Two way slab

slab must be continuously supported on all four edges

W: uniformly distributed load on slab

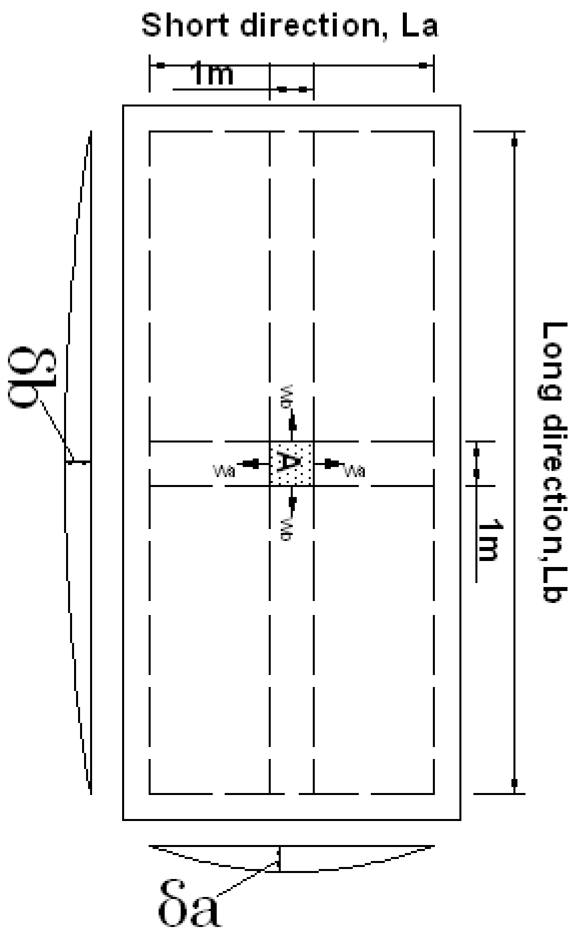
$$\delta = \frac{5W_a.\, l_a^4}{384E_cI_e} = \frac{5W_b.\, l_b^4}{384\; E_cI_e}$$

$$\frac{W_a}{W_b} = \left(\frac{l_b}{l_a}\right)^4$$

$$W = W_a + W_b \ldots\ldots\ldots\ldots (1)$$

let aspect ratio of slab

$$\frac{l_b}{l_a} = 2$$



$$\frac{W_a}{W_b} = (2)^4 = 16 \dots \dots \dots \dots (2)$$

solve eq 1) and eq 2) we get

$$W_a = \frac{16}{17} W = 0.94\ W$$

$$W_b = \frac{1}{17} W = 0.06\ W$$

$$\text{If} \frac{l_b}{l_a} \geq 2.0 \text{ Design as one way slab}$$

$$\text{If} \frac{l_b}{l_a} < 2.0 \text{ Design as two way slab}$$

Two way solid slab design

Method 3

coefficient method (ACI code 1963 elastic)

Design middle strip design for M_1 and M_2

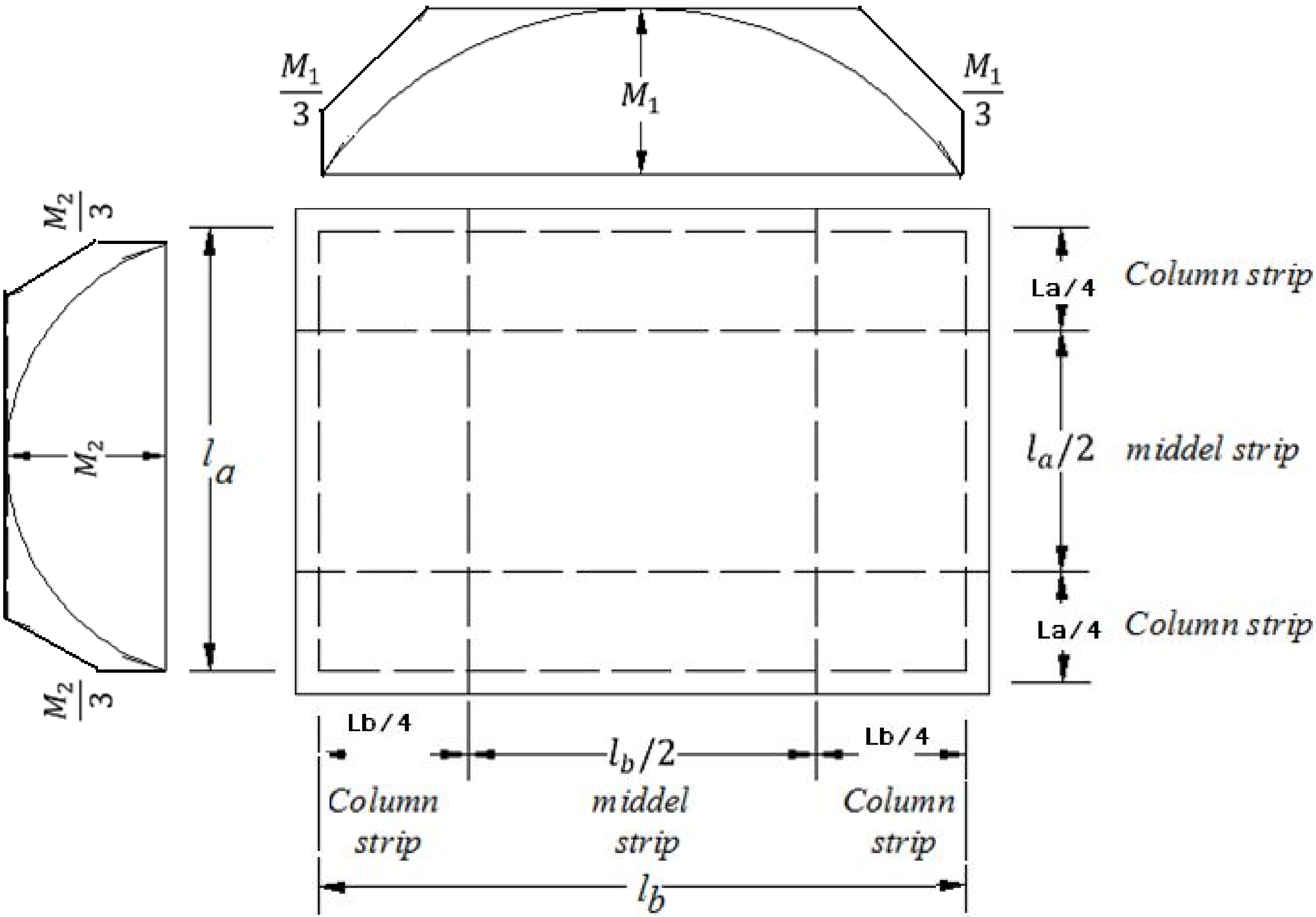
Design column strip design for $\frac{1}{2}\left(M_1 + \frac{M_1}{3}\right) = \frac{2}{3}M_1$

As we find (As) and (S) for middle strip→

$$As_{column\ strip} = \frac{2}{3}As_{middle\ strip}$$

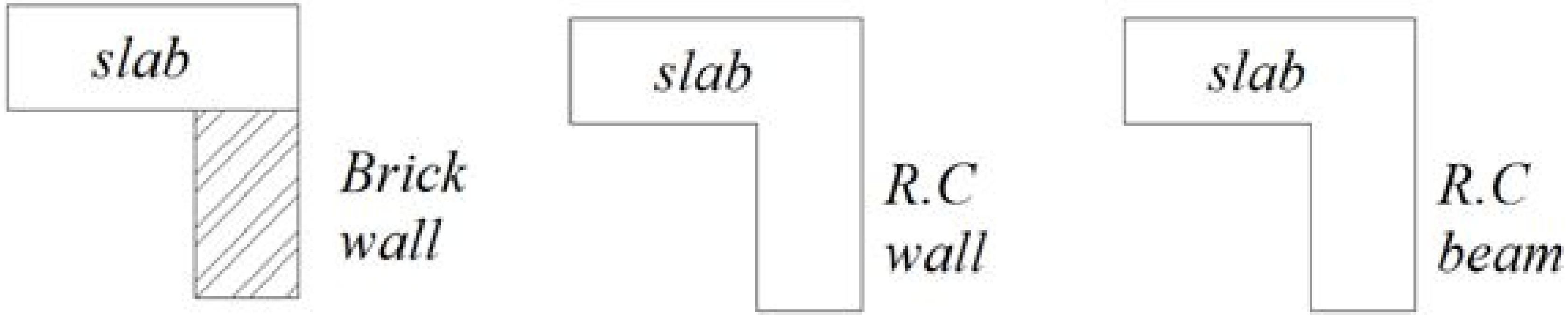
$$S_{column\ strip} = \frac{3}{2}S_{middle\ strip}$$

Aspect ratio l_a/l_b=(1.0↔0.5)



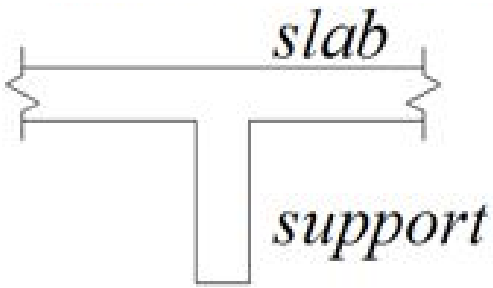
$\frac{M_1}{3}$
M_1
$\frac{M_1}{3}$
$\frac{M_2}{3}$
M_2
$\frac{M_2}{3}$
l_a
La/4
Column strip
$l_a/2$
middel strip
La/4
Column strip
Lb/4
$l_b/2$
Lb/4
Column strip
middel strip
Column strip
l_b

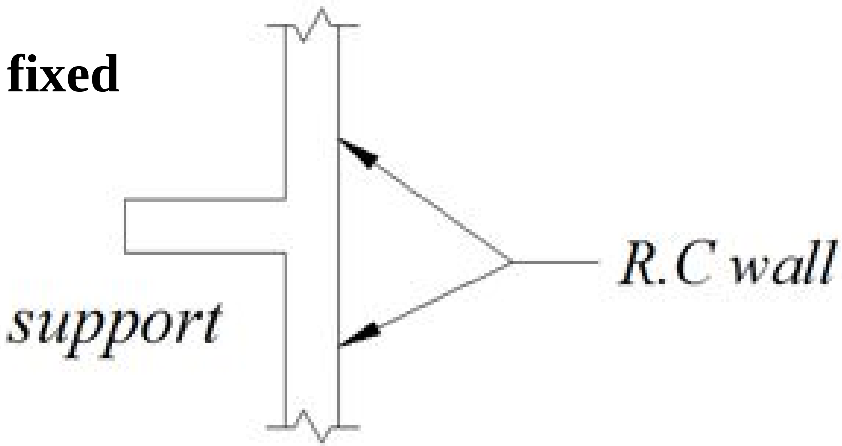
|(a)discontinuous slab or (b)supporting have small torsional resistance



(a) **(b)**

(c) slab is continuous a cross support (d) fixed





(c) **(d)**

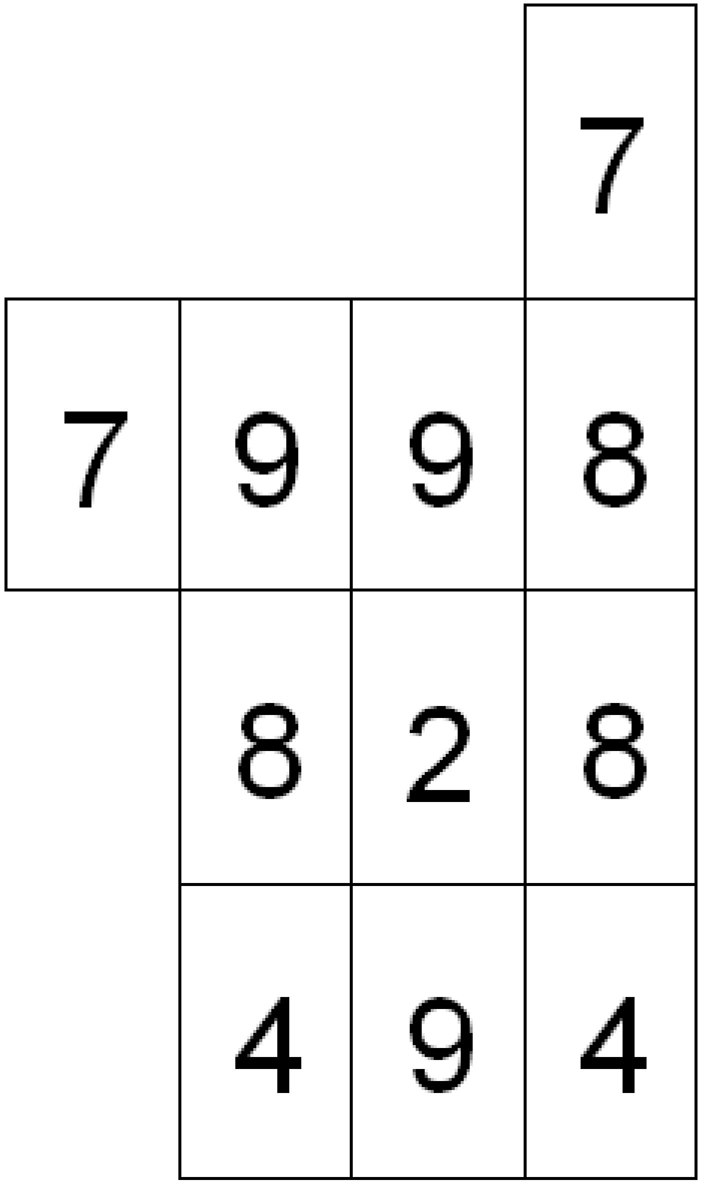

7
7
9
9
8
8
2
8
4
9
4

Table 8.3 coefficients for negative moment in slabs

$Ma- = (Ca-) * W * La^2$

$Mb- = (Cb-) * W * Lb^2$

Where W=total uniform live lead plus live load

Ratio m=la/lb		Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
1.00	Ca-		.045		.050	.075	.071		.033	.061
	Cb-		.045	.076	.050			.071	.061	.033
0.95	Ca-		.050		.055	.079	.075		.038	.065
	Cb-		.041	.072	.045			.067	.056	.029
0.90	Ca-		.055		.060	.080	.079		.043	.068
	Cb-		.037	.070	.040			.062	.052	.025
0.85	Ca-		.060		.066	.082	.083		.049	.072
	Cb-		.031	.065	.034			.057	.046	.021
0.80	Ca-		.065		.071	.083	.086		.055	.075
	Cb-		.027	.061	.029			.051	.041	.017
0.75	Ca-		.069		.076	.085	.088		.061	.078
	Cb-		.022	.056	.024			.044	.036	.014
0.70	Ca-		.074		.081	.086	.091		.068	.081
	Cb-		.017	.050	.019			.038	.029	.011
0.65	Ca-		.077		.085	.087	.093		.074	.083
	Cb-		.014	.043	.015			.031	.024	.008
0.60	Ca-		.081		.089	.088	.095		.080	.085
	Cb-		.010	.035	.011			.024	.018	.006
0.55	Ca-		.084		.092	.089	.096		.085	.086
	Cb-		.007	.028	.008			.019	.014	.005
0.50	Ca-		.086		.094	.090	:097		.089	.088
	Cb-		.006	.022	.006			.014	.010	.003

Table 8.4 coefficients for dead load positive moment in slabs

$$Ma+.dL=(Ca+.dL)*Wd*La^2$$

$$Ma+.dL=(Cb+.dL)*Wd*Lb^2$$

Where Wd =total uniform daed load

Ratio m=la/lb		Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
1.00	Ca.dl	.036	.018	.018	.027	.027	.033	.027	.020	.023
	Cb.dl	.036	.018	.027	.027	.018	.027	.033	.023	.020
0.95	Ca.dl	.040	.020	.021	.030	.028	.036	.031	.022	.024
	Cb.dl	.033	.016	.025	.024	.015	.024	.031	.021	.017
0.90	Ca.dl	.045	.022	.025	.033	.029	.039	.035	.025	.026
	Cb.dl	.029	.014	.024	.022	.013	.021	.028	.019	.015
0.85	Ca.dl	.050	.024	.029	.036	.031	.042	.040	.029	.028
	Cb.dl	.026	.012	.022	.019	.011	.017	.025	.017	.013
0.80	Ca.dl	.056	.026	.034	.039	.032	.045	.045	.032	.029
	Cb.dl	.023	.011	.020	.016	.009	.015	.022	.015	.010
0.75	Ca.dl	.061	.028	.040	.043	.033	.048	.051	.036	.031
	Cb.dl	.019	.009	.018	.013	.007	.012	.020	.013	.007
0.70	Ca.dl	.068	.030	.046	.046	.035	.051	.058	.040	.033
	Cb.dl	.016	.007	.016	.011	.005	.009	.017	.011	.006
0.65	Ca.dl	.074	.032	.054	.050	.036	.054	.065	.044	.034
	Cb.dl	.013	.006	.014	.009	.004	.007	.014	.009	.005
0.60	Ca.dl	.081	.034	.062	.053	.037	.056	.073	.048	.036
	Cb.dl	.010	.004	.011	.007	.003	.06	.012	.007	.004
0.55	Ca.dl	.088	.035	.071	.056	.038	.058	.081	.052	.037
	Cb.dl	.008	.003	.009	.005	.002	.004	.009	.005	.003
0.50	Ca.dl	.095	.037	.080	.059	.039	.061	.089	.056	.038
	Cb.dl	.006	.002	.007	.004	.001	.003	.007	.004	.002

Table 8.5 coefficients for live load positive moment in slabs

$Ma+.LL=(Ca+.LL)*WL*La2$

$Mb+.LL=(Cb+.LL)*WL*Lb^2$

Where WL =total uniform live load

Ratio m=la/lb		Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
1.00	Ca.LL	.036	.027	.027	.032	.032	.035	.032	.028	.030
	Cb.LL	.036	.027	.032	.032	.027	.032	.035	.030	.028
0.95	Ca	.040	.030	.031	.035	.034	.038	.036	.031	.032
	Cb	.033	.025	.029	.029	.024	.029	.032	.027	.025
0.90	Ca	.045	.034	.035	.039	.037	.042	.040	.035	.036
	Cb	.029	.022	.027	.026	.021	.025	.029	.024	.022
0.85	Ca	.050	.037	.040	.043	.041	.046	.045	.040	.039
	Cb	.026	.019	.024	.023	.019	.022	.026	.022	.020
0.80	Ca	.056	.041	.045	.048	.044	.051	.051	.044	.042
	Cb	.023	.017	.022	.020	.016	.019	.023	.019	.017
0.75	Ca	.061	.045	.051	.052	.047	.055	.056	.049	.046
	Cb	.019	.014	.019	.016	.013	.016	.020	.016	.013
0.70	Ca	.068	.049	.057	.057	.051	.060	.063	.054	.050
	Cb	.016	.012	.016	.014	.011	.013	.017	.014	.011
0.65	Ca	.074	.053	.064	.062	.055	.064	.070	.059	.054
	Cb	.013	.010	.014	.011	.009	.010	.014	.011	.009
0.60	Ca	.081	.058	.071	.067	.059	.068	.077	.065	.059
	Cb	.010	.007	.011	.009	.007	.008	.011	.009	.007
0.55	Ca	.088	.062	.080	.072	.063	.073	.085	.070	.063
	Cb	.008	.006	.009	.007	.005	.006	.009	.007	.006
0.50	Ca	.095	.066	.088	.077	.067	.078	.092	.076	.067
	Cb	.006	.004	.007	.005	.004	.005	.007	.005	.004

Table 8.6 ratio of load W in la , lb directions for shear in slab supports

Ratio m=la/lb		Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
1.00	Wa	.50	.50	.17	.50	.83	.71	.29	.33	.67
	Wb	.50	.50	.83	.50	.17	.29	.71	.67	.33
0.95	Wa	.55	.55	.20	.55	.86	.75	.33	.38	.71
	Wb	.45	.45	.80	.45	.14	.25	.67	.62	.29
0.90	Wa	.60	.60	.23	.60	.88	.79	.38	.43	.75
	Wb	.40	.40	.77	.40	.12	.21	.62	.57	.25
0.85	Wa	.66	.66	.28	.66	.90	.83	.43	.49	.79
	Wb	.34	.34	.72	.34	.10	.17	.57	.51	.21
0.80	Wa	.71	.71	.33	.71	.92	.86	.49	.55	.83
	Wb	.29	.29	.67	.29	.08	.14	.51	.45	.17
0.75	Wa	.76	.76	.39	.76	.94	.88	.56	.61	.86
	Wb	.24	.24	.61	.24	.06	.12	.44	.39	.14
0.70	Wa	.81	.81	.45	.81	.95	.91	.62	.68	.89
	Wb	.19	.19	.55	.19	.05	.09	.38	.32	.11
0.65	Wa	.85	.85	.53	.85	.96	.93	.69	.74	.92
	Wb	.15	.15	.47	.15	.04	.07	.31	.26	.08
0.60	Wa	.89	.89	.61	.89	.97	.95	.76	.80	.94
	Wb	.11	.11	.39	.11	.03	.05	.24	.20	.06
0.55	Wa	.92	.92	.69	.92	.98	.96	.81	.85	.95
	Wb	.08	.08	.31	.08	.02	.04	.19	.15	.05
0.50	Wa	.94	.94	.76	.94	.99	.97	.86	.89	.97
	Wb	.06	.06	.24	.06	.01	.03	.14	.11	.03

Table 8.3 Negative moment (total load)

$$Ma = C_a^- W_u l_a^2$$

$$M_b = C_b^- W_u l_b^2$$

$$W_u = 1.2 * W_d + 1.6\, W_l$$

$$d_s = t - 25mm$$

$$d_l = d_S - 10\ mm$$

Table 8.4 positive moment (for dead load)

$$M_a^+ = C_{a_{DL}}^+ W_{uDL} l_a^2$$

$$M_b^+ = C_{b_{DL}}^+ W_{uDL} l_b^2$$

Table 8.5 positive moment (for live load)

$$M_a^+ = C_{a_{LL}}^+ W_{uLL} l_a^2$$

$$M_b^+ = C_{b_{LL}}^+ W_{uLL} l_b^2$$

Total positive moment

$$\text{Total } M_a^+ = C_{a_{DL}}^+ W_{uDL} l_a^2 + C_{a_{LL}}^+ W_{uLL} l_a^2$$

$$\text{Total } M_b^+ = C_{b_{DL}}^+ W_{uDL} l_b^2 + C_{b_{LL}}^+ W_{uLL} l_b^2$$

$$\text{Total } M_a^+ = (C_{a_{DL}}^+ W_{uDL} + C_{a_{LL}}^+ W_{uLL}) l_a^2$$

$$\text{Total } M_b^+ = (C_{b_{DL}}^+ W_{uDL} + C_{b_{LL}}^+ W_{uLL}) l_b^2$$

Table 8.6 (Ratio of load in l_a , l_b direction)

To find

$W_a \rightarrow$ to check shear

W_b and load transferred to beam.

Example :fc' =30 MPa, fy=400MPa,$W_{l.l}$=5 kN/m^2, all supports are R.C beams .Design floor slab.

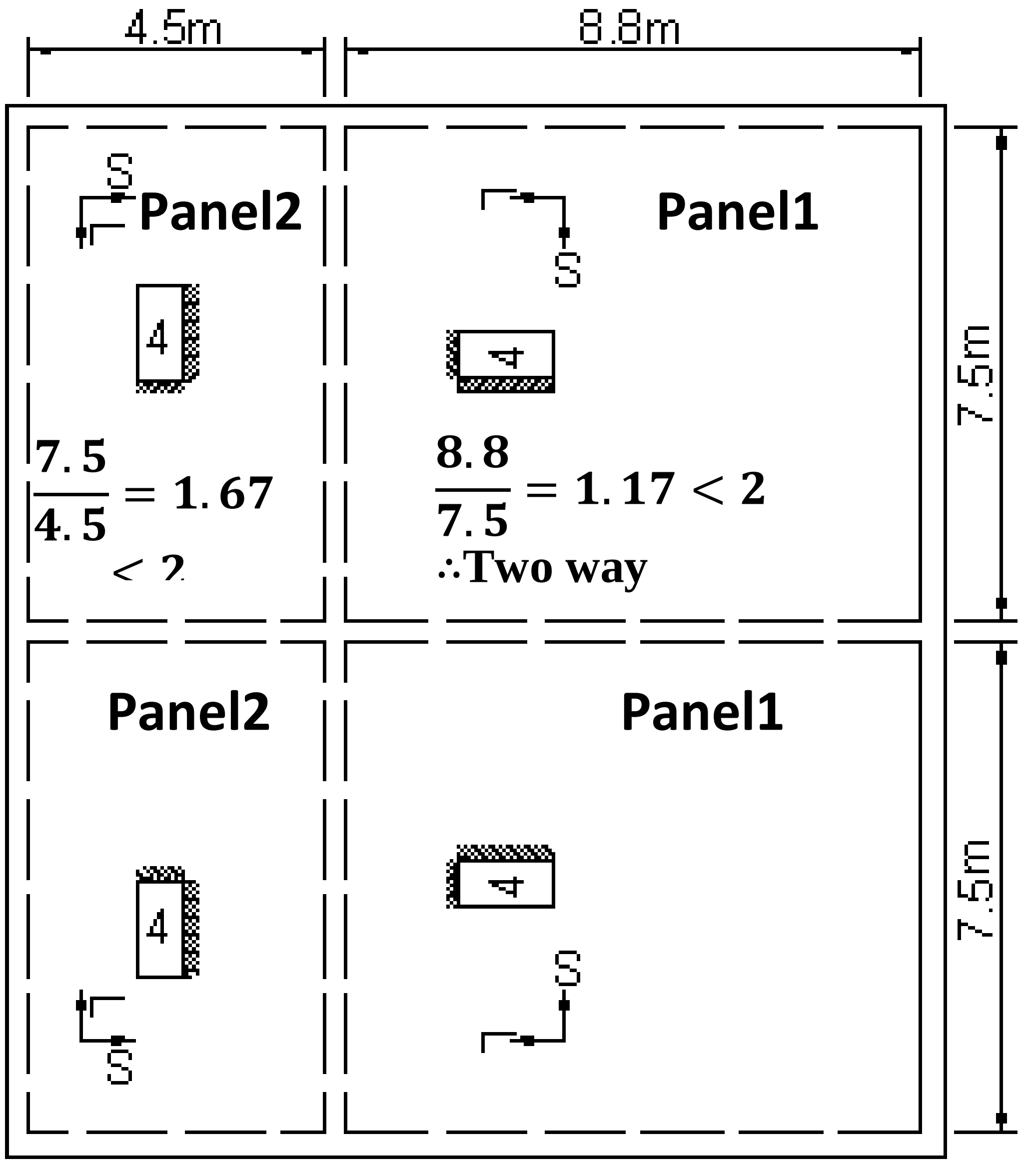
Solution

$$t = \frac{Perimeter}{180}$$

$$= \frac{(8.8 + 7.5) * 2}{180}$$

$$= 0.181 \rightarrow use\ t$$

$$= 190\ mm$$



Dead load

$$\text{Slab weight} = 0.19 * 24.5 = 4.66\,\frac{\text{kN}}{\text{m}^2}$$

$$\text{tiling with moetar} = 0.04 * 24.5 = 0.98\,\frac{\text{kN}}{\text{m}^2}$$

$$\text{plastring} = 0.48\,\frac{\text{kN}}{\text{m}^2}$$

$$\sum \text{D.L} = 6.12\,\frac{\text{kN}}{\text{m}^2}$$

$$W_u = 1.2 * W_d + 1.6\,W_l = 1.2 * 6.12 + 1.6 * 5 = 15.34\,\frac{\text{kN}}{\text{m}^2}$$

$$d_s = t - 25 = 165\text{mm}$$

$$d_l = d_s - 10 = 155\text{mm}$$

panel	direction	C^-	C_{+l}	C_{+D}	M-	M +	R-	R+	ω -	ω +	$\bar{\rho} = \omega * \frac{fc'}{fy}$	ρ^+	As_-	As_+
Case 4 m= $\frac{4.5}{7.5}$ = 0.6	Short L=4.5 m d_s=165 mm	0.089	0.067	0.053	27.65⟶36	18.75	0.0489	0.0254	0.0505	0.026	0.0038	0.00195	625	322⟶342
	Long L=7.5 m d_s=155 mm	0.011	0.009	0.007	9.49	6.94	0.01463	0.0107	0.0145	0.011	0.00109	0.00082	169⟶342	128⟶342
Case 4 m= $\frac{7.5}{8.8}$ = 0.85	Short L=7.5 m ds=165 mm	0.066	0.043	0.036	56.95	34.22	0.0775	0.0465	0.0810	0.048	0.00607	0.0036	1002	594
	Long L=8.8 m d_s=155 mm	0.034	0.023	0.019	40.4⟶36	25	0.0555	0.0385	0.0575	0.0395	0.0043	0.00296	668	459
									$< _{max}$	$< _{max}$				

$$M^{-}{}_{short} = C_{-}W_u l_n{}^2 = 0.089 * 15.34 * (4.5)^2 = 27.65\ kN.m$$

$$M^{-}{}_{long} = 0.011 * 15.34 * (7.5)^2 = 9.49\ kN.m$$

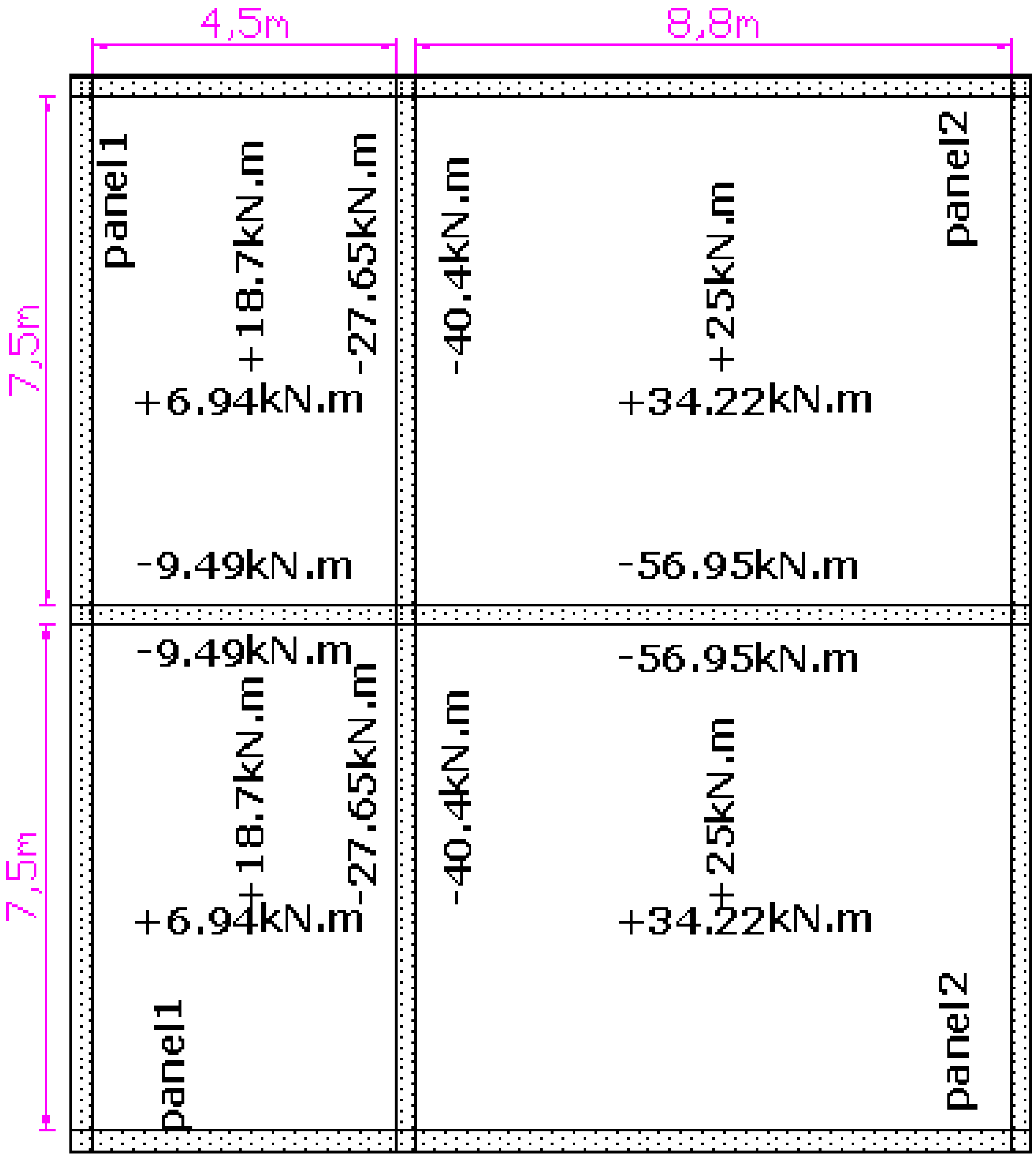
$$M^{+} = (C_{+D} 1.2 W_d + C_{+L} * 1.6 * W_l) l_n{}^2$$

$$M^{+}{}_{short} = (0.053 * 1.2 * 6.12 + 0.067 * 1.6 * 5) * (4.5)^2$$

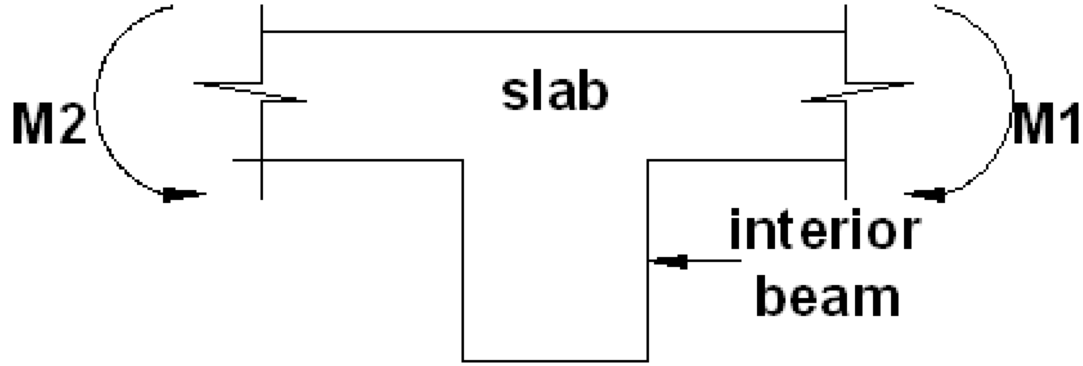
$$= 18.75\ kN.m$$

$$M^{+}{}_{long} = (0.007 * 1.2 * 6.12 + 0.009 * 1.6 * 5) * (7.5)^2$$

$$= 6.94\ kN.m$$


4,5m
8,8m
7,5m
7,5m
panel1
+18.7kN.m
-27.65kN.m
-40.4kN.m
+25kN.m
panel2
+6.94kN.m
+34.22kN.m
-9.49kN.m
-56.95kN.m
-9.49kN.m
-56.95kN.m
+18.7kN.m
-27.65kN.m
-40.4kN.m
+25kN.m
+6.94kN.m
+34.22kN.m
panel1
panel2

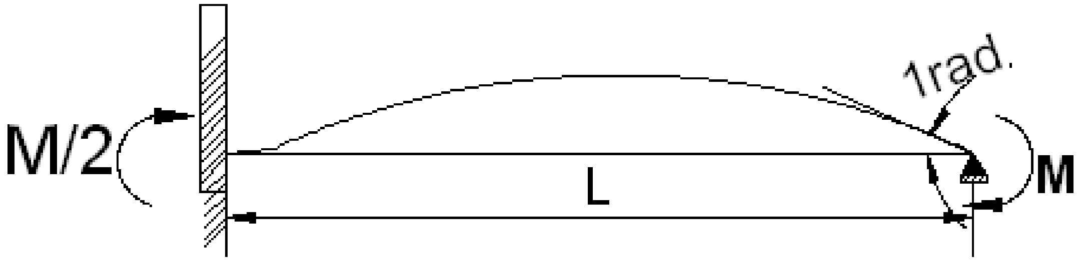
$M1 > M2$



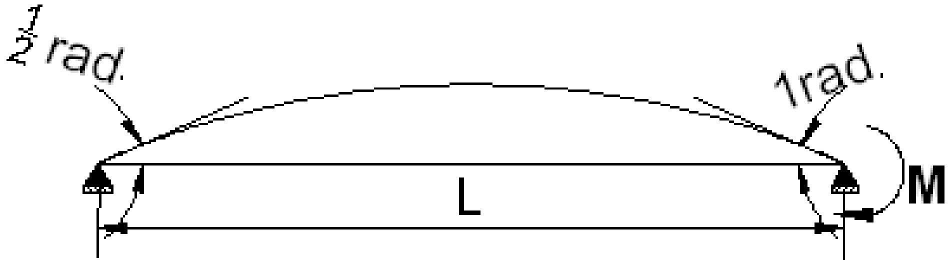
$$if\ \frac{M2}{M1} \geq 0.8 \rightarrow design\ for\ max.moment$$

$if\ \frac{M2}{M1} < 0.8$ →redistribute the difference between M1&M2 in accordance with the relative flexural stiffness of the panels.

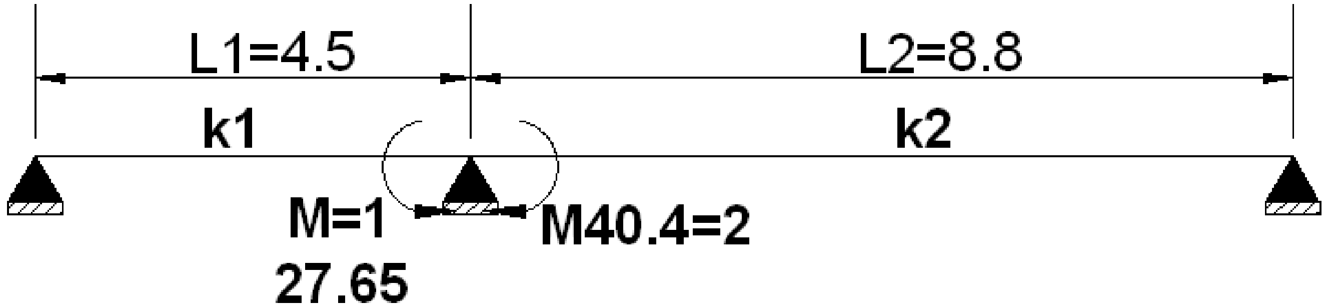
flexural stiffness



M=k(flexural stiffness)=(4EI)/L



M=k(flexural stiffness)=(3EI)/L



$$\frac{M1}{M2} = \frac{27.65}{40.4} = 0.65 < 0.8 \rightarrow$$

$$\therefore redistribute\ the - ve\ moments$$

$$\overline{k1} = relative\ stiffness\ of\ panel\ 1 = \frac{k1}{k1 + k2}$$

$$= \frac{\frac{3EI}{L1}}{\frac{3EI}{L1} + \frac{3EI}{L2}} = \frac{L2}{L1 + L2} = \frac{8.8}{4.5 + 8.8} = 0.66$$

$$\overline{k2} = relative\ stiffness\ of\ panel\ 2 = \frac{k2}{k1 + k2} = 0.34$$

$$M = M1 + \overline{k1}\,\Delta M = 27.65 + 0.66(40.4 - 27.65) = 36kN.m$$

$$OR\ M = M2 - \overline{k2}\,\Delta M = 40.4 + 0.34(40.4 - 27.65)$$

$$= 36kN.m$$

Let Ø=0.9 to be check later

$$R = \frac{Mu}{\emptyset fc' bd^2} = \frac{36}{0.9 * 30000 * 1 * 0.165^2} = 0.0489$$

$$\omega_{max} = 0.364\beta 1 = 0.364 * 0.83 = 0.302 \rightarrow all\ \omega < \omega_{max}\ O.K$$

$$\rho = \omega \frac{fy}{fc'}$$

$$\rho_t = 0.85\beta 1 \frac{fc'}{fy}\frac{\varepsilon_{cu}}{\varepsilon_{cu} + 0.005}$$

$$= 0.85 * .83 * \frac{30}{400} * \frac{0.003}{0.003 + 0.005} = 0.198 \rightarrow all\ \rho < \rho_t$$

$$\rightarrow \therefore \emptyset = 0.9\ O.K$$

$$for\ fy = 400MPa, As_{min} = 0.0018bt$$

$$= 0.0018 * 1000 * 190 = 342mm^2/m_{width}$$

$$S_{max}$$

$$= \min(3t = 3 * 190$$

$$= 570mm, 450mm)\ code\ 1963\ method3$$

S_{max}

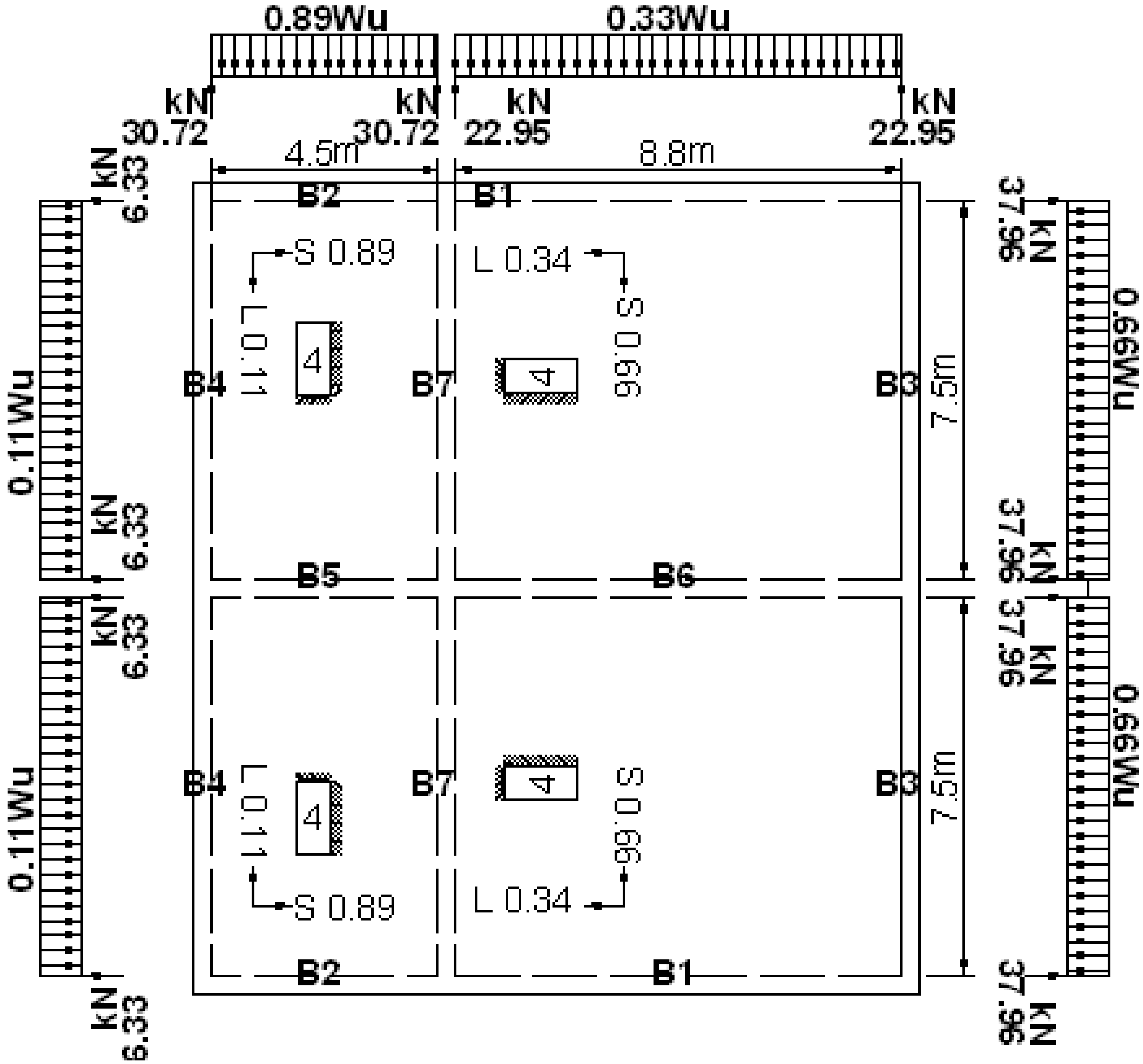
$= \min(2t = 2 * 190$

$= 380mm, 500mm)$ $code\ 2002\ direct\ design\ method,\ \ ACI$ 13.3.2

$\therefore S_{max} = 400mm$

$$As_{col.strip} = \frac{2}{3} As_{middle\ strip} \rightarrow S_{col.strip} = \frac{3}{2} S_{middle\ strip}$$

Check shear capacity



Max. reaction(Vu)=37.96kN

Vud=37.96-0.66*15.34*0.165=36.29kN

$$Vc = 0.17\sqrt{30} * 1.0 * 0.165 = 0.153MN$$

$$\frac{Vud}{\emptyset} = \frac{36.29}{0.75} = 48.38kN < Vc\ OK$$

Load transferred to beams

- Elastic stage:

$$B1\begin{cases} Wd = 0.66 * 6.12 * \dfrac{7.5}{2} = 15.15\dfrac{kN}{m} \rightarrow 17.42\dfrac{kN}{m} \\ Wl = 0.66 * 5 * \dfrac{7.5}{2} = 12.37\dfrac{kN}{m} \rightarrow 14.23\dfrac{kN}{m} \end{cases}$$

$$B2\begin{cases} Wd = 0.11 * 6.12 * \dfrac{7.5}{2} = 2.52\dfrac{kN}{m} \rightarrow 9.18\dfrac{kN}{m} \\ Wl = 0.11 * 5 * \dfrac{7.5}{2} = 2.06\dfrac{kN}{m} \rightarrow 7.5\dfrac{kN}{m} \end{cases}$$

$$B3\begin{cases} Wd = 0.34 * 6.12 * \dfrac{8.8}{2} = 9.15\dfrac{kN}{m} \rightarrow 15.3\dfrac{kN}{m} \\ Wl = 0.34 * 5 * \dfrac{8.8}{2} = 7.48\dfrac{kN}{m} \rightarrow 12.5\dfrac{kN}{m} \end{cases}$$

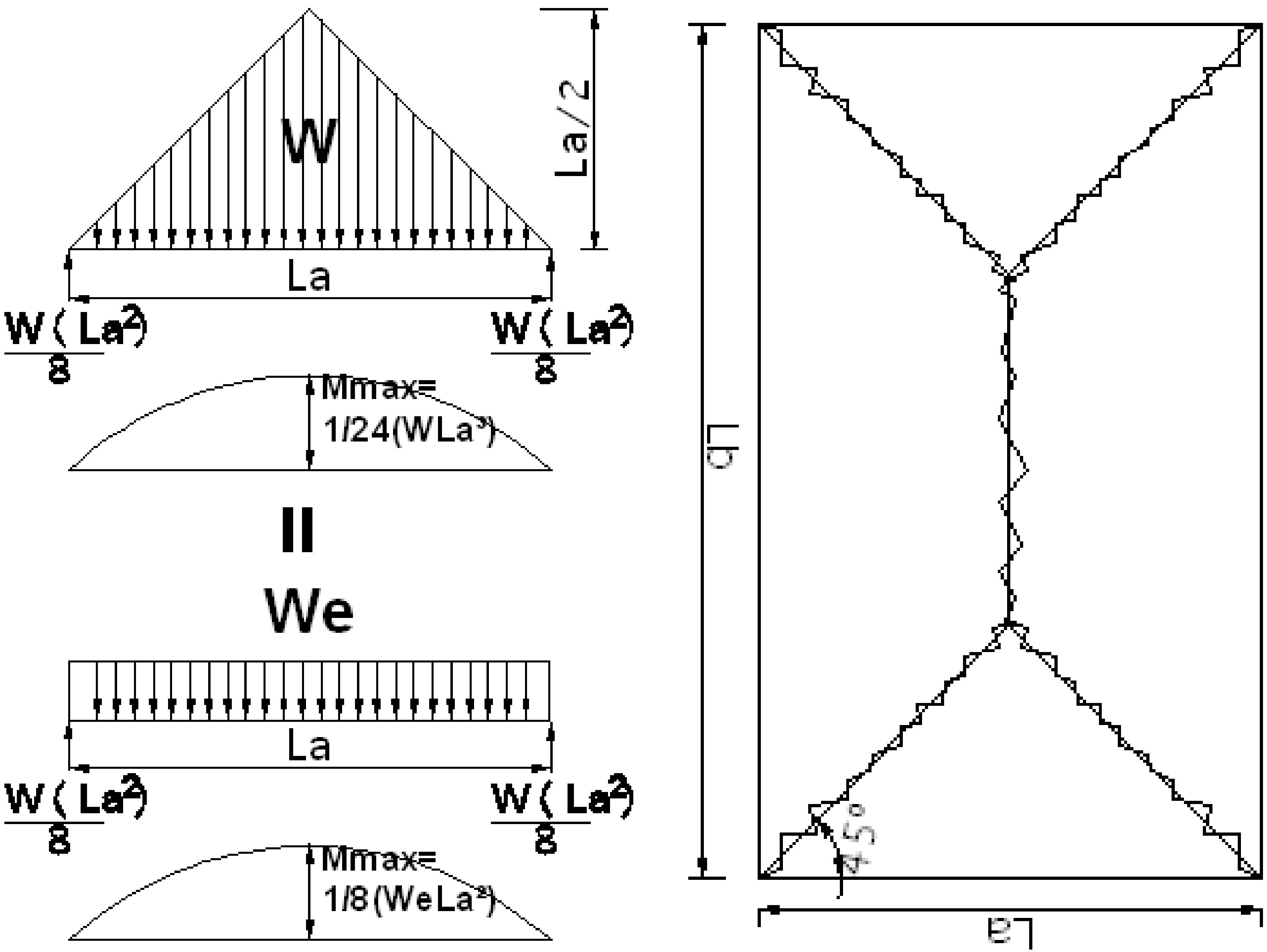
$$B4\begin{cases} Wd = 0.89 * 6.12 * \dfrac{4.5}{2} = 12.25\dfrac{kN}{m} \\ Wl = 0.89 * 5 * \dfrac{4.5}{2} = 10.01\dfrac{kN}{m} \end{cases}$$

$$B5\begin{cases} Wd = \left(0.11 * 6.12 * \dfrac{7.5}{2}\right) * 2 = 5.49\dfrac{kN}{m} \rightarrow 18.36\dfrac{kN}{m} \\ Wl = (0.11 * 5 * \dfrac{7.5}{2} = 4.12\dfrac{kN}{m} \rightarrow 15\dfrac{kN}{m} \end{cases}$$

$$B6\begin{cases} Wd = \left(0.66 * 6.12 * \frac{7.5}{2}\right) * 2 = 30.29\,\frac{kN}{m} \rightarrow 34.85\,\frac{kN}{m} \\ Wl = (0.66 * 5 * \frac{7.5}{2} = 24.75\ kN/m \rightarrow 28.46\,\frac{kN}{m} \end{cases}$$

$$B7\begin{cases} Wd = 0.34 * 6.12 * \frac{8.8}{2} + 0.89 * 6.12 * \frac{4.5}{2} = 21.41\,\frac{kN}{m} \rightarrow 27.42\,\frac{kN}{m} \\ Wl = 0.34 * 5 * \frac{8.8}{2} + 0.89 * 5 * \frac{4.5}{2} = 17.5\,\frac{kN}{m} \rightarrow 22.4\,\frac{kN}{m} \end{cases}$$

• Loads transferred to beams at failure stage of slab



$$\left(\frac{W_e la^2}{8}\right) = \left(\frac{W la^3}{24}\right) \rightarrow W_e = \frac{W la}{3} \quad short\ panel$$

$$W_e = \frac{W la}{3}\left(\frac{3 - m^2}{2}\right), \quad m = \frac{la}{lb} \quad long\ panel$$

$$B1 = \begin{cases} W_{ed} = \dfrac{6.12 * 7.5}{3}\left(\dfrac{3 - 0.85^2}{2}\right) = 17.42\,\dfrac{kN}{m} \\ W_{el} = \dfrac{5 * 7.5}{3}\left(\dfrac{3 - 0.85^2}{2}\right) = 14.23\,\dfrac{kN}{m} \end{cases}$$

$$B2 = \begin{cases} W_{ed} = \dfrac{6.12 * 4.5}{3} = 9.18 \dfrac{kN}{m} \\ W_{el} = \dfrac{5 * 4.5}{3} = 7.5 \dfrac{kN}{m} \end{cases}$$

$$B3 = \begin{cases} W_{ed} = \dfrac{6.12 * 7.5}{3} = 15.3 \dfrac{kN}{m} \\ W_{el} = \dfrac{5 * 7.5}{3} = 12.5 \dfrac{kN}{m} \end{cases}$$

$$B4 = \begin{cases} W_{ed} = \dfrac{6.12 * 4.5}{3} \left(\dfrac{3 - 0.6^2}{2} \right) = 12.12 \dfrac{kN}{m} \\ W_{el} = \dfrac{5 * 4.5}{3} \left(\dfrac{3 - 0.6^2}{2} \right) = 9.9 \dfrac{kN}{m} \end{cases}$$

$$B5 = \begin{cases} W_{ed} = \left(\frac{6.12 * 4.5}{3}\right) * 2 = 18.36 \frac{kN}{m} \\ W_{el} = \left(\frac{5 * 4.5}{3}\right) * 2 = 15 \frac{kN}{m} \end{cases}$$

$$B6 = \begin{cases} W_{ed} = \left[\frac{6.12 * 7.5}{3}\left(\frac{3 - 0.85^2}{2}\right)\right] * 2 = 34.85 \frac{kN}{m} \\ W_{el} = \left[\frac{5 * 7.5}{3}\left(\frac{3 - 0.85^2}{2}\right)\right] * 2 = 28.46 \frac{kN}{m} \end{cases}$$

$$B7 = \begin{cases} W_{ed} = \frac{6.12 * 7.5}{3} + \frac{6.12 * 4.5}{3}\left(\frac{3 - 0.6^2}{2}\right) = 27.42\,\frac{kN}{m} \\ W_{el} = \frac{5 * 7.5}{3} + \frac{5 * 4.5}{3}\left(\frac{3 - 0.6^2}{2}\right) = 22.4\,\frac{kN}{m} \end{cases}$$

Bent and cut off points in two way slab(approximate values):

Bent: Ln/6 from face of support

Cut off: Ln/4 from face of support

Panel 1 Long dir. Middle Strip	As^{+}=342mm^2, useØ10@225mm((2b,1s) <$S_{max.}$)=349mm^2> As^{+}=342mm^2 At discontinuous edge: Ø10@225(2b) →2/3*349=232mm^2< As_{min} add(Ø8@450=111mm^2) S_{av}=(225+450)/2=337mm<S_{max} OK
	As^{-}_{req}=342mm^2<As^{-}_{prov}= (2/3*349)*2=465mm^2 OK

Panel 1 Long dir. Column Strip	Since As_{min} control for middle strip, therefore, the reinforcement distribution will be fixed for column strip

Panel 2 Short dir. Middle Strip	$As^{+}=594mm^{2}$, useØ12@175mm((2b,1s) $<S_{max.})=646mm^{2}> As^{+}=594mm^{2}$ At discontinuous edge: Ø12@175(2b) $\rightarrow 2/3*646=430mm^{2}>As_{min}$ OK Spacing (175,350mm)$<S_{max}$ OK
	$As^{-}_{req}=1002mm^{2}$ $As^{-}_{prov}=(2/3*646)*2=861mm^{2}< As^{-}=1002mm^{2}$ $As^{-}_{add}=1002-861=141mm^{2}$ (use Ø8@350=$143mm^{2}>141mm^{2}$)

Panel 2 Short dir. Column Strip	As^{+}_{req}=2/3*594=396mm^2> As_{min}, useØ12@3/2*175=260mm((2b,1s) <$S_{max.}$)=435mm^2> As^{+}=396mm^2 OK At discontinuous edge: Ø12@260(2b) →2/3*435=290mm^2<As_{min} As_{add}=342-290=52mm^2 (use Ø8@780=64mm^2>52mm^2) S_{av} =(260+520mm)/2=390mm<S_{max} OK
	As^{-}_{req}=2/3*1002=668mm^2 As^{-}_{prov}= (2/3*435)*2=580mm^2< As^{-}=668mm^2

	As_{add}=668-580=88mm^2 (use Ø8@520=96mm^2>88mm^2)

Panel 1 Short dir. Middle Strip	As^{+}_{req}=342mm^2, useØ10@225mm((2b,1s)<$S_{max.}$)=349mm^2> As^{+}=342mm^2 At discontinuous edge: Ø10@225(2b) →2/3*349=232mm^2< As_{min} As_{add}=342-232=110mm^2 (Ø8@450=111mm^2>110) S_{av}=(225+450)/2=337mm<S_{max} OK

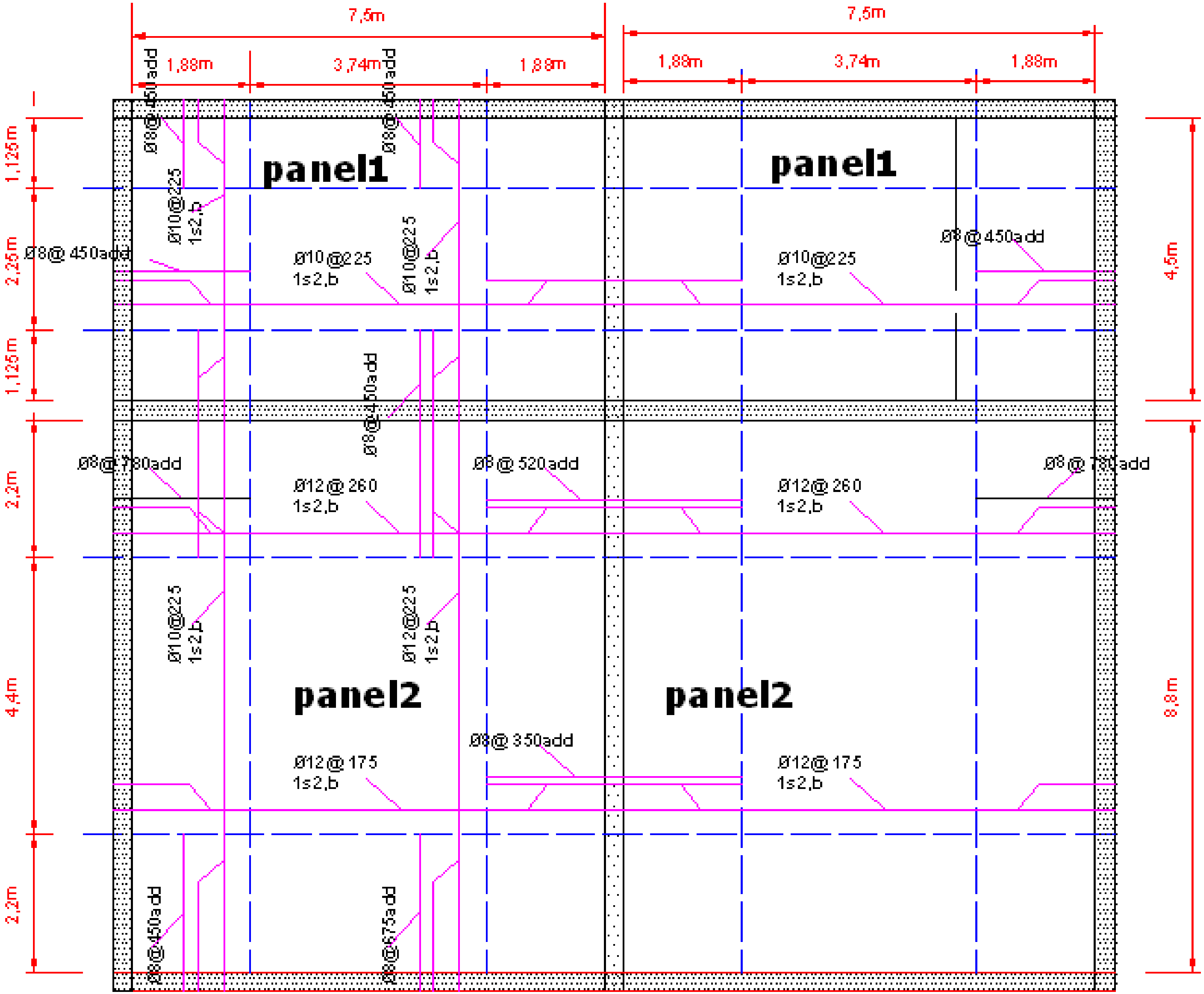
	As^{-}_{req}=625mm^2 As^{-}_{prov}= (2/3*349)=232mm^2

Panel 2 Long dir. Middle Strip	As^{+}=459mm^2, useØ12@225mm((2b,1s) <$S_{max.}$)=503mm^2> As^{+}=459mm^2 At discontinuous edge: Ø12@225(2b) →2/3*503=335mm^2<As_{min} As_{add}=342-335=7mm^2 (use Ø8@675=74mm^2>7mm^2) S_{av}=(225+450)/2=337mm<S_{max} OK
	As^{-}_{req}=668mm^2 As^{-}_{prov}= (2/3*503)=335mm^2

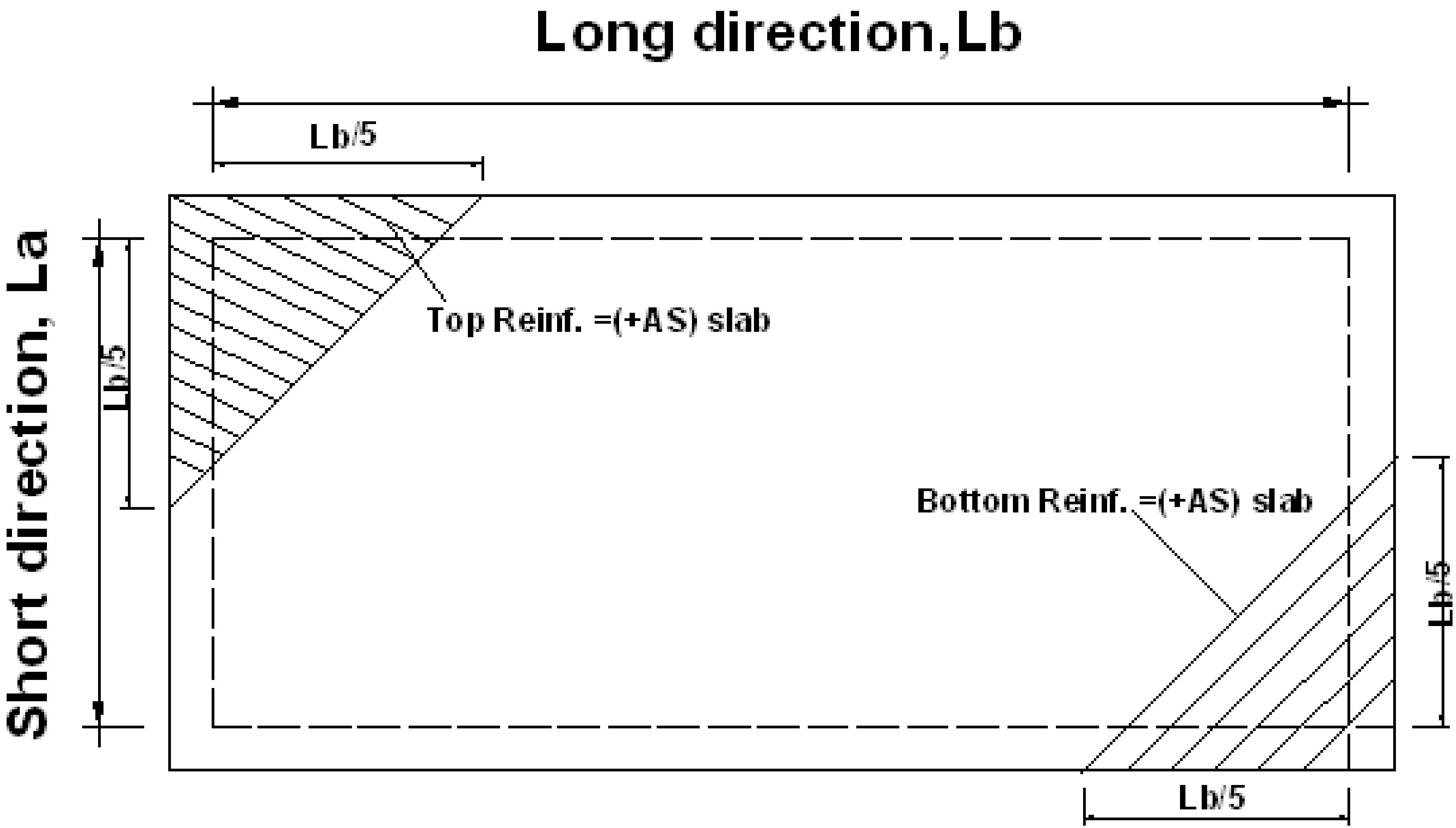
(Panel 2 Long dir.+ Panel 1Short dir.) Middle Strip	As^{-}_{req} =668mm^2, As^{-}_{prov}=335+232=567mm^2 $<$ 668mm^2 useØ8@450mm=111mm^2 add $>$ 668-567=101mm^2

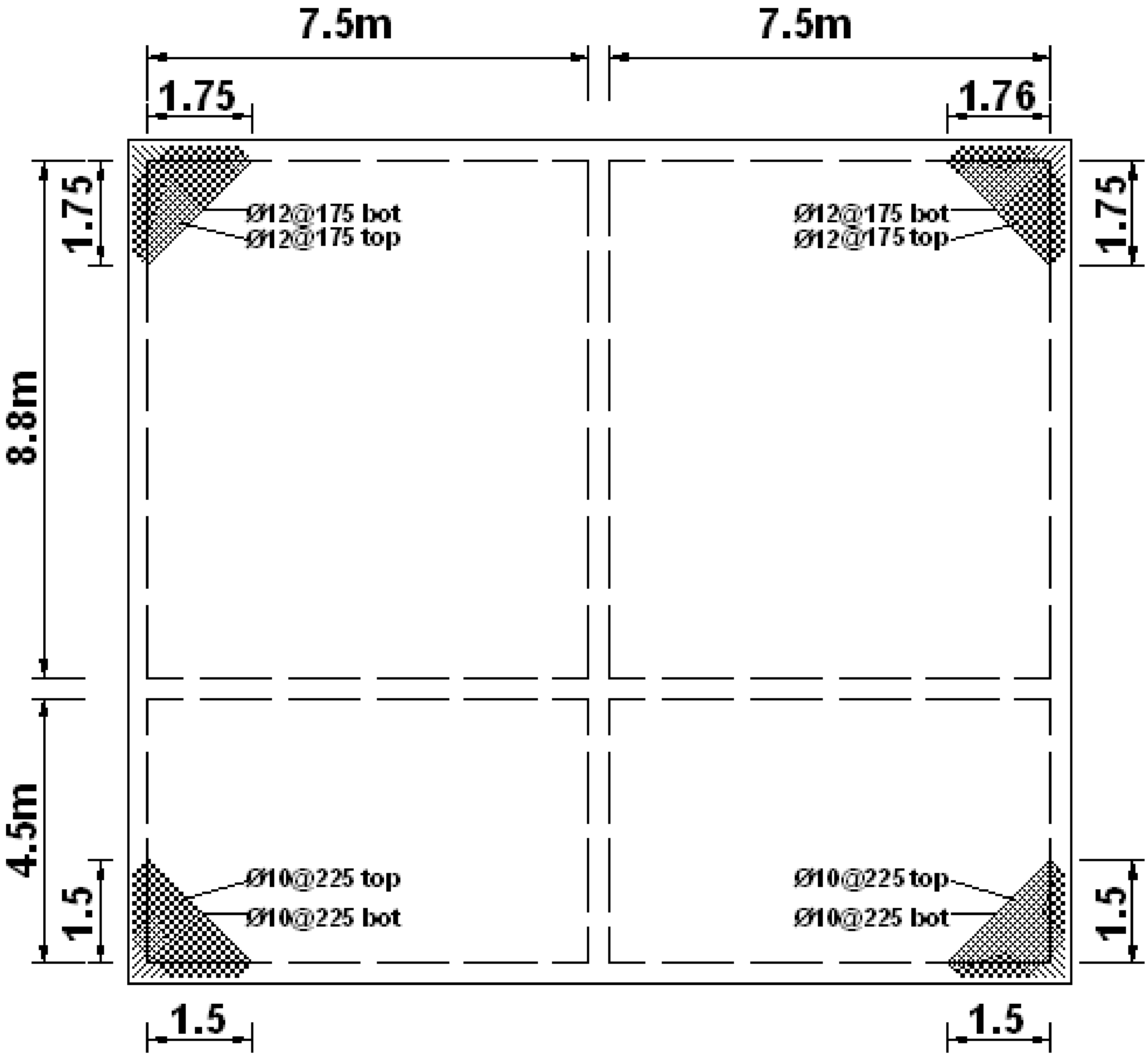
Panel 1 Short dir. Column Strip	Since As_{min} control for middle strip, therefore, the reinforcement distribution will be fixed for column strip

Panel 2 Long dir. Column Strip	As^{+}_{req}=2/3*459=306mm^2<As_{min}, useØ10@225=349mm^2((2b,1s) > As_{min} At discontinuous edge: Ø10@225(2b) →2/3*349=232mm^2<As_{min} As_{add}=342-232=110mm^2 (use Ø8@450=111mm^2>110mm^2) S_{av} =(225+450mm)/2=337mm<S_{max} OK
	As^{-}_{req}=2/3*668=445mm^2< As^{-}_{prov}= 2/3(349+349)=465mm^2


7,5m
7,5m
1,88m
3,74m
1,88m
1,88m
3,74m
1,88m
1,125m
2,25m
1,125m
2,2m
4,4m
2,2m
4,5m
8,8m
panel1
panel1
panel2
panel2
Ø8@450add
Ø8@450add
Ø8@450add
Ø8@450add
Ø10@225
1s2,b
Ø10@225
1s2,b
Ø10@225
1s2,b
Ø10@225
1s2,b
Ø8@450add
Ø8@450add
Ø8@780add
Ø8@520add
Ø8@780add
Ø12@260
1s2,b
Ø12@260
1s2,b
Ø10@225
1s2,b
Ø12@225
1s2,b
Ø8@350add
Ø12@175
1s2,b
Ø12@175
1s2,b
Ø8@450add
Ø8@675add

Corner reinforcement:




7.5m
7.5m
1.75
1.76
1.75
1.75
8.8m
Ø12@175 bot
Ø12@175 top
Ø12@175 bot
Ø12@175 top
4.5m
1.5
1.5
Ø10@225 top
Ø10@225 bot
Ø10@225 top
Ø10@225 bot
1.5
1.5