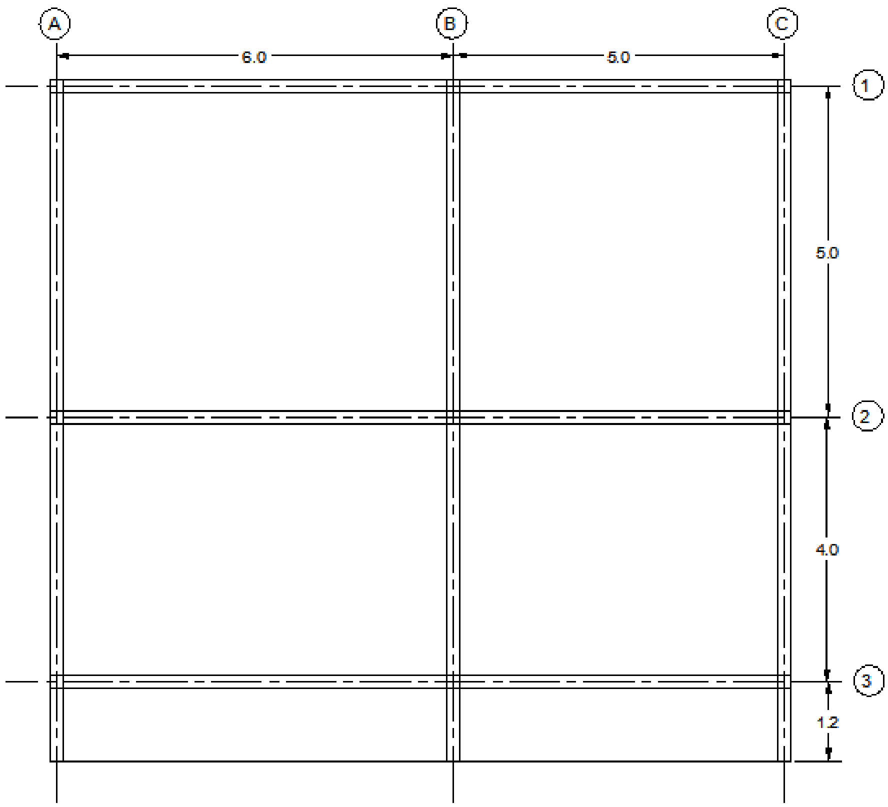
Example 2.3 Two Way slab design

1. Design the two way slab beam supported floor system if it is intended to be used for office building ,assume the partition wall load to be 2 $^{kN}/_{m^2}$

   Use C25/30   S400   Cover 25 mm



Solution

Step 1: Material property

C25/30 $f_{cd} = \frac{0.85*25}{1.5} = 14.1667\ Mpa f_{ctm} = 2.6\ mpa$

S400$f_{yd} = \frac{400}{1.15} = 347.826\ Mpa$

Step 2: Depth determination

Assumption: - Slab is lightly reinforced ($\rho = 0.5\ \%$)

$$\rho_o = \sqrt{f_{ck}} * 10^{-3} = 5 * 10^{-3}$$

For $\rho \le \rho_o \frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} \quad + \; 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]$

Panel 1& 2 – End span of two way slab   From Table 7.4 N

K=1.3 $\frac{l}{d} = 24.05$because we used S400 multiply the value by $\frac{500}{f_{yk}} = 1.25$

$$\frac{l}{d} = 24.05 * 1.25 = 30.0625 \qquad l = l_x = 5000\, mm \qquad d = 166.320\, mm$$

Panel 3& 4 – Interior span

K=1.5 $\frac{l}{d} = 27.75$ because we used S400 multiply the value by $\frac{500}{f_{yk}} = 1.25$

$$\frac{l}{d} = 27.75 * 1.25 = 34.6875 \qquad l = l_x = 4000\, mm \qquad d = 115.315\, mm$$

Cantilever

K = 0.4

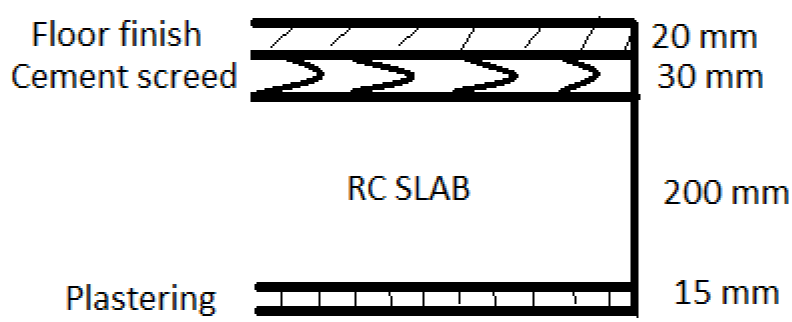
$$\frac{l}{d} = 0.4\left[11 + 1.5\sqrt{5}\right] * \frac{500}{400} \qquad d = 129.729\, mm$$

Governing depth is from panel 1 and panel 2.

Using $\emptyset\ 10\ and\ cover\ 25\ mm$H $= 166.320 + 25 + \frac{10}{2} = 196.32 \quad Use\ H = 200\ mm$

Step 3: Loading

- <u>Permanent load</u>



Floor finish	$20 * 10^{-3} * 27$	$0.54 \ ^{KN}/_{m^2}$
Cement screed	$30 * 10^{-3} * 23$	$0.69 \ ^{KN}/_{m^2}$
RC slab	$200 * 10^{-3} * 25$	$5 \ ^{KN}/_{m^2}$
Plastering	$15 * 10^{-3} * 25$	$0.375 \ ^{KN}/_{m^2}$
Load from paretion		$2 \ ^{KN}/_{m^2}$
		$G_k = 8.605 \ ^{KN}/_{m^2}$

- <u>Variable Loading</u>

For office $Q_k \ from \ 2 \ to \ 3 \ ^{KN}/_{m^2} \quad take \ Q_k = \ 3 \ ^{KN}/_{m^2}$

- **Design load for the slab**

$$P_d = 1.35\,DL + 1.5\,LL = 1.35 * 8.605 + 1.5 * 3 = 16.116 \ ^{KN}/_{m^2}$$

Parapet wall on the cantilever
Using 20 cm HCB with height of 1.5 m $P_{d,par} = 1.35(0.2 * 1.5 * 23) = 9.315\,KN$

Step 4: Analysis

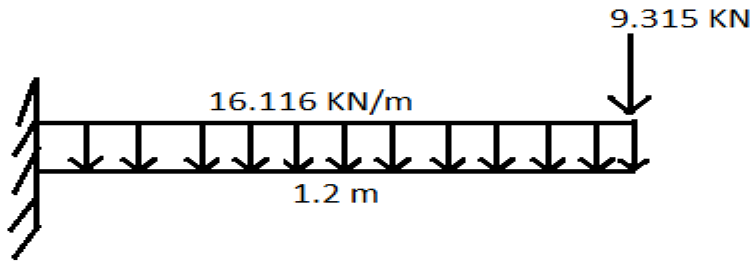
NB panel 3 & 4 are assumed to be simply supported at the intersection between the panel and cantilever.

$$M_{sx} = \beta_{sx} q {l_x}^2 M_{sy} = \beta_{sy} q {l_x}^2 \qquad q = 16.116 \ ^{KN}/_{m^2}$$

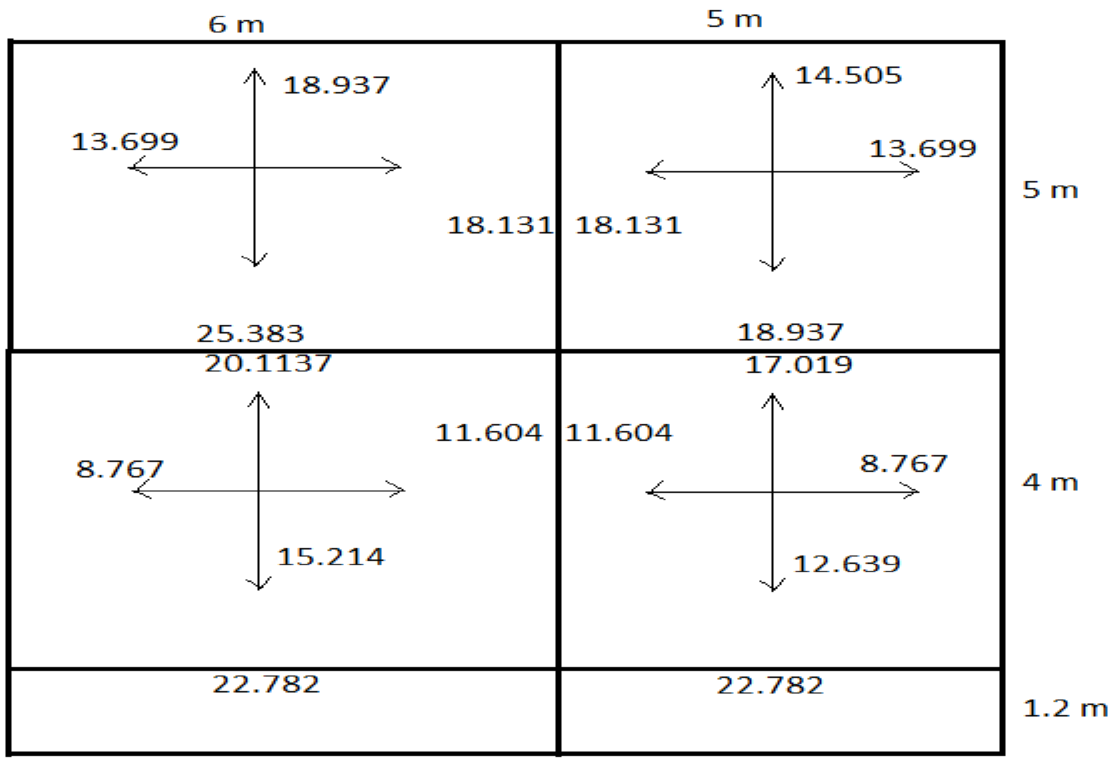
P	Type	l_y	l_x	$\frac{l_y}{l_x}$	$\beta_{sx,sup}$	$\beta_{sx,span}$	$\beta_{sy,sup}$	$\beta_{sy,span}$	$M_{sx,sup}$	$M_{sx,spa}$	$M_{sy,sup}$	$M_{sy,span}$
1	*	6	5	1.1	0.063	0.047	0.045	0.034	25.383	18.93	18.131	13.699
2	*	5	5	1	0.047	0.036	0.045	0.034	18.937	14.505	18.131	13.699
3	*	6	4	1.5	0.078	0.059	0.045	0.034	20.113	15.214	11.604	8.767
4	*	5	4	1.25	0.066	0.049	0.045	0.034	17.019	12.639	11.604	8.767

"*" = adjacent side discontinues

Cantilever Taking 1 m strip

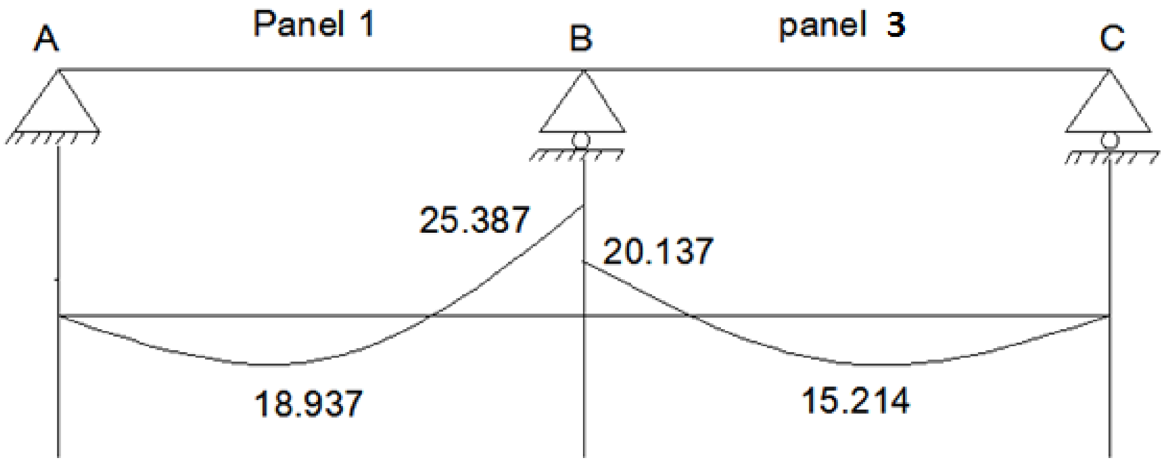


$$m = 16.11675 * \frac{1.2^2}{2} + 9.315 * 1.2 \; = 22.782 \; {}^{KNm}/_{m}$$



Step 5: Adjust the unequal edge moment

- <u>Between Panel 1 and panel 3</u>



$$Change = \frac{25.383 - 20.1137}{20.1137} * 100 = 26.197\% > 10\% \ \ use\ moment\ distribution$$

	Member	Stiffness	`	D.F
Joint B	BA	$\frac{I}{5}$	$0.45I$	0.444
	BC	$\frac{I}{4}$		0.556

	B		
D.F	0.444	0.556	
	25.383	-20.1137	
	-3.339	-2.929	
	-23.043	-23.043	

Adjusted support moment is 23.043 $^{KNM}/_{m}$

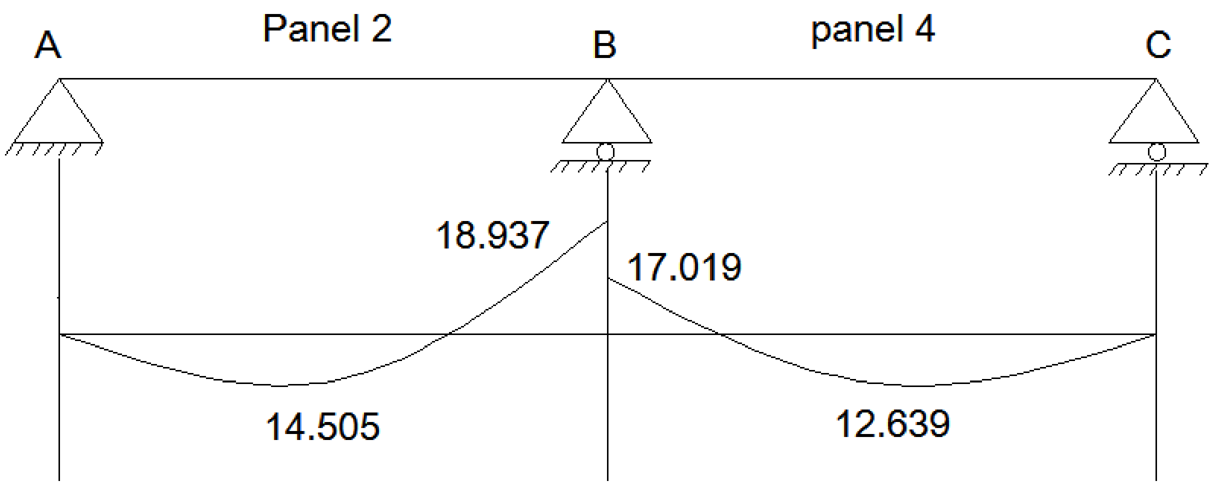
<u>Span moment on panel 1</u>

$$M_1 = (25.383 + 18.937) - 23.043 = 21.277\ ^{KNm}/_{m}$$

<u>Span moment on panel 3</u>

$$M_3 = (20.1137 + 15.214) - 23.043 = 12.2847\ ^{KNm}/_{m}$$

- Between Panel 2 and panel 4



$$Change = \frac{18.937 - 17.019}{17.019} * 100 = 11.219\% > 10\% \ \ use\ moment\ distribution$$

	Member	Stiffness	`	D.F
Joint B	BA	$\frac{I}{5}$	$0.45I$	0.444
	BC	$\frac{I}{4}$		0.556

	B		
D.F	0.444	0.556	
	18.937	-17.019	
	-0.8515	-1.0664	
	18.085	-18.085	

Adjusted support moment is 18.085 $^{KNM}/_{m}$

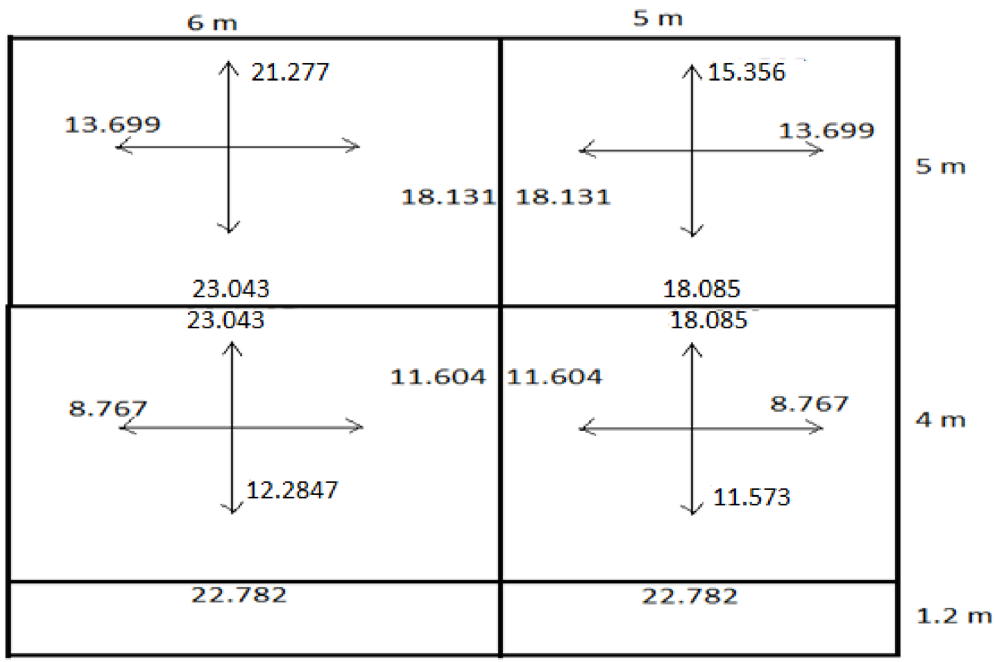
Span moment on panel 2

$$M_2 = (18.937 + 14.505) - 18.085 = 15.396\, ^{KNm}/_{m}$$

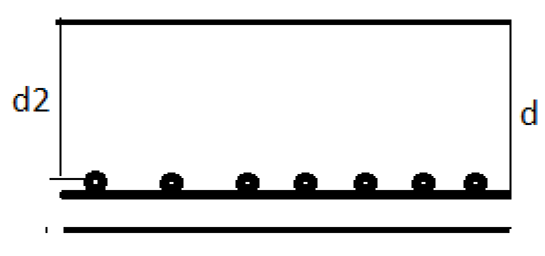
Span moment on panel 4

$$M_4 = (17.019 + 12.684) - 18.085 = 11.573\, {}^{KNm}/_{m}$$

The adjusted design moment is given below



Step: 6 Design for flexure



$$d = 200 - 25 - \frac{10}{2} = 170\ mm d_2 = 200 - 25 - 10 - \frac{10}{2} = 160\ mm$$

$$a_s = 78.5\ mm^2 f_{cd} = 14.1667\ mpa \quad f_{yd} = 347.826\ mpa$$

$$A_{s,min} = 0.26 * \frac{f_{ctm}}{f_{yk}} b_t d > 0.013 b_t d \quad = \quad 287.3\ mm^2$$

$$S_{min} = \frac{b * a_s}{A_s} = \frac{1000 * 78.5}{287.3} = 273.372\ mm$$

$$Use\ \emptyset\ 10\ C|C\ 270\ mm$$

$$S_{max} = \begin{cases} 3h \\ 400 \end{cases} = 400\ mm$$

M_{sd}	d	μ	K_z	Z	A_s	$Spacing$	$Spacing\ prov$
13.699	160	0.0377	0.978	156.48	251.691	312.048	$\emptyset 10\ C\|C\ 270$
21.277	170	0.0519	0.971	165.07	370.578	211.83	$\emptyset 10\ C\|C\ 210$
15.356	170	0.0375	0.978	166.26	265.538	295.625	$\emptyset 10\ C\|C\ 270$
12.2847	160	0.0338	0.977	156.32	225.937	347.44	$\emptyset 10\ C\|C\ 270$
11.573	170						$\emptyset 10\ C\|C\ 270$
8.767	160						$\emptyset 10\ C\|C\ 270$
18.131	170	0.044	0.973	165.41	315.136	249.09	$\emptyset 10\ C\|C\ 240$
23.043	170	0.056	0.969	164.73	402.165	195.193	$\emptyset 10\ C\|C\ 190$
18.085	170	0.044	0.973	165.41	314.336	249.732	$\emptyset 10\ C\|C\ 240$
11.604	170						$\emptyset 10\ C\|C\ 270$
22.782	170	0.0556	0.969	164.73	397.609	197.429	$\emptyset 10\ C\|C\ 190$

Secondary reinforcement = 20% As main $= 0.2 * 197.429 = 39.4854\ mm^2$
Provide $\emptyset 10\ C|C\ 270$

Step 7: Check shear capacity of the slab

$$V_{RD,C} = \left[C_{RD,C} * K(100\rho f_{ck})^{\frac{1}{3}} + K_1 \sigma_{CP} \right] b_w d > (V_{min} + K_1 \sigma_{CP}) b_w d$$

$$C_{RD,C} = \frac{0.18}{\gamma_c} = 0.12 \qquad K1 = 0.15$$

$$V_{min} = 0.035 K^{3/2} f_{ck}^{\frac{1}{2}}$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2 \qquad K = 2$$

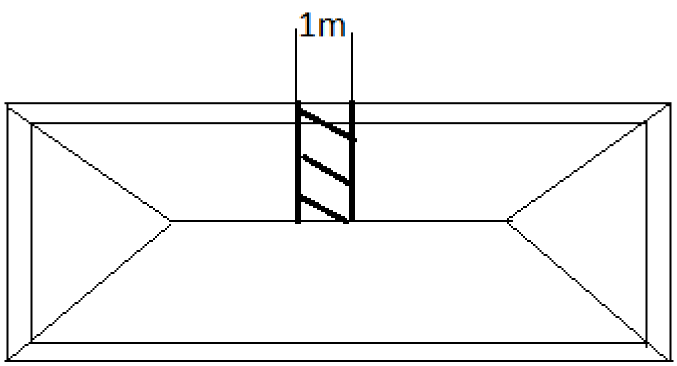
Taking minimum reinforcement $\emptyset 10\ C|C\ 270 \rho = \frac{A_s}{b_w d} = 1.7102 * 10^{-3}$

$$\sigma_{CP} = \frac{N_{ed}}{A_c} < 0.2 f_{cd} = 0$$

Taking one meter strip B=1000 mm and d=170 mm

$$V_{RD,c} = 84.146\ KN$$

Maximum acting shear



Assuming the beam width to be 200 mm

$$V_{sd} = P_d(0.5 l_n - d) b_w$$

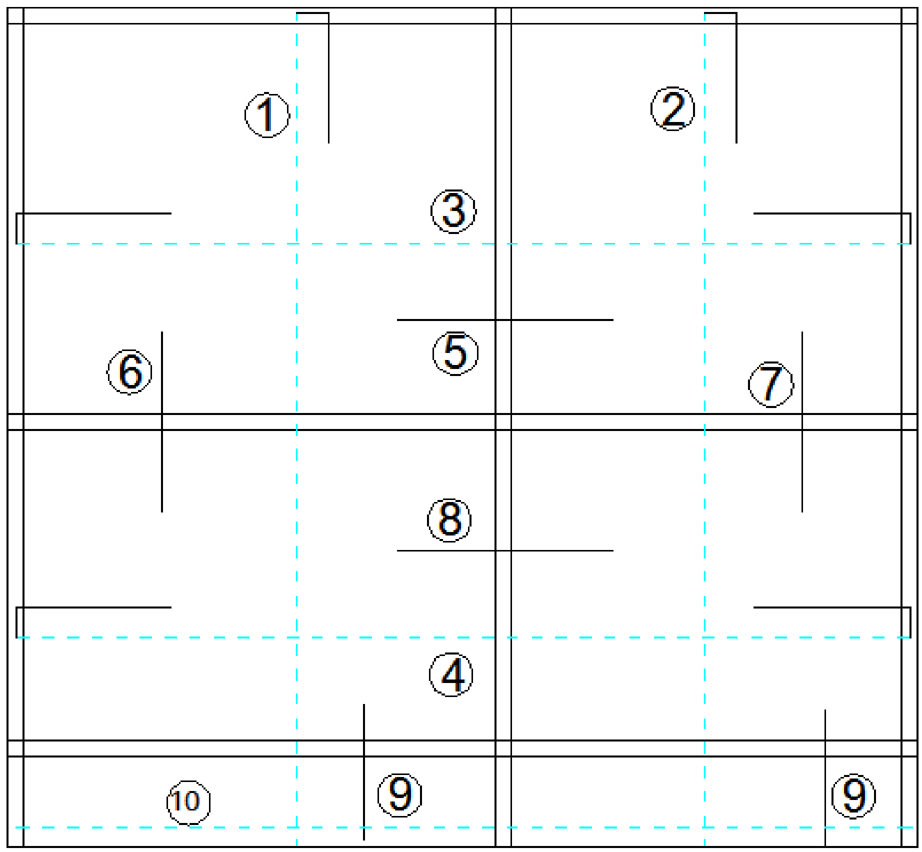
$$p_d = 16.116\ {}^{KN}/_{m^2}\ l_n = 5 - 0.2 = 4.8\ m \quad taking\ unit\ meter\ width$$

$$V_{sd} = 16.116(0.5(4.8) - 0.17) * 1$$

$$V_{sd} = 35.940\ KN$$

$$V_{RD,C} > V_{sd} \qquad The\ seection\ is\ adequte$$

Step 8: Detailing



1	$\emptyset 10\ \ c/c\ \ \ 210\, mm$	6	$\emptyset 10\ \ c/c\ \ \ 190\, mm$
2	$\emptyset 10\ \ c/c\ \ \ 270\, mm$	7	$\emptyset 10\ \ c/c\ \ \ 240\, mm$
3	$\emptyset 10\ \ c/c\ \ \ 270\, mm$	8	$\emptyset 10\ \ c/c\ \ \ 270\, mm$
4	$\emptyset 10\ \ c/c\ \ \ 270\, mm$	9	$\emptyset 10\ \ c/c\ \ \ 190\, mm$
5	$\emptyset 10\ \ c/c\ \ \ 240\, mm$	10	$\emptyset 10\ \ c/c\ \ \ 270\, mm$

Step 9: Load transfer to beam

To consider pattern loading, load is transferred separately for dead and live load cases.

$$\text{Factored dead load} = 1.35 * 8.605 = 11.61675\ {}^{KN}/_{m^2}$$

$$Factored\ live\ load = 1.5 * 3 = 4.5\ {}^{KN}/_{m^2}$$

$$factored\ load\ on\ the\ parapet\ wall = 9.315\ KN$$

$$\boldsymbol{V_i = \beta_{vi} q_i l_x}$$

Case 1 Dead load

$$q_i = 11.61675\ {}^{KN}/_{m^2}$$

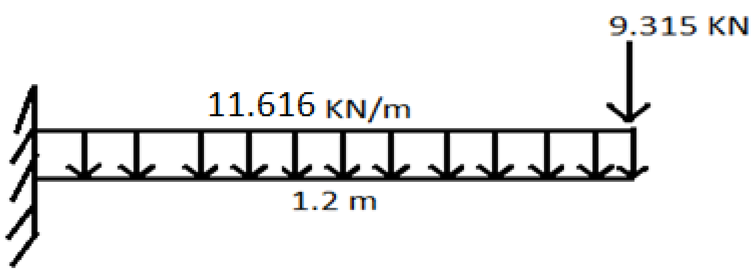
P	Type	l_y	l_x	$\frac{l_y}{l_x}$	$\beta_{vx,c}$	$\beta_{vx,d}$	$\beta_{vy,c}$	$\beta_{vy,d}$	$V_{x,c}$	$V_{x,d}$	V_{yc}	V_{yd}
1	*	6	5	1.1	0.47	0.31	0.4	0.26	27.299	18.006	23.23	15.102
2	*	5	5	1	0.4	0.26	0.4	0.26	23.23	15.102	23.23	15.102
3	*	6	4	1.5	0.54	0.35	0.4	0.26	25.092	16.263	18.58	12.081
4	*	5	4	1.25	0.485	0.32	0.4	0.26	22.536	14.87	18.58	12.081

Case 2 Live load

$$q_i = 4.5\ {}^{KN}/_{m^2}$$

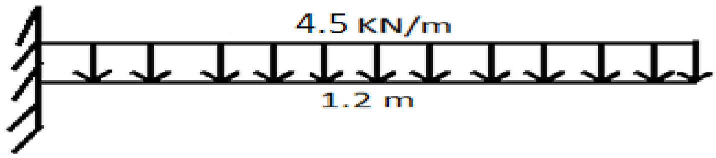
P	Type	l_y	l_x	$\frac{l_y}{l_x}$	$\beta_{vx,c}$	$\beta_{vx,d}$	$\beta_{vy,c}$	$\beta_{vy,d}$	$V_{x,c}$	$V_{x,d}$	V_{yc}	V_{yd}
1	*	6	5	1.1	0.47	0.31	0.4	0.26	10.575	6.975	9	5.85
2	*	5	5	1	0.4	0.26	0.4	0.26	9	5.85	9	5.85
3	*	6	4	1.5	0.54	0.35	0.4	0.26	9.72	6.3	7.2	4.68
4	*	5	4	1.25	0.485	0.32	0.4	0.26	8.73	5.76	7.2	4.68

Load transfer on the cantilever part

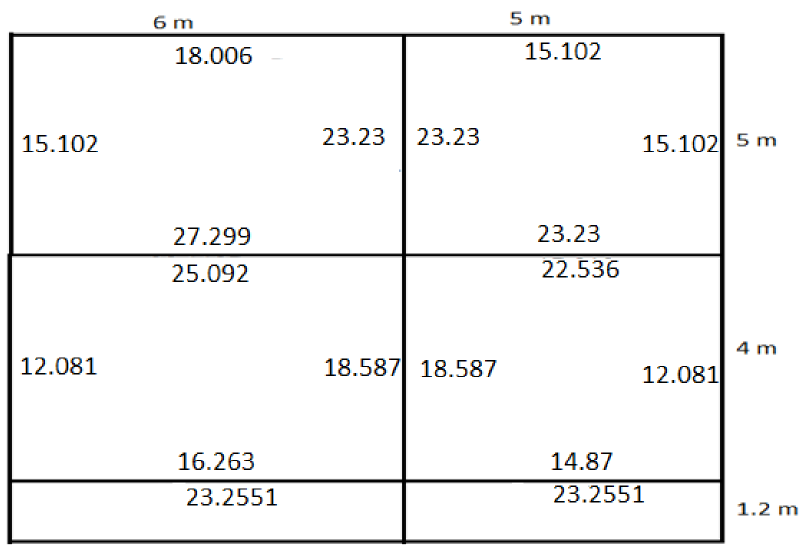


Dead load case only $V = 23.2551\ KN$

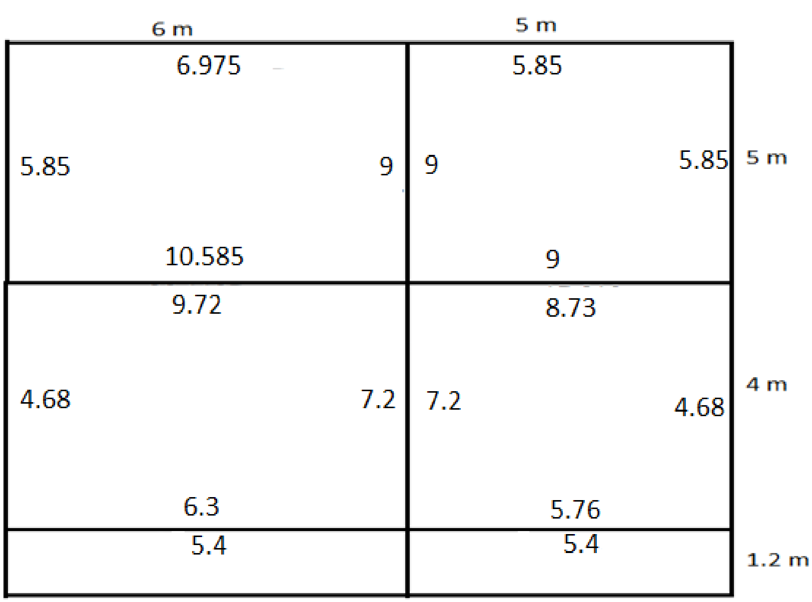
Live load case only $V = 5.4\ KN$



Load on beam due to dead load only



Load on beam due to live load only



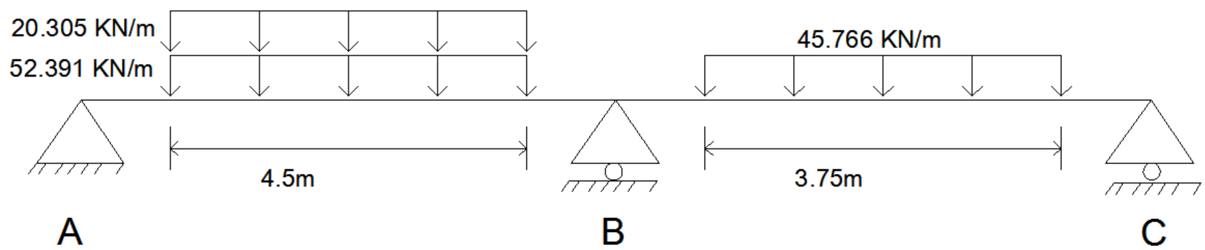
Loading on beam

- Load from slab
- partition load directly supported on the beam
- Own weight of the beam

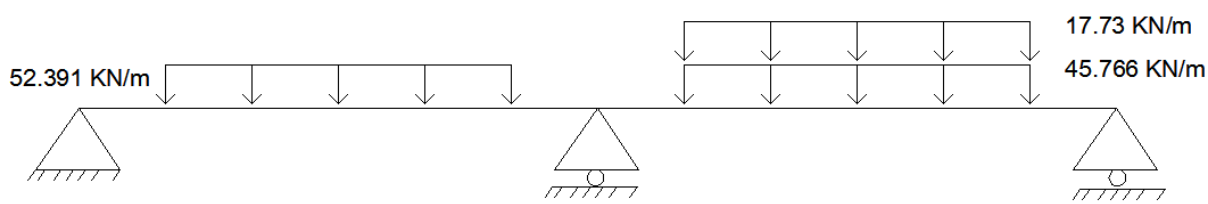
▪ For this particular case without partition load on beam and excluding the self-weight

The load on axis 2 will be

Maximum span moment at AB



Maximum span moment at BC



Maximum support moment at B