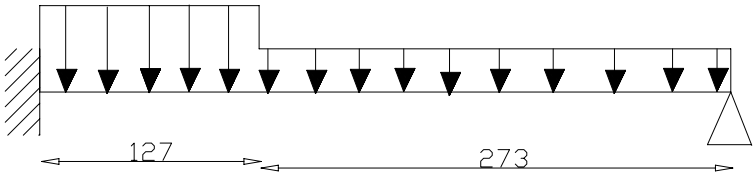
$$M_{xs} = \frac{wb^2}{12} = \frac{7.536 * 4^2}{12} = 10.048\,KN - m/m$$

$$M_{xf} = \frac{wb^2}{24} = \frac{7.536 * 4^2}{24} = 5.024\,KN - m/m$$

Y- DIRECTION

Middle strip

15.072 KN/m *7.536KN/m*



Cantilever moment

$$M_x = 15.072 * 1.27^2 + 7.532 * 0.73 * (1.27 + \frac{0.73}{2})$$

$$= 21.144KN - m/m$$

$$M_{xs} = \frac{2}{3} * 21.144 = 14.10KN - m/m$$

$$M_{ys} = \frac{1}{3} * 21.144 = 7.08KN - m/m$$

Edge strip



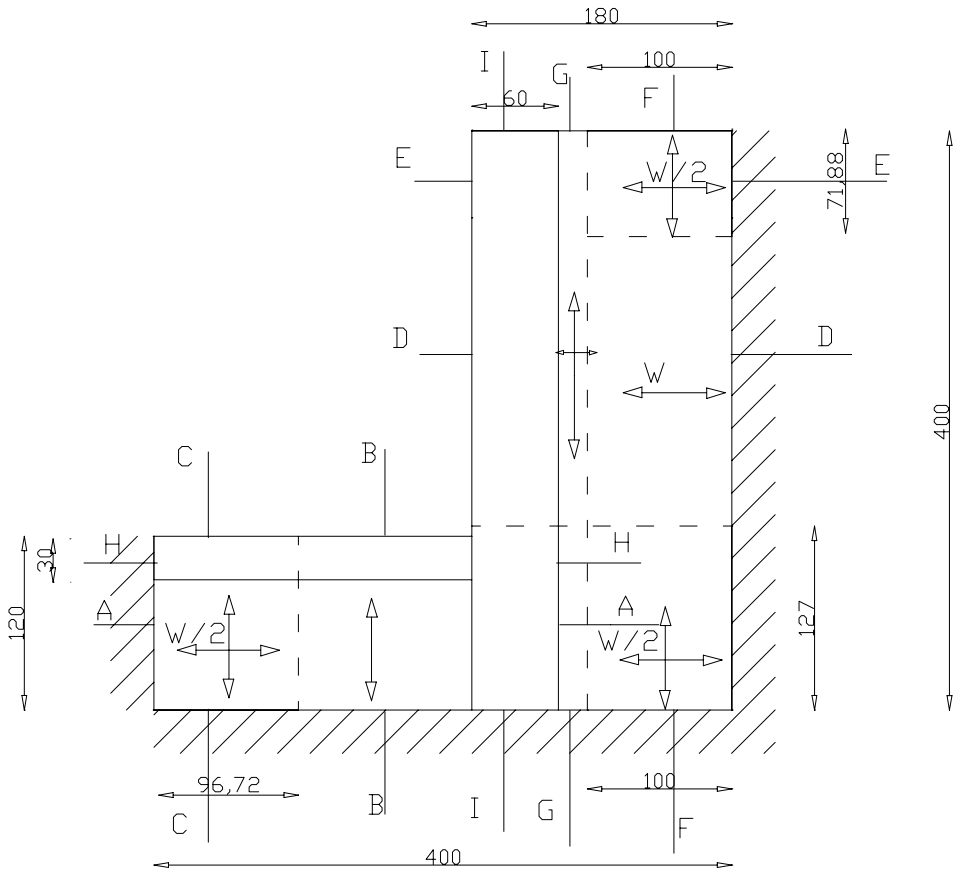
Cantilever moment

$$M_x = 7.536 * \frac{1.27^2}{2} = 6.077KN - m/m$$

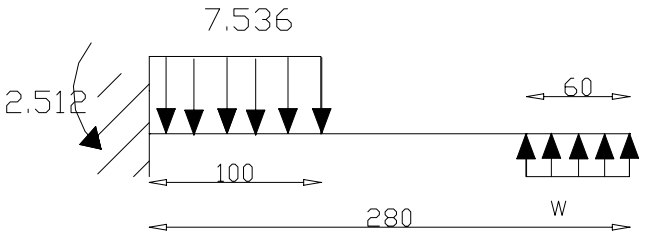
$$M_{xs} = \frac{2}{3} * 6.077 = 4.051 KN - m/m$$

$$M_{ys} = \frac{1}{3} * 6.077 = 2.025 KN - m/m$$

In order to support the slab strip cut by the hole an arrangement of strong band provided as shown in the sketch



Strip A –A



2.512 – 7.536 * 1 * 0.5 + 0.6 * w * 2.5 = 0 ⇨ W = 0.84

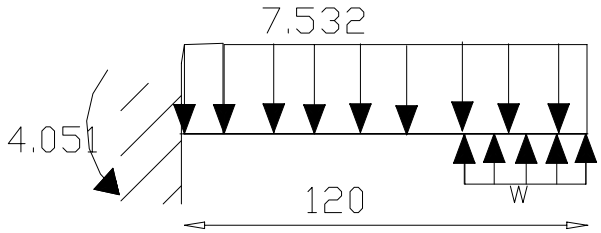
Cantilever moment

$$M_x = \frac{7.532 * 1^2}{2} = 3.768 KN - m/m$$

$$M_{xs} = \frac{2}{3} * 3.768 = 2.51 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 3.768 = 1.256 KN - m/m$$

Strip B –B



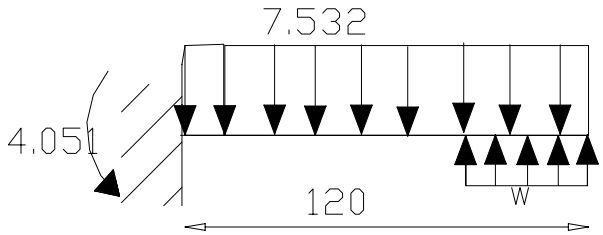
14.095 + 0.3 * w * 1.05 – 15.072 * 1.2 * 0.6 = 0 ⇨ w = -10.215 KN-m/m

Cantilever moment $\frac{15.072 * 2^2}{2} = 10.85 KN - m/m$

$$M_{ys} = \frac{2}{3} * 10.85 = 7.234 KN - m/m$$

$$M_{yf} = \frac{1}{3} * 10.85 = 3.617 KN - m/m$$

Strip C – C



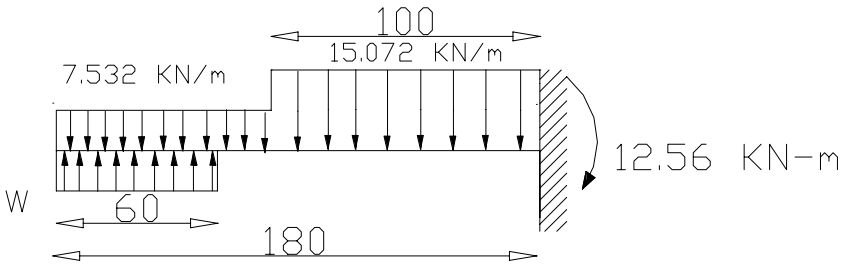
4.051 * 7.536 * 1.2 * 0.6 + 0.3 * w * 1.05 = 0 ⇨ w = 4.36 KN/m

Cantilever moment $M_y = \frac{7.536 * 1.2^2}{2} = 5.425 KN - m/m$

$$M_{ys} = \frac{2}{3} * 5.425 = 3.617 KN - m/m$$

$$M_{yf} = \frac{1}{3} * 5.425 = 1.808 KN - m/m$$

Strip D –D



12.56 – 15.072 * 1.0 * 0.5 – 7.536 * 0.8 * 1.4 + 0.6 * w * 1.5 = 0

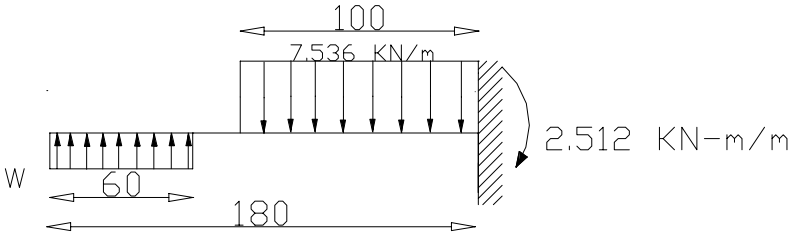
⇨ w = 2.80 KN/m

Cantilever moment = $15.072 * 1 * 0.5 = 7.536 KN - m/m$

$$M_{xs} = \frac{2}{3} * 7.536 = 5.024 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 7.536 = 2.512 KN - m/m$$

Strip E – E



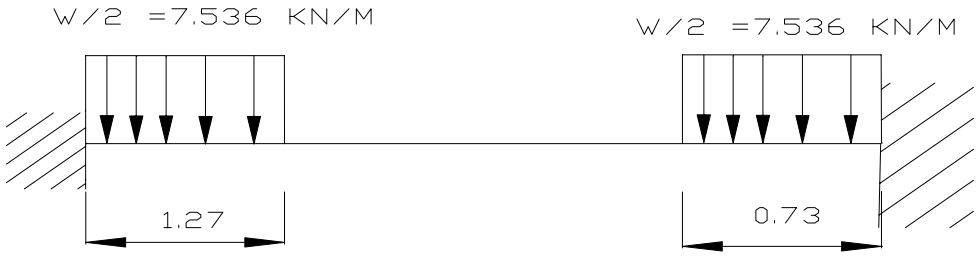
2.512 – 7.536 * 1 * 0.5 + 0.6 * w * 1.5 = 0 ⇨ w = 3.7955KN/m

Cantilever moment = $\frac{7.536 * 1}{2} = 3.768 KN - m/m$

$$M_{xs} = \frac{2}{3} * 3.768 = 2.512 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 3.768 = 1.256 KN - m/m$$

Strip F –F

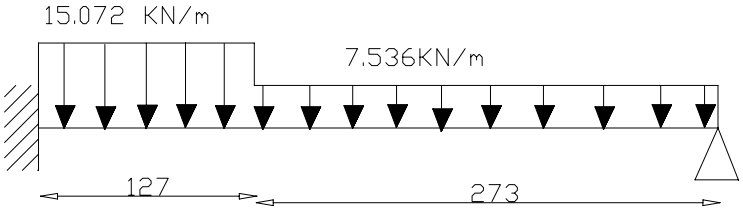


Cantilever moment = $1.27 * 7.536 * \frac{1.27}{2} = 6.077 KN - m/m$

$$M_{xs} = \frac{2}{3} * 6.077 = 4.051 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 6.077 = 2.025 KN - m/m$$

Strip G-G



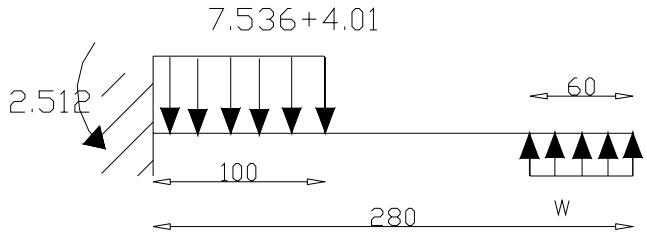
Cantilever moment =

$$15.072 * \frac{1.27^2}{2} + 7.536 * 0.73 * \left(1.27 * \frac{0.73}{2}\right) = 21.1488 KN - m/m$$

$$M_{xs} = \frac{2}{3} * 21.1488 = 14.099 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 21.1488 = 7.049 KN - m/m$$

Strip H – H



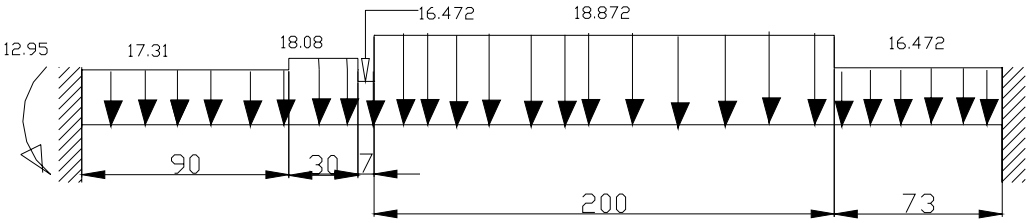
2.512 – (7.536+4.01)*1.0 * 0.5 + w * 0.6 * 2.5 = 0 ⇨ w = 2.174 KN/m

Cantilever moment M = (7.536 + 4.01) * 1 * 0.5 = 5.773 KN/m

M_{xs} = 2/3(5.773) = 3.848 KN/m

M_{xf} = 1/3(5.773) = 1.924 KN/m

Strip I – I



Cantilever Moment

$$= 17.31 * \frac{0.9^2}{2} + 18.08 * 0.3 * 1.05 + 0.07 * 16.472 * 1.235 + 0.73 * 18.872 * 1.635 + 0.73 * 18.872 * 1.635 = 57.69 KN - m/m$$

$$M_{xs} = \frac{2}{3} * 57.69 = 38.46 KN - m/m$$

$$M_{xf} = \frac{1}{3} * 57.69 = 19.23 KN - m/m$$

Design Reinforcement

Design constants

$f_{cd} = 11.33 Mpa$

$f_{yd} = 260.87 Mpa$

D = 150 *mm*

d_1 = 130 *mm*, d_2=120*mm*

$$\rho_{\min} = \frac{0.5}{f_{yk}} = \frac{0.5}{300} = 0.017$$

$$A_{s\min} = \rho_{\min bd}$$

$$S_{\max} = \begin{cases} 2D = 300 \\ 350 \end{cases}$$

$$S_{\max} \le 300mm$$

= 0.0017 * 1000 * 130 = 217.1 mm^2

Strip	Design Field Moment	μ_s	K_z	A_s	Spacing Calculated	Spacing Provided
I - I	19.23	0.104	0.938	604.52	130	Use Φ10 c/c170 *mm*
G - G	7.05	0.037	0.968	214.76	366	Use Φ10 c/c 300*mm*
Strip	Design support Moment	μ_s	K_z	A_s	Spacing Calculated	Spacing Provided
I - I	38.46	0.201	0.882	1285.8	61.1	Use Φ10 c/c 60 *mm*
G - G	14.1	0.074	0.952	436.7	169.4	Use Φ10 c/c 165 *mm*
B -B	7.234	0.0378	0.966	220.82	355.6	Use Φ10 c/c 300 *mm*
F - F	4.05	0.021	0.976	122.4	641.67	Use Φ10 c/c 300 *mm*

For the rest field and support moments Use minimum reinforcement
i.e. Φ 10 c/c 300 *mm*

1.4 Typical slab Reinforcement

Design constants

$f_{cd} = 11.33 Mpa$

$f_{yd} = 260.87 Mpa$

D = 150 *mm*

d_x = 120 *mm* = d_s

d_y = 130 mm

$$\rho_{\min} = \frac{0.5}{f_{yk}} = \frac{0.5}{300} = 0.017$$

$$A_{s\min} = \rho_{\min bd}$$

= 0.0017 * 1000 * 130 = 221 mm^2

$$S_{\max} = \begin{cases} 2D = 300 \\ 350 \end{cases}$$

$$S_{\max} \le 300mm$$

Reinforcement Location	Design Moment (KN-m)	Depth (mm)	μ_s	k_z	As (mm^2)	Spacing Calculated (mm)	Spacing Provided (mm)
Support	11.6	130	0.06	0.96	356.30	220.43	*Φ10 c/c 215*
	9.844	130	0.051	0.962	301.73	260.29	*Φ10 c/c 240*
	13.103	130	0.068	0.954	404.994	194.00	*Φ10 c/c 190*
	15.974	130	0.0834	0.947	497.389	157.90	*Φ10 c/c 150*
	14.61	130	0.0762	0.949	453.958	173.00	*Φ10 c/c 170*
	9.64	130	0.05	0.963	295.17	266.07	*Φ10 c/c 240*
	15.8	130	0.0824	0.946	492.491	159.47	*Φ10 c/c 155*
	6.13	130	0.032	0.968	186.731	420.60	*Φ10 c/c 240*
	5.38	130	0.028	0.974	162.875	482.209	*Φ10 c/c 240*
	6.13	130	0.032	0.968	186.732	420.605	*Φ10 c/c 240*
	15.27	130	0.079	0.95	473.966	165.707	*Φ10 c/c 160*
	8.03	130	0.0419	0.968	244.609	321.08	*Φ10 c/c 240*
	8.03	130	0.0419	0.968	244.609	321.08	*Φ10 c/c 240*
	15.27	130	0.079	0.95	473.966	165.707	*Φ10 c/c 160*
	11.32	130	0.059	0.96	347.702	225.882	*Φ10 c/c 220*
	8.76	130	0.0457	0.955	270.418	290.374	*Φ10 c/c 240*
	11.32	130	0.0591	0.96	347.702	225.882	*Φ10 c/c 220*
	9.205	130	0.048	0.964	281.565	278.94	*Φ10 c/c 240*
	9.205	130	0.048	0.964	281.565	278.94	*Φ10 c/c 240*

Reinforcement Location	Design Moment (KN-m)	Depth (mm)	μ_s	k_z	As (mm^2)	Spacing Calculated (mm)	Spacing Provided (mm)
Short Span S_1	16.31	130	0.0837	0.947	507.73	154.69	*Φ10 c/c 150*
Long Span	12.35	120	0.0756	0.95	415.28	189.13	*Φ10 c/c 185*
Short Span S_3	9.19	130	0.0479	0.962	281.69	278.82	*Φ10 c/c 240*
Long Span	9.19	120	0.0563	0.96	305.80	256.83	*Φ10 c/c 240*
Short Span S_4	16.51	130	0.0862	0.948	513.59	152.92	*Φ10 c/c 150*
Long Span	12.37	120	0.0758	0.95	416.06	188.77	*Φ10 c/c 185*
Short Span S_5	8.043	130	0.0419	0.968	245.00	320.57	*Φ10 c/c 240*
Long Span	5.05	120	0.0309	0.97	166.18	472.63	*Φ10 c/c 240*
Short Span S_6	5.38	130	0.02808	0.972	163.21	481.22	*Φ10 c/c 240*
Long Span	4.00	120	0.0245	0.975	131.05	599.31	*Φ10 c/c 240*
Short Span S_7	5.38	130	0.02808	0.972	163.21	481.22	*Φ10 c/c 240*
Long Span	4.00	120	0.0245	0.975	131.05	599.31	*Φ10 c/c 240*
Short Span S_8	8.04	130	0.0419	0.968	245.00	320.57	*Φ10 c/c 240*
Long Span	5.05	120	0.0309	0.97	166.17	472.63	*Φ10 c/c 240*
Short Span S_9	15.87	130	0.0828	0.946	494.67	158.77	*Φ10 c/c 155*
Long Span	11.81	120	0.0723	0.952	396.22	198.22	*Φ10 c/c 195*
Short Span S_{10}	6.77	130	0.0353	0.97	205.80	381.63	*Φ10 c/c 240*
Long Span	6.85	120	0.0419	0.968	225.95	347.59	*Φ10 c/c 240*
Short Span S_{11}	6.77	130	0.0353	0.97	205.80	381.63	*Φ10 c/c 240*
Long Span	6.847	120	0.0419	0.968	225.95	347.59	*Φ10 c/c 240*
Short Span S_{12}	15.72	130	0.0821	0.945	490.66	160.67	*Φ10 c/c 155*
Long Span	11.65	120	0.0713	0.952	390.92	200.91	*Φ10 c/c 195*

1.5 Check for shear resistance

The Shear force V_c carried by concrete in members without significant axial force shall be taken as:-

$V_c = 0.25\, f_{ctd}\, K_1\, K_2$, where $K_1 = (1+50\rho) < 2.0$

$K_2 = 1.6-d > 1.0$

[EBCS-2,1995 Article 4.5.3.1]

$$\rho = \frac{A_s}{b_w d} = \frac{507.726*10^6}{1(0.13)} = 0.0039$$

$K_1 = (1+50\rho) = (1+50(0.0039)) = 1.195 < 2.0$

$K_1 = 1.195$

$K_2 = 1.6-d = 1.6-0.13 = 1.47 > 1.0$

$K_2 = 1.4$

V_c = 0.25(10^6) 1.195)(1.47)(1)(0.13)
= 57.09 *KN/m*

Therefore, Maximum Shear from above design is 36.76 KN/m << 57.09 KN/m Satisfies the requirement

1.6 Development length

The design bond strength of deformed bar may be considered as twice the tensile strength of the bar.

f_{bd} = 2 *f_{ctd} = 2 * 1.032 = 2.0364Mpa

The basic anchorage length

$$L_b = \frac{\phi}{4}\frac{f_{yd}}{f_{bd}}$$

For ϕ 10 *mm*

$$L_b = \frac{10}{4} * \frac{260.87}{2.06} = 315.976mm$$

For the required length

$$L_{bNet} = aL_b \frac{A_{scal}}{A_{seff}} \geq L_{b,\min} \quad 10\phi$$

L_{bmin} = 0.6L_b > 10ϕ or ≥ 200*mm*

For ϕ 10 L_{bmin} = 0.6*315.97
= 189.58 >100

The length of lap (L_o) shall be at least equal to

$l_o > a$,$L_{bnet} \geq L_{omin}$

The value of a from the table = 1
For ϕ10 L_o > 1 * 189.565 = 189.585 > (L_{omin} = 200)

∴ Finally use development length L_o = *200 mm*

Distance of zero moment from support is assumed to be (l/4)
For 4 meter length (4000/4) = 1000 mm total length = 1200 mm
For 5 meter length (5000/4) = 1250 mm total length = 1450 mm
For 3 meter length (3000/4) = 750 mm total length = 950 mm