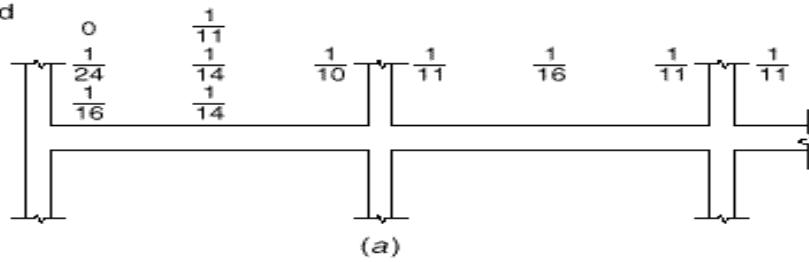
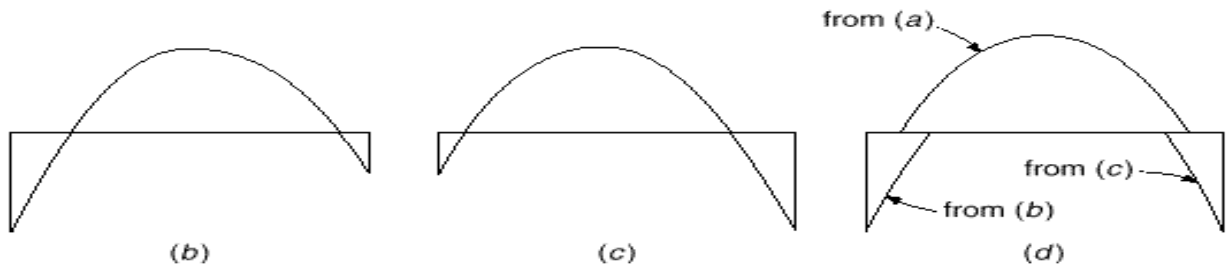
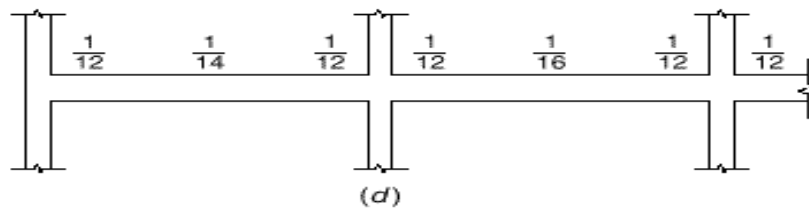
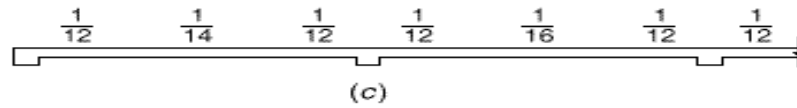
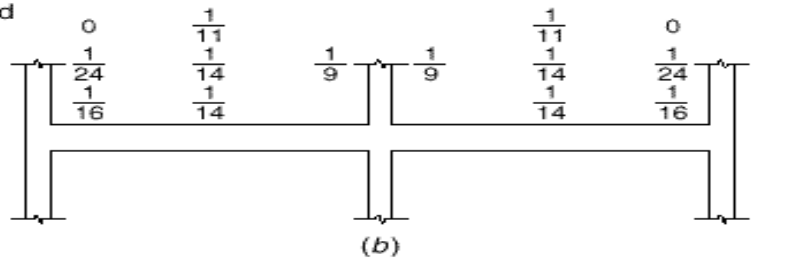


Discontinuous end
unrestrained:
Spandrel:
Column:



Discontinuous end
unrestrained:
Spandrel:
Column:



and moment envelope for a continuous beam: (a) maximum positive moment; (b) maximum negative moment at right end; (d) composite moment envelope.

One way joist Slab formula:

$$R_n = \frac{M_u/0.9}{b * d^2} * 10^6 = Mpa , \quad d = h - 20 - \frac{d_b}{2} - d_s = _mm$$

$$m = \frac{f_y}{0.85 * f_c} = _ , \quad \rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 * R_n * m}{f_y}} \right) = _$$

$$A_s = \rho * b * d = _mm^2 , \quad A_{s\ min} = \max \left[\begin{array}{l} \frac{1.4}{420} * b_w * d \\ \frac{\sqrt{f_c}}{4 * 420} * b_w * d \end{array} \right] = _mm^2$$

$$n = \frac{A_s}{A_b} = \text{real number} \quad \text{for flexural} \rightarrow n \phi d_b$$

$$A_{s\ shrinkage} = 0.0018 * 1000 * h_f = _mm^2$$

$$S_{max} = \min \left(\frac{A_b}{A_s} * 1000 = _mm , 300mm , 4 * h_f \right)$$

$$V_u = \frac{C_v * w_u * L_n}{2} , \quad \phi V_c = 1.1 * 0.75 * \frac{\sqrt{f_c}}{6} * b_w * d * 10^{-3} = _kn$$

$\phi V_c > V_u$ no need to shear reinforcement

Beam formula:-

$$R_n = \frac{M_u/0.9}{1000 * d^2} * 10^6 = _Mpa$$

$$d = 600 - 40 - d_s - \frac{d_b}{2} = _mm \text{ (one layer)}$$

$$d = 600 - 40 - d_s - 25 - \frac{S}{2} = _mm \text{ (two layer)}$$

$$m = \frac{f_y}{0.85 * f_c} = _ , \quad \rho = \frac{1}{m} \left(1 - \sqrt{1 - \frac{2 * R_n * m}{f_y}} \right) = _$$

$$A_s = \rho * b * d = _mm^2 ,$$

$$A_{s\ min} = \max \left[\begin{array}{l} \frac{1.4}{420} * b_w * d \\ \frac{\sqrt{f_c}}{4 * 420} * b_w * d \end{array} \right] = _mm^2$$

check $A_s > A_{s \min}$ ok , $n = \frac{A_s}{A_b} = \text{real number}$

check $b_{\min} = 80 + 60 + (n - 1)(d_b + 25) > b$ ok

if $\frac{a_b}{d} = \beta_1 \left(\frac{600}{600 + f_y} \right) > \frac{a}{d} \quad \therefore f_s = f_y$

if $\frac{a_{TCL}}{d_t} = 0.375 \beta_1 > \frac{a}{d_t} \quad \therefore \text{Tension - controlled section}$

$$\phi V_c = 0.75 * \frac{\sqrt{f_c}}{6} * b_w * d * 10^{-3} = _kn$$

$$V_s = \frac{V_u - \phi V_c}{0.75} = _kn$$

$$A_v = 2 * \pi \frac{d_s^2}{4} = _mm^2 \text{ (two leg)}$$

$$S = \frac{A_v * f_y * d}{V_s} * 10^{-3} _mm$$

if $V_s \leq 2V_c$

$$S_{max} = \min\left(600, \frac{d}{2}\right)$$

if $2V_c < V_s \leq 4V_c$

$$S_{max} = \min\left(300, \frac{d}{4}\right)$$

if $V_s > 4V_c$

need to increase the section

if $V_u < \frac{1}{2} \phi V_c$

no need for shear reinforcement

if $\phi V_c > V_u > \frac{1}{2} \phi V_c$

need to provide minimum

$$a_{v \min} = \max\left\{\left(\frac{1}{16} \sqrt{f_c} * b_w * \frac{s}{f_y}\right), \left(\frac{b_w * s}{3f_y}\right)\right\}$$

Example 1:

The figure shown below shows the architectural plan for 3-story building. The slab will be designed to carry super imposed dead load (1.5KN/m^2) and Live load (3KN/m^2) by compressive strength 25 Mpa and steel yield strength 420 Mpa.

Given data:

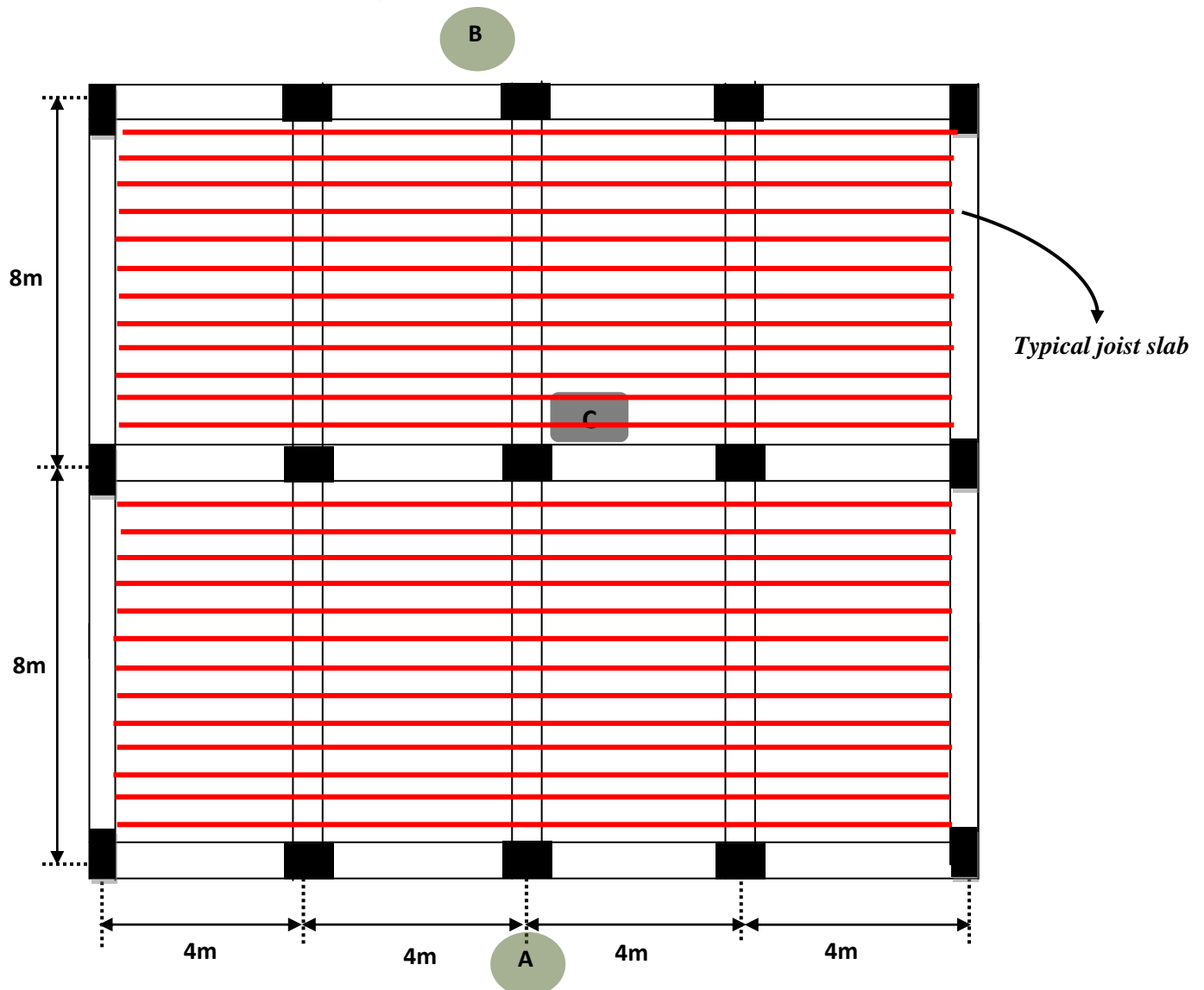
The one way joist slab is used.

Weight of block = 12KN/m^3

$h_f = 50\text{ mm}$, $h_w = 250\text{ mm}$, $b_w = 120\text{ mm}$, $S = 500\text{ mm}$

The beams dimensions = (300,600) mm

The columns dimensions = (300,500) mm



Determine the following:

- 1- Check if Slab thickness meets SBC304.
- 2- Design the slab for Max +Ve moment.
- 3- Design the slab for Max -Ve moment.
- 4- Check the slab for Shear reinforcement.
- 5- Draw the details and show all value.
- 6- Design the beam (A-B) for flexure and shear.
- 7- Calculate the axial load acting on column C.

Solution:

$$h_{min} = \frac{l}{21} = \frac{4000}{21} = 190.5 \text{ mm}$$

$$h_{min} = \frac{l}{18.5} = \frac{4000}{18.5} = 216.2 \text{ mm}$$

$$h_{min} = \max(190.5, 216.2) = 216.2 \text{ mm}$$

$$h = 300 \text{ mm} > h_{min} \rightarrow \text{ok}$$

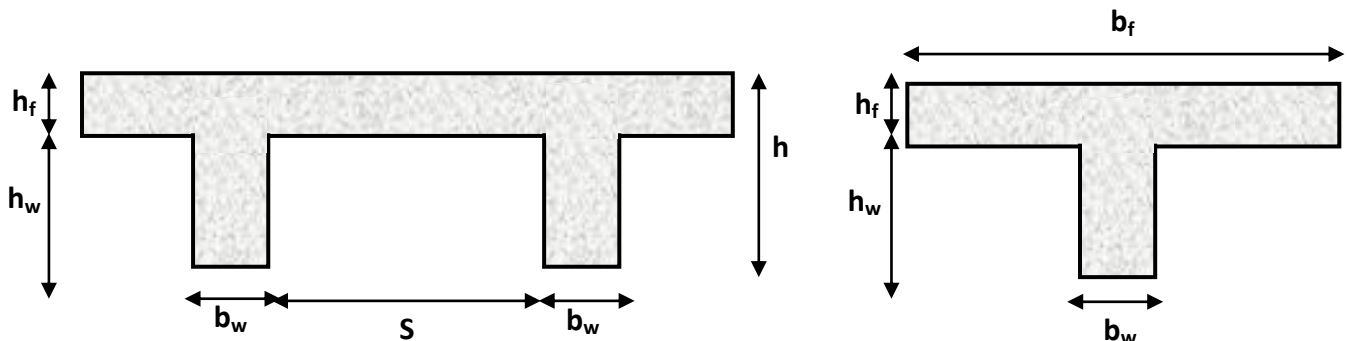
$$h_f \geq \left(50 \text{ mm}, \frac{500}{12} = 41.67 \text{ mm} \right) = 50 \text{ mm} \quad h_f = 50 \text{ mm} \quad \text{ok}$$

$$h_w \geq (3.5 * 120 = 4200 \text{ mm}) \quad h_w = 250 \text{ mm} \quad \text{ok}$$

$$b_w \geq 100 \text{ mm} \quad b_w = 120 \text{ mm} \quad \text{ok}$$

$$S \geq 800 \text{ mm} \quad S = 500 \text{ mm} \quad \text{ok}$$

$$b_f = 500 + 120 = 620 \text{ mm}$$



CE472

DL = own weight + superimposed

$$DL = (0.05 * 25) + 1.5 = 2.75 \text{ kn/m}^2$$

$$LL = 3 \text{ kn/m}^2$$

$$Wu = 1.4DL + 1.7LL$$

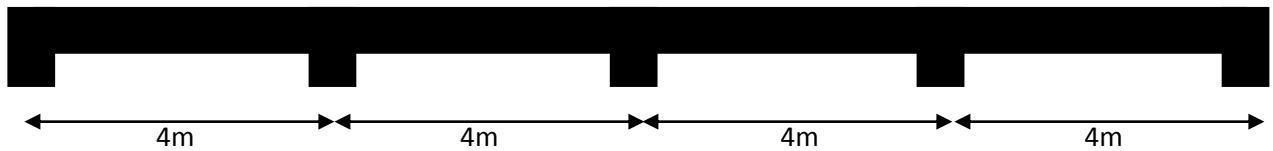
$$Wu = 1.4(2.75) + 1.7(3) = 8.95 \text{ kn/m}^2$$

Load on typical rip:

$$DL_{/rip} = (2.75 * 0.62) + (0.25 * 0.12 * 25) + (0.5 * 0.25 * 12) = 3.955 \text{ kn/m}$$

$$LL_{/rip} = (3 * 0.62) = 1.86 \text{ kn/m}$$

$$Wu_{/rip} = 1.4(3.955) + 1.7(1.86) = 8.7 \text{ kn/m}$$



Lu (m)	4m			4m			4m			4m		
Ln (m)	3.7m			3.7m			3.7m			3.7m		
Cm	1/24	1/14	1/10	1/11	1/16	1/11	1/11	1/16	1/11	1/10	1/14	1/24
Wu(kn/m ²)		8.7	8.7							8.7	8.7	
Mu(kn.m)		8.51	11.91							11.91	8.51	
d (mm)		267	267							267	267	
Rn (Mpa)		0.214	1.55							1.55	0.214	
P		0.00051	0.0038							0.0038	0.00051	
As (mm ²)		84.75	123							123	84.75	
As _{min} (mm ²)		106.8	106.8							106.8	106.8	
As (mm ²)		106.8	123							123	106.8	
N		1.36	1.57							1.57	1.36	
Use		2φ10	2φ10							2φ10	2φ10	
As _{shrinkage} (mm ²)		90	90							90	90	
S _{max} (mm)		200	200							200	200	
Cv			1.15							1.15		
Vu (kn)			18.51							18.51		
φVc (kn)			22.03							22.03		

sample of calculations

$$L_u = 4m, \quad L_n = 4 - 0.15 - 0.15 = 3.7m$$

$$C_m = \frac{1}{10}, \quad M_u = \frac{1}{10} * 8.7 * 3.7^2 = 11.91 \text{ kn.m}$$

$$d = 300 - 20 - 8 - \frac{10}{2} = 267 \text{ mm}, \quad R_n = \frac{11.91/0.9}{120 * 267^2} * 10^6 = 1.55 \text{ Mpa}$$

$$m = \frac{420}{0.85 * 25} = 19.765, \quad \rho = \frac{1}{19.765} \left(1 - \sqrt{1 - \frac{2 * 1.55 * 19.765}{420}} \right) = 0.0038$$

$$A_s = 0.0038 * 120 * 267 = 123 \text{ mm}^2, \quad A_{s \text{ min}} = \max \left[\begin{array}{l} \frac{1.4}{420} * 120 * 267 \\ \frac{\sqrt{f_c}}{4 * 420} * 100 * 225 \end{array} \right] = 106.8 \text{ mm}^2$$

$$n = \frac{A_s}{A_b} = \frac{123}{78.5} = 1.57 \approx 2$$

for flexural → use 2 Ø10

$$A_{s \text{ shrinkage}} = 0.0018 * 1000 * 50 = 90 \text{ mm}^2$$

$$S_{max} = \min \left(\frac{78.5}{90} * 1000 = 872.2 \text{ mm}, 300 \text{ mm}, 4 * 50 = 200 \right) = 200 \text{ mm}$$

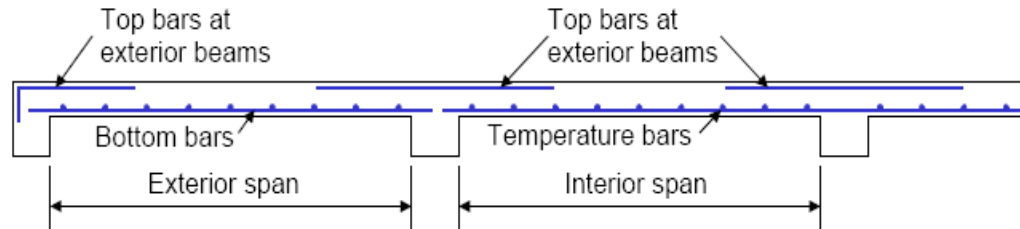
for shrinkage → use Ø10@200 mm

$$C_v = 1.15, \quad V_u = \frac{1.15 * 8.7 * 3.7}{2} = 18.51 \text{ kn}$$

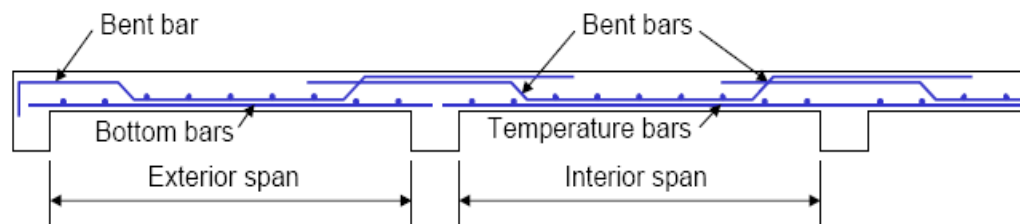
$$\phi V_c = (1.1)0.75 * \frac{\sqrt{25}}{6} * 120 * 267 * 10^{-3} = 22.03 \text{ kn}$$

$\phi V_c > V_u$ → shear reinforcement is not needed

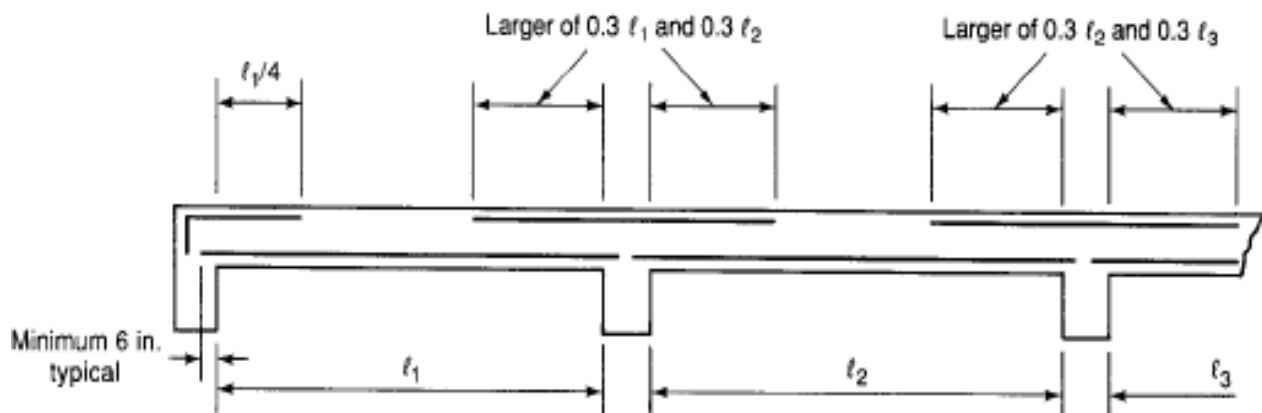
Typical reinforcement in a one-way slab



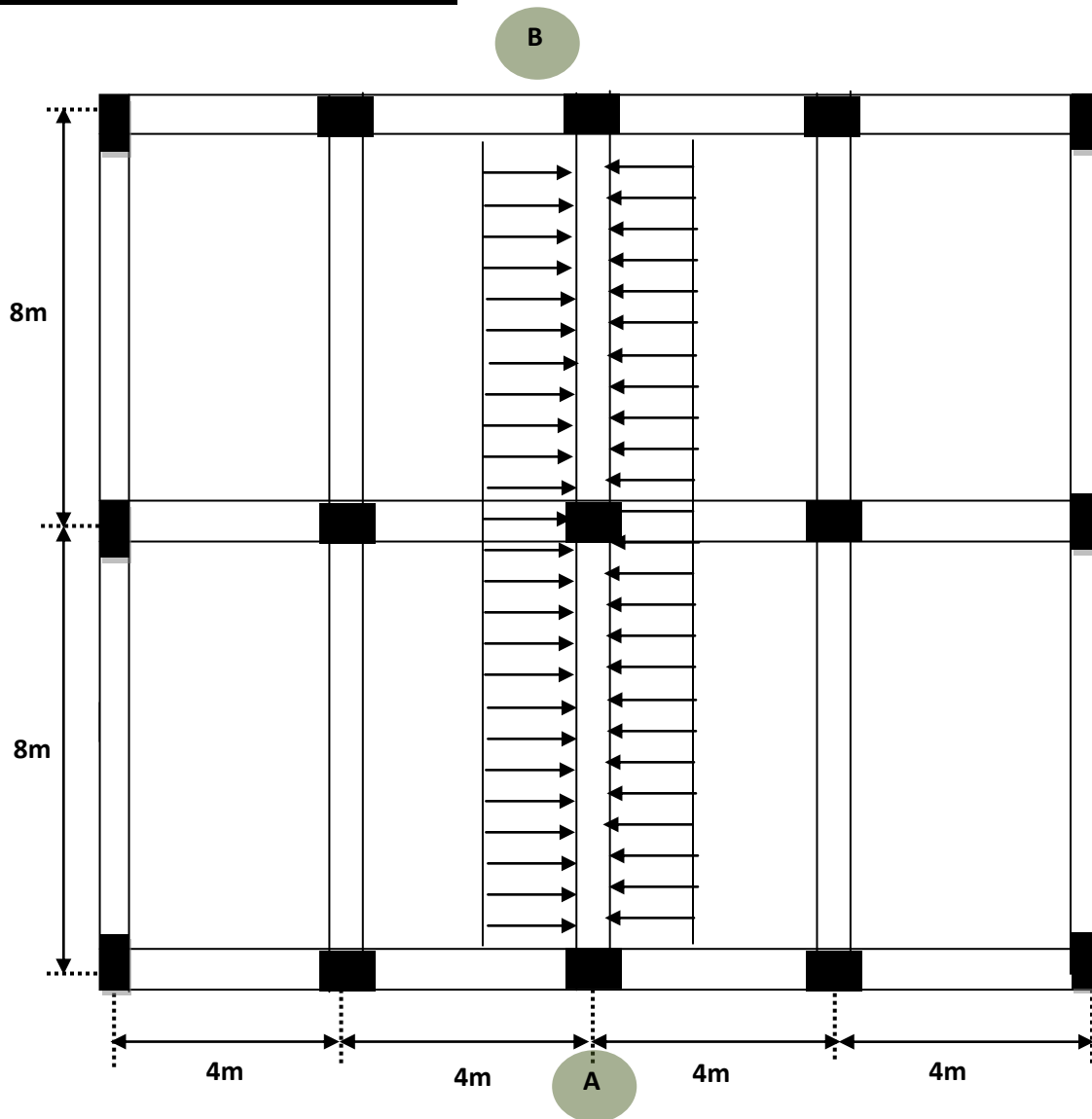
(a) Straight top and bottom bars



(b) Alternate straight and bent bars



(c) One-way slab

Design of beam (A-B) :-**Load acting on the beam (A-B)**

1- Dead load:

$$DL = \frac{3.955}{0.62} * (1.85 + 1.85) + (1.5 * 0.3) + (0.6 * 0.3 * 25) = 28.55 \text{ kn/m}$$

2- Live load

$$LL = \frac{1.86}{0.62} * (1.85 + 1.85) + (3 * 0.3) = 12 \text{ kn/m}$$

$$Wu = 1.4(28.55) + 1.7(12) = 60.37 \text{ kn/m}$$



Lu (m)	8m			8m		
Ln(m)	7.7m			7.7m		
Wu(kn/m)	60.37					
Cm	1/16	1/14	1/9	1/9	1/14	1/16
Mu (kn.m)		255.7	397.7	397.7	255.7	
d(mm)		517.5 (two layer)	517.5(two layer)	517.5(two layer)	517.5 (two layer)	
Rn(Mpa)		3.54	5.5	5.5	3.54	
P		0.0093	0.0155	0.0155	0.0093	
As(mm ²)		1443.825	2406.375	2406.375	1443.825	
As _{min} (mm ²)		517.5	517.5	517.5	517.5	
n		4.59≈5	7.66≈8	7.66≈8	4.59≈5	
Fs=Fy		Ok	Ok	Ok	Ok	
TCL		Ok	Ok	Ok	Ok	
b _{min} (mm)		OK	OK	OK	OK	
Use		5Ø20	8Ø20	8Ø20	5Ø20	
Cv	1		1.15	1.15		1
Vu (kn)	232.4		267.3	267.3		232.4
φVc (kn)	97.03		97.03	97.03		97.03
Vs (kn)	180.5		227.03	227.03		180.5
Av (mm ²)	157		157	157		157
S (mm)	189.1		150.31	150.31		189.1
S _{max} (mm)	258.75		258.75	258.75		258.75
Use	Ø10@185		Ø10@150	Ø10@150		Ø10@185

sample of cacluations

$$L_u = 8m, \quad L_n = 8 - 0.15 - 0.15 = 7.7m$$

$$C_m = \frac{1}{9}, \quad M_u = \frac{1}{9} * 60.37 * 7.7^2 = 397.7 \text{ kn.m}$$

$$d = 600 - 40 - 10 - 20 - \frac{25}{2} = 517.5 \text{ mm (two layer)}$$

$$R_n = \frac{397.7/0.9}{300 * 517.5^2} * 10^6 = 5.5 \text{ Mpa}$$

$$m = \frac{420}{0.85 * 25} = 19.765, \quad \rho = \frac{1}{19.765} \left(1 - \sqrt{1 - \frac{2 * 5.5 * 19.765}{420}} \right) = 0.0155$$

$$A_s = 0.0155 * 300 * 517.5 = 2406.375 \text{ mm}^2,$$

$$A_{s \min} = \frac{1.4}{420} * 300 * 517.5 = 517 \text{ mm}^2$$

$$A_{s \min} = \frac{\sqrt{25}}{4 * 420} * 300 * 517.5 = 462.1 \text{ mm}^2$$

$$A_{s \min} = \max(517, 462.1) = 517 \text{ mm}^2$$

$$n = \frac{2406.375}{314} = 7.66 \approx 8$$

$$a = \frac{8 * 314 * 420}{0.85 * 25 * 300} = 165.5 \text{ mm}$$

$$\frac{a_b}{d} = 0.5 > \frac{a}{d} = 0.32 \quad \therefore f_s = f_y$$

$$\frac{a_{TCL}}{d_t} = 0.31875 > \frac{a}{d_t} = 0.306 \quad \therefore \text{Tension - controlled section}$$

for flexure use 8Ø20

$$C_v = 1.15, \quad V_u = \frac{1.15 * 60.3 * 7.7}{2} = 267.3 \text{ kn}$$

$$\phi V_c = 0.75 * \frac{\sqrt{25}}{6} * 300 * 517.5 * 10^{-3} = 97.03 \text{ kn}$$

$$V_s = \frac{267.3 - 97.03}{0.75} = 227.03 \text{ kn}$$

$$A_v = 2 * \pi \frac{10^2}{4} = 157 \text{ mm}^2 \text{ (two leg)}$$

$$S = \frac{157 * 420 * 517.5}{227.03} * 10^{-3} = 150.31 \text{ mm}$$

$$V_s \leq 2V_c$$

$$S_{max} = \min\left(300, \frac{517.5}{2} = 258.75\right) = 258.75 \text{ mm}$$

use Ø10@150mm

Axial load on column C:-

Beams in x- direction:

$$DL = \left\{ (0.3 * 0.6 * 25) * \frac{4}{2} \right\} + \left\{ (0.3 * 0.6 * 25) * \frac{4}{2} \right\} = 18 \text{ kn}$$

$$LL = 0 + 0 = 0 \text{ kn}$$

Beams in y- direction:

$$DL = \left\{ (0.3 * 0.6 * 25) + (1.5 * 0.3) + \left(\frac{3.955}{0.62} * 3.7 \right) \right\} * \frac{8}{2}$$

$$+ \left\{ (0.3 * 0.6 * 25) + (1.5 * 0.3) + \left(\frac{3.955}{0.62} * 3.7 \right) \right\} * \frac{8}{2} = 228.42 \text{ kn}$$

$$LL = \left\{ \frac{1.86}{0.62} (3.7) + (3 * 0.3) \right\} * \frac{8}{2} + \left\{ \frac{1.86}{0.62} (3.7) + (3 * 0.3) \right\} * \frac{8}{2} = 96 \text{ kn}$$

$$Wu = 1.4(18 + 228.42) + 1.7(96) = 508.188 \text{ kn}$$

Example 2:**Given data:-**

$$f_c = 30 \text{ Mpa} , f_y = 420 \text{ Mpa}$$

$$\text{super imposed} = 2.5 \text{ kn/m}^2 , LL = 3 \text{ kn/m}^2$$

$$h_f = 50 \text{ mm} , h_w = 250 \text{ mm} , b_w = 120 \text{ mm} , S = 500 \text{ mm} , \text{Weight of block} = 12 \text{ kn/m}^3$$

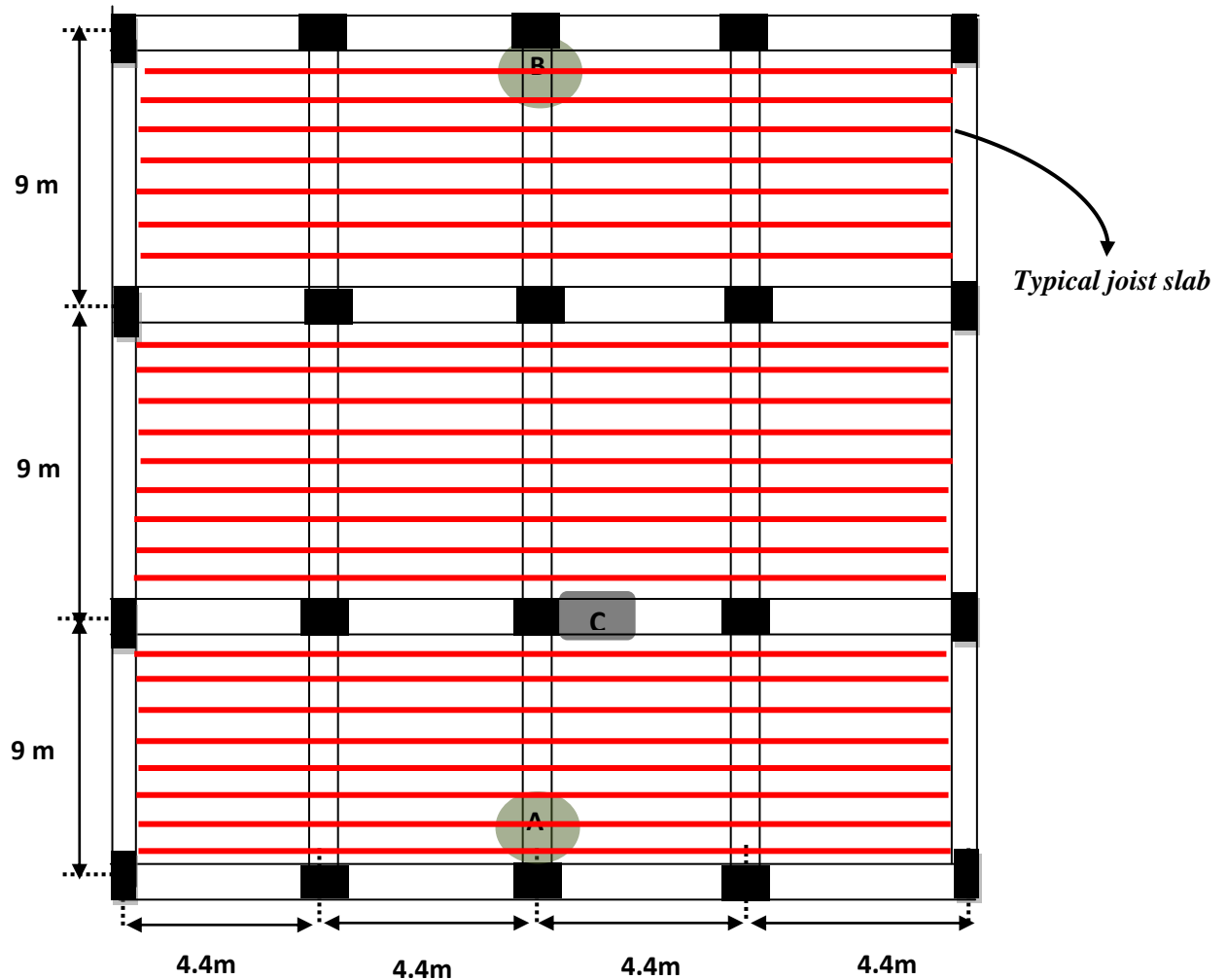
$$\text{beams size} = (700,400) , \text{columns size} (500,400) , \text{weight of wall} = 15 \text{ kn/m}$$

Required:-

Design the one way joist slab for flexure and shear.

Design the beam (A-D) for flexure and shear.

Compute the axial load on column C.



$$h_{min} = \frac{l}{28} = \frac{4400}{21} = 209.5 \text{ mm}$$

$$h_{min} = \frac{l}{24} = \frac{4400}{18.5} = 237.8 \text{ mm}$$

$$h_{min} = \max(209.5, 237.8) = 237.8 \text{ mm}$$

$$h > h_{min} \rightarrow ok$$

$$h = 300 \text{ mm} > h_{min} \rightarrow ok$$

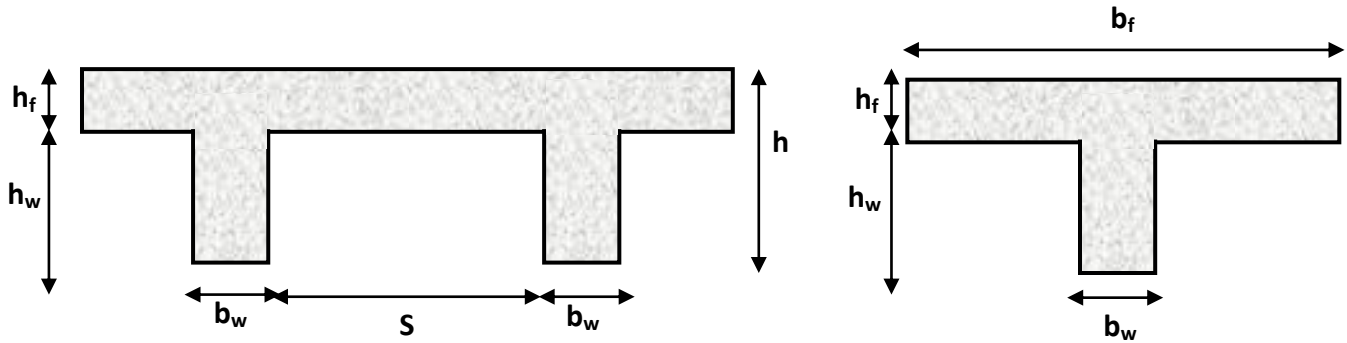
$$h_f \geq \left(50 \text{ mm}, \frac{500}{12} = 41.67 \text{ mm} \right) = 50 \text{ mm} \quad h_f = 50 \text{ mm} \quad ok$$

$$h_w \nlessgtr (3.5 * 100 = 350 \text{ mm}) \quad h_w = 250 \text{ mm} \quad ok$$

$$b_w \geq 100 \text{ mm} \quad b_w = 120 \text{ mm} \quad ok$$

$$S \nlessgtr 800 \text{ mm} \quad S = 500 \text{ mm} \quad ok$$

$$b_f = 500 + 120 = 620 \text{ mm}$$



CE472

$DL = \text{own weight} + \text{superimposed}$

$$DL = (0.05 * 25) + 2.5 = 3.75 \text{ kn/m}^2$$

$$LL = 3 \text{ kn/m}^2$$

$$Wu = 1.4DL + 1.7LL$$

$$Wu = 1.4(3.75) + 1.7(3) = 10.35 \text{ kn/m}^2$$

Load on typical rip:

$$DL_{/rip} = (3.75 * 0.62) + (0.25 * 0.12 * 25) + (0.5 * 0.25 * 12) = 4.575 \text{ kn/m}$$

$$LL_{/rip} = (3 * 0.62) = 1.86 \text{ kn/m}$$

$$Wu_{/rip} = 1.4(4.575) + 1.7(1.86) = 9.567 \text{ kn/m}$$



Lu (m)	4.4m			4.4m			4.4m			4.4m		
Ln (m)	4m			4m			4m			4m		
Cm	1/24	1/14	1/10	1/11	1/16	1/11	1/11	1/16	1/11	1/10	1/14	1/24
Wu(kn/m ²)		9.567	9.567							9.567	9.567	
Mu(kn.m)		10.93	15.31							15.31	10.93	
d (mm)		267	267							267	267	
Rn (Mpa)		0.275	1.99							1.99	0.275	
P		0.00066	0.0049							0.0049	0.00066	
As (mm ²)		109.3	157							157	109.3	
As _{min} (mm ²)		106.8	106.8							106.8	106.8	
As (mm ²)		109.3	157							157	109.3	
N		1.39	1.999							1.999	1.39	
Use		2φ10	2φ10							2φ10	2φ10	
As _{shrinkage} (mm ²)		90	90							90	90	
S _{max} (mm)		200	200							200	200	
Cv			1.15							1.15		
Vu (kn)			22.0041							22.0041		
φVc (kn)			24.13							24.13		

sample of cacluations

$$L_u = 4.4m, \quad L_n = 4.4 - 0.2 - 0.2 = 4m$$

$$C_m = \frac{1}{10}, \quad M_u = \frac{1}{10} * 9.567 * 4^2 = 15.31 \text{ kn.m}$$

$$d = 300 - 20 - 8 - \frac{10}{2} = 267 \text{ mm}, \quad R_n = \frac{15.31/0.9}{120 * 267^2} * 10^6 = 1.99 \text{ Mpa},$$

$$m = \frac{420}{0.85 * 30} = 16.471, \quad \rho = \frac{1}{16.471} \left(1 - \sqrt{1 - \frac{2 * 1.99 * 16.471}{420}} \right) = 0.0049$$

$$A_s = 0.0049 * 120 * 267 = 157 \text{ mm}^2, \quad A_{s \text{ min}} = \max \left[\begin{array}{l} \frac{1.4}{420} * 120 * 267 \\ \frac{\sqrt{f_c}}{4 * 420} * 100 * 225 \end{array} \right] = 106.8 \text{ mm}^2$$

for flexural → use 2Ø110

$$A_{s \text{ shrinkage}} = 0.0018 * 1000 * 50 = 90 \text{ mm}^2$$

$$S_{\text{max}} = \min \left(\frac{78.5}{90} * 1000 = 872.2 \text{ mm}, 300 \text{ mm}, 4 * 50 = 200 \right) = 200 \text{ mm}$$

for shrinkage → use Ø10@200 mm

$$C_v = 1.15, \quad V_u = \frac{1.15 * 9.567 * 4}{2} = 22.0041 \text{ kn}$$

$$\phi V_c = (1.1)0.75 * \frac{\sqrt{30}}{6} * 120 * 267 * 10^{-3} = 24.13 \text{ kn}$$

$\phi V_c > V_u \rightarrow$ *shear reinforcment is not needed*

Load acting on the beam (A-B)

1- Dead load:

$$DL = \frac{4.575}{0.62} * (2 + 2) + 15 + (2.5 * 0.4) + (0.7 * 0.4 * 25) = 52.52 \text{ kn/m}$$

2- Live load

$$LL = \frac{1.86}{0.62} * (2 + 2) + (3 * 0.4) = 13.2 \text{ kn/m}$$

$$Wu = 1.4(52.52) + 1.7(13.2) = 95.968 \text{ kn/m}$$



Lu (m)	9m			9m			9m		
Ln(m)	8.6m			8.6m			8.6m		
Wu(kn/m)	95.968								
Cm	1/16	1/14	1/10	1/11	1/16	1/11	1/10	1/14	1/16
Mu (kn.m)		507	710				710	507	
d(mm)		615.5	615.5				615.5	615.5	
Rn(Mpa)		3.72	5.21				5.21	3.72	
P		0.00962	0.014				0.014	0.00962	
As(mm²)		2368.444	3446.8				3446.8	2368.444	
As_{min} (mm²)		821	821				821	821	
N		7.54~8	10.97~11				10.97~11	7.54~8	
b_{min} (mm)									
Fs =Fy		Ok	Ok				Ok	Ok	
TCL		Ok	Ok				Ok	Ok	
Use		8φ20	11φ20				11φ20	8φ20	
Cv			1.15				1.15		
Vu (kn)			474.6				474.6		
φVc (kn)			168.56				168.56		
Vs (kn)			408.1				408.1		
Av (mm²)			226.08				226.08		
S (mm)			143.21				143.21		
S_{max} (mm)			307.75				307.75		
Use			Φ12@140				Φ12@140		

sample of cacluations

$$L_u = 9m, \quad L_n = 9 - 0.2 - 0.2 = 8.6m$$

$$C_m = \frac{1}{10}, \quad M_u = \frac{1}{10} * 95.968 * 8.6^2 = 710 \text{ kn.m}$$

$$d = 600 - 40 - 12 - 20 - \frac{25}{2} = 615.5 \text{ mm (two layer)}$$

$$R_n = \frac{710/0.9}{400 * 615.5^2} * 10^6 = 5.21 \text{ Mpa}$$

$$m = \frac{420}{0.85 * 30} = 16.471, \quad \rho = \frac{1}{16.471} \left(1 - \sqrt{1 - \frac{2 * 5.21 * 16.471}{420}} \right) = 0.014$$

$$A_s = 0.014 * 400 * 615.5 = 3446.8 \text{ mm}^2,$$

$$A_{s \text{ min}} = \frac{1.4}{420} * 400 * 615.5 = 821 \text{ mm}^2$$

$$A_{s \text{ min}} = \frac{\sqrt{30}}{4 * 420} * 400 * 615 = 803 \text{ mm}^2$$

$$A_{s \text{ min}} = \max(821, 803) = 821 \text{ mm}^2$$

$$n = \frac{3151.4}{314} = 10.97 \approx 11$$

$$a = \frac{11 * 314 * 420}{0.85 * 30 * 400} = 142.22 \text{ mm}$$

$$\frac{a_b}{d} = 0.5 > \frac{a}{d} = 0.23 \quad \therefore f_s = f_y$$

$$\frac{a_{TCL}}{d_t} = 0.31875 > \frac{a}{d_t} = 0.222 \quad \therefore \text{Tension - controlled section}$$

for flexure use 11Ø20

$$C_v = 1.15, \quad V_u = \frac{1.15 * 95,968 * 8.6}{2} = 474.6 \text{ kn}$$

$$\phi V_c = 0.75 * \frac{\sqrt{30}}{6} * 400 * 615.5 * 10^{-3} = 168.56 \text{ kn}$$

$$V_s = \frac{474.6 - 168.56}{0.75} = 408.1 \text{ kn}$$

$$A_v = 2 * \pi \frac{12^2}{4} = 226.08 \text{mm}^2 \text{ (two leg)}$$

$$S = \frac{226.08 * 420 * 615.5}{408.1} * 10^{-3} = 142.21 \text{mm}$$

$$V_s < 2V_c$$

$$S_{max} = \min \left(300, \frac{615.5}{2} = 307.75 \right) = 300 \text{ mm}$$

use $\emptyset 12 @ 140 \text{mm}$

Axial load on column C:-

Beams in x- direction:

$$DL = \left\{ \left((0.4 * 0.7 * 25) + 15 \right) * \frac{4.4}{2} \right\} + \left\{ \left((0.4 * 0.7 * 25) + 15 \right) * \frac{4.4}{2} \right\} = 96.8 \text{ kn}$$

$$LL = 0 + 0 = 0 \text{ kn}$$

Beams in y- direction:

$$DL = \left\{ \left(\frac{4.575}{0.62} * (2 + 2) + 15 + (2.5 * 0.4) + (0.7 * 0.4 * 25) \right) * \frac{9}{2} \right\} \\ + \left\{ \left(\frac{4.575}{0.62} * (2 + 2) + 15 + (2.5 * 0.4) + (0.7 * 0.4 * 25) \right) * \frac{9}{2} \right\} = 472.68 \text{ kn}$$

$$LL = \left\{ \left(\frac{1.86}{0.62} * (2 + 2) + (3 * 0.4) \right) * \frac{9}{2} \right\} + \left\{ \left(\frac{1.86}{0.62} * (2 + 2) + (3 * 0.4) \right) * \frac{9}{2} \right\} = 118.8 \text{ kn}$$

$$Wu = 1.4(96.8 + 472.68) + 1.7(118.8) = 999.232 \text{ kn}$$