Design of Reinforced Concrete Structures (II)

Discussion

Eng. Mohammed R. Kuheil

Review

The thickness of one-way ribbed slabs

After finding the value of total load (Dead and live loads), the elements are designed. Based on the mechanism of load transfer, the ribs are the first elements to take the load applied. The design of it is based on three requirements that must be fulfilled (deflection, shear, and flexure).
To fulfill deflection requirement, the following table that shows minimum thicknesses for ribs and beams is used. When choosing this thickness, the deflection requirement is accomplished.

Cases	Simply Supported	One End Continuous	Both End Continuous	Cantilever
Min. required thickness	L/16	L/18.5	L/21	L/8

L: is the span length in the direction of bending form center to center of support (In ribs, the support is beam).

Hint: when the case is cantilever the length of span taken from the face of the support to the end of span.

Loads

1. Dead load

Dead load in buildings is include own weight, covering materials, and equivalent partition load and external walls.

2. Live load

To find the live load applied on a building must be refer to the general codes (IBC, UBC and ASCE 7-10).

The value of live load varies according to the usage of the building, According ASCE 7-10, Ch. (4):

For regular residential building $= 200\ \mathrm{kg/m^2} = 0.20\ \mathrm{t/m^2}$.

For dance halls and ballrooms $= 490\ \mathrm{kg/m^2}$

3. Special loads

This loads are include seismic forces, wind load, and other dynamic loads. Our concern in this course are to find live and calculate the dead loads only and the special loads are used in advanced courses.

The dead load details:

1. Own weight

Total own weight = Block weight + concrete weight

Block weight = (the thickness of block in cm) kg

Example: when the thickness of block is equal 20 cm $\rightarrow$ the weight of block is equal 20 kg.

Concrete weight = the volume of concrete $\times\ \Upsilon_c$

Υ_c = the unit weight of reinforcement concrete

Hint: 1. the unit weight of plain conctete (with out reinforcement) = 2.4 t/m^3

2. the unit weight of reinforcement conctete = 2.5 t/m^3

The volume of concrete = the total volume of the representative sample − the volume block

The total volume of the representative sample =

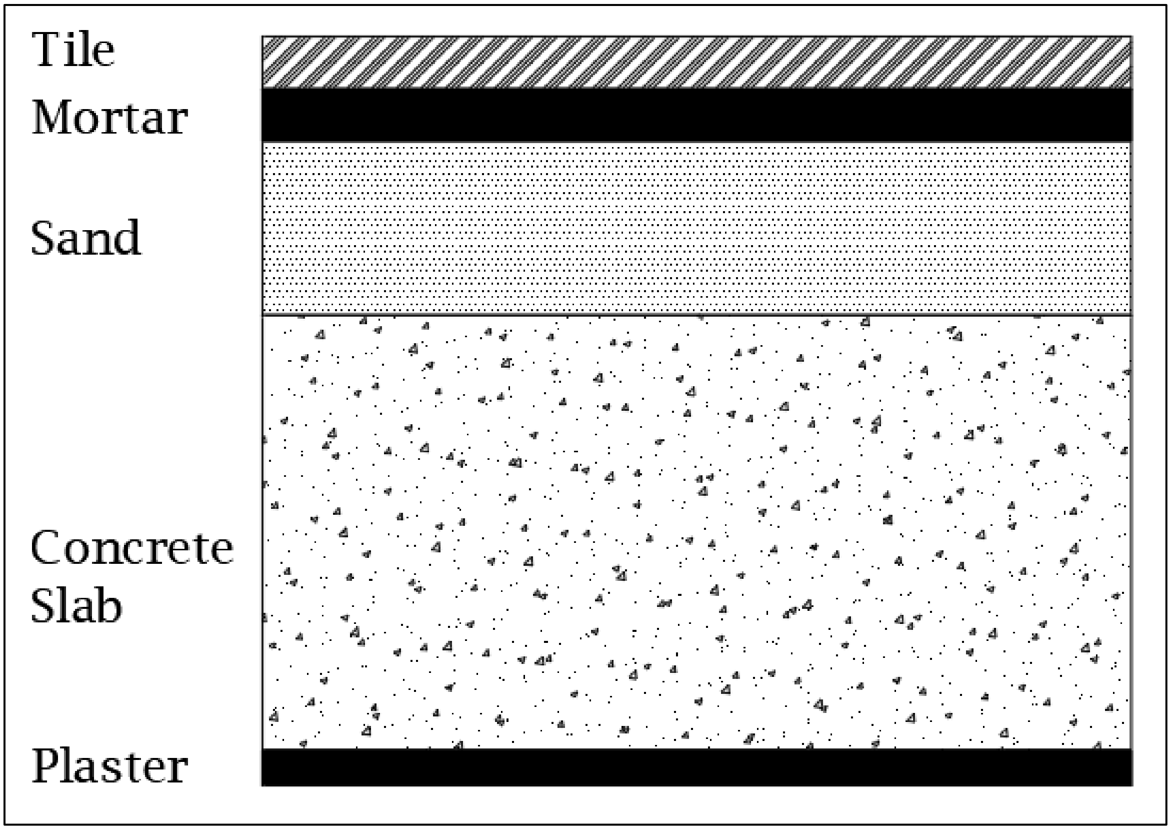
(the length of block + the width of rib) × (block width) × total thickness

$$\text{the total own weight per unit area } = \frac{\text{Conrete weight (ton)} + \text{Block weight (ton)}}{\text{Area of the representative sample}}\ \ t/m^2$$

2. Covering Materials

Material	Thickness (cm)	Unit Weight (t/m^3)
Sand	10	1.7
Mortar	3	2.2
Tile	2.5	2.5
Plaster	2	2.2

The total weight of covering material $= \sum(\text{Thickness (h)} \times \text{unit weight})$



Covering Materials

3. Equivalent partition load (EPL) (internal walls)

$1\ \text{m}^2$ from the block wall $= \dfrac{1}{0.4 \times 0.2} = 12.50$ Blocks.

The weight of $1\ \text{m}^2 = 12.5 \times$ the weight of 1 block

The weight of plaster $\text{t/m}^2 =$ thickness (2 faces) $\times$ unit weight

The total weight of EPL every 1m^2 from the wall $=$ weight of plaster $+$ weight of block

The total weight of EPL (ton) $=$ (total weight / m^2) $\times$ height of story $\times$ total length of EPL

$$\text{The total weight of EPL } (\text{t/m}^2) = \frac{\text{the total weight of EPL (ton)}}{\text{net area of slab}}$$

Hint: The net area of slab $=$ total area of slab $-$ all open areas

4. External walls

The calculation of the external wall is the same of the internal wall but the external wall is carry directly on the exterior beams and the internal wall carried on the slab.

$1\ \mathrm{m}^2$ from the block wall $= \dfrac{1}{0.4 \times 0.2} = 12.50$ Blocks.

the weight of $1\ \mathrm{m}^2 = 12.5 \times$ weight of 1 block

the weight of plaster $\mathrm{t/m}^2 =$ thickness (2 faces) $\times$ unit weight

the total weight of EPL every $1\mathrm{m}^2$ from the wall $=$ weight of plaster $+$ weight of block

the total weight of EPL $(\mathrm{t/m'}) =$ (total weight m^2) $\times$ height of story

Load combinations:

Dead and live loads (DL+LL):

1.4 D

1.2 D + 1.6 L

Dead (D), live (L) and wind (W):

1.2 D + 1.0 L

1.2 D + 0.8 W

1.2 D + 1.6 W + 1.6 L

0.9 D + 1.6 W

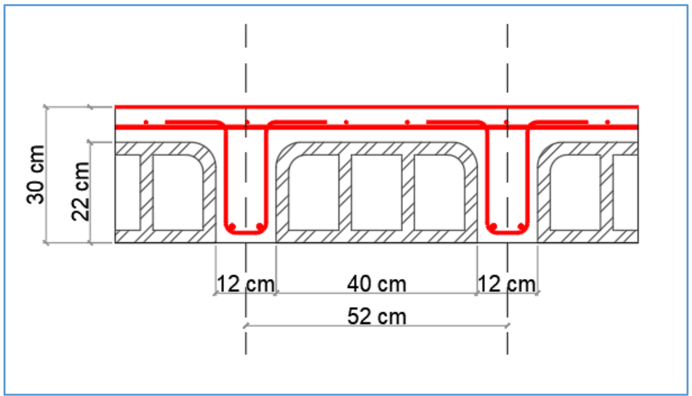
Dead (D), live (L) and Earthquake (E):

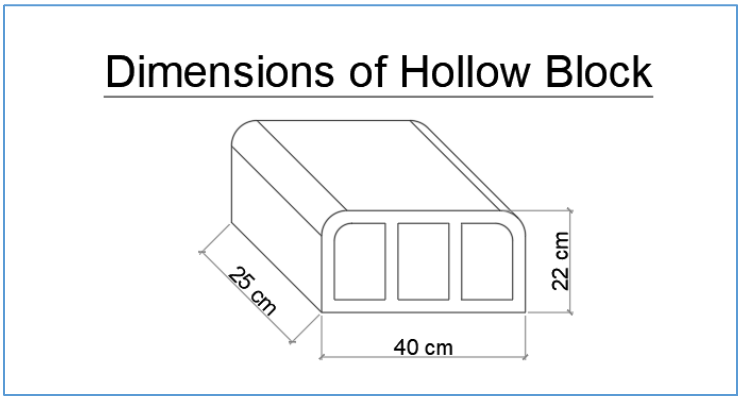
1.2 D + 1.0 L + 1.0 E

0.9 D + 1.0 E

Example:

Calculate the factored load (dead and live loads) per unit area for residential building.





Given:

Total area of the floor $= 250\ m^2$

Total thickness of slab = 30 cm

Topping slab = 8 cm

The width of rib = 12 cm

The total length of EPL = 60 m'

The total length of exterior walls $= 65\ m^2$

The thickness of EPL = 12 cm

The thickness of plaster = 1.5 cm for all elements, but in the exterior face is equal 2 cm

The thickness of sand = 13 cm

The mortar thickness = 3 cm

The thickness of plaster = 2 cm

The thickness of tile = 2.5 cm

Hint:

The area of stair $= 13\ \mathrm{m}^2$

Another open area $= 15\ \mathrm{m}^2$

Solution:

1. Dead load:

Own weight

$V_T = (0.12 + 0.40) \times 0.25 \times 0.30 = 0.039\ \mathrm{m}^3$

$V_B = (0.22 \times 0.25 \times 0.40) = 0.022\ \mathrm{m}^3$

$V_C = (0.039 - 0.022) = 0.017\ \mathrm{m}^3$

$W_C = 0.017 \times 2.5 = 0.0425\ \mathrm{ton}$

$W_B = 0.022\ \mathrm{ton}$

$W_T = 0.0425 + 0.022 = 0.0645\ \mathrm{ton}$

$$w_T(\text{per unit area}) = \frac{W_C + W_B}{\text{Area of the representative sample}}$$

$$\text{the weigth of total sample} = \frac{0.0425 + 0.022}{0.52 \times 0.25} = 0.50\ \mathrm{t/m^2}$$

Equivalent partition load

$\text{the total weight of EPL}\ /\mathrm{m}^2 \text{from the wall} = (12.5 \times 0.012) + (0.04 \times 2.2) = 0.238\ \mathrm{ton}$

$\text{the total weight of EPL(ton)} = 0.238 \times 3.00 \times 60 = 42.84\ \mathrm{ton}$

$$\text{the total weight of EPL}\ (\mathrm{t/m^2}) = \frac{42.84}{250 - 13 - 15} = 0.193\ \mathrm{t/m^2}$$

Covering Materials

The total weight of covering materials $= \sum(\text{Thickness (h)} \times \text{unit weight})$

The total weight of covering materials $= \sum(0.13 \times 1.7 + 0.03 \times 2.2 + 0.02 \times 2.2 + 0.025 \times 2.5)$

The total weight of covering materials $= 0.394\ \text{t/m}^2$

The total service dead load = own weight + EPL + covering materials

The total service dead load $= 0.50 + 0.193 + 0.35 = 1.043\ \text{t/m}^2$.

2. Live load

From the ASCE 7-10 Chapter 4 L.L. for regural residential building $= 200\ \text{kg/m}^2 = 0.20\ \text{t/m}^2$

Factored load $= \max(1.4 \times \text{DL or } 1.2 \times \text{DL} + 1.6 \times \text{LL})$

$\text{U}\ (\text{t/m}^2) = \max(1.4 \times 1.043 \text{ or } 1.2 \times 1.043 + 1.6 \times 0.2)$

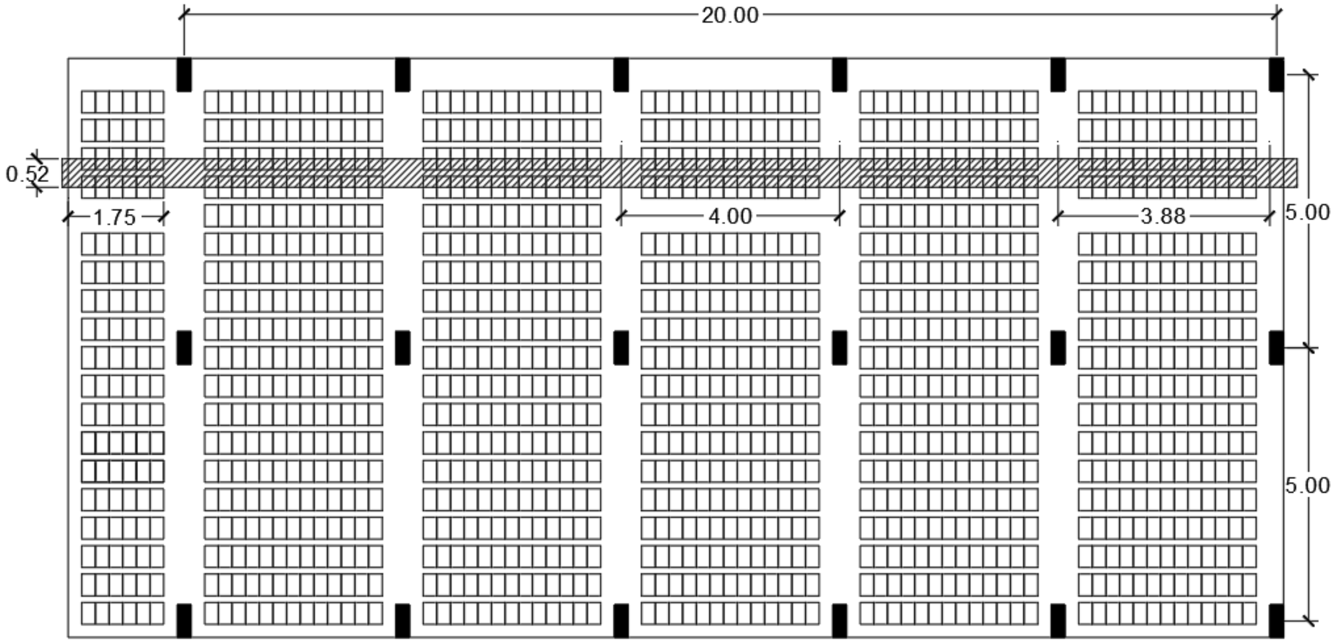
$\text{U}\ (\text{t/m}^2) = \max(1.46 \text{ or } 1.57)$

$\text{U}\ (\text{t/m}^2) = 1.57\ \text{t/m}^2$.

Moment and Shear design:

According the following example.

Example: For the one way ribbed slab shown in figure below, design any of the typical ribs and main interior beam.



Solution: From deflection control

The thickness of slab ($h_{min.}$) $= \max.\left(\frac{3.88}{18}, \frac{4}{21}, \frac{1.75}{8}\right) = 0.219\ \text{m} = 22\ \text{cm}$

Used Hollow block $25 \times 40 \times 17$ cm.

Topping slab thickness $= 22 - 17 = 5$ cm.

The topping slab is designed as a continuous beam supported by the ribs. Due to the large number of supporting ribs, the maximum bending moment is taken as $M_u = W_u l_c^2/12$

Assume $DL = 0.85\ t/m^2$, $LL = 0.20\ t/m^2$ $f'c = 200\ kg/cm^2$ $fy = 4200\ kg/cm^2$

$W_u = 1.2 \times 0.85 + 1.6 \times 0.2 = 1.34\ t/m^2$

For a strip 1 m width $W_u = 1.34\ t/m'$

$$M_u = \frac{1.34 \times 0.4^2}{12} = 0.018\ t.m$$

$$t = \sqrt{\frac{3 \times M_u}{\phi b \sqrt{f'c}}}$$

$$t = \sqrt{\frac{3 \times 0.018 \times 10^5}{0.9 \times 100 \times \sqrt{200}}} = 2.06 \text{ cm} < 5 \text{ cm}$$

But the t is not to be less than 1/12 the clear distance between ribs, nor less than 5.00 cm

$$t = \max.\left(2.06, \frac{1 \times 40}{12}, 5\right) = 5 \text{ cm.}$$

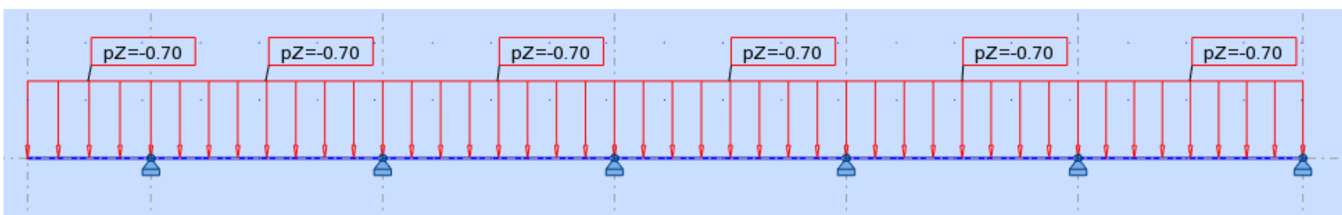
Area of shrinkage reinforcement $A_s = 0.0018 \times b \times h = 0.0018 \times 100 \times 5 = 0.9 \text{ cm}^2/\text{m}$

Use 4 ϕ 6 mm or ϕ 8 mm @ 50 cm in both direction.

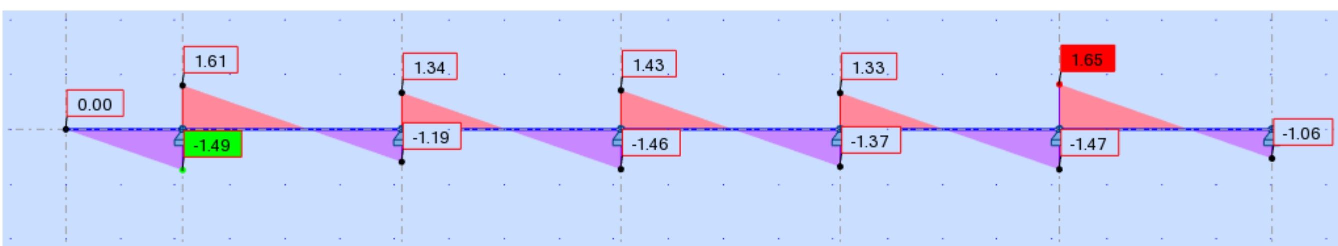
Now, we must be check for shear and bending moment (Using ROPOT structural analysis software)

To find the load per meter length, take a strip (shown in the figure above).

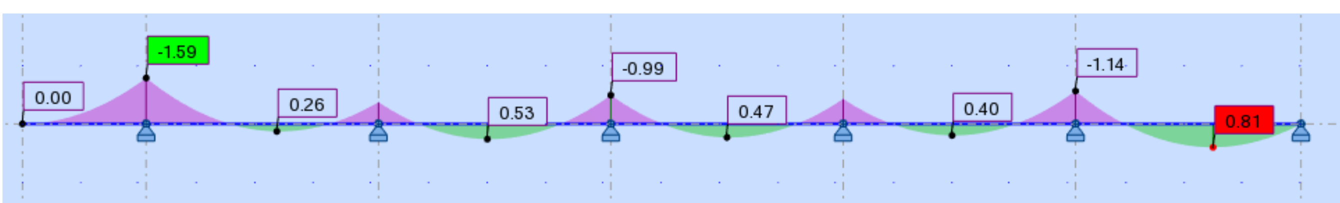
Factored load $= (1.2 \times 0.85 + 1.6 \times 0.20) \times 0.52$ (width of strip) $= 0.70$ t/m′



The result from ROPOT structural analysis software



S.F.D.



B.M.D.

Shear design:

$V_{u\ (max.)} = 1.65$ ton.

Now, calculate the capacity for rib for shear ϕV_c

Resistance force(ϕV_c) must be greater than applied force $V_{u\ (max.)}$

$\phi V_c = \phi \times 0.53 \times \sqrt{f'c} \times b \times d$

$d = \text{thickness of slab (h)} - \text{cover} - \text{stirrup} - 0.5 \times d_b$

$d = 22 - 2 - 0.6 - 0.6$ assume ϕ 12 mm reinforcing bars and ϕ 6 mm stirrups

$d = 18.80$ cm

$$\phi V_c = 0.75 \times 0.53 \times \sqrt{200} \times 12 \times \frac{18.80}{1000} = 1.27 \text{ ton.}$$

Shear strength provided by rib concrete ϕV_c may be taken 10 % greater than those for beams.

It is permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

$1.1 \times \phi V_c = 1.1 \times 1.27 = 1.4$ ton.

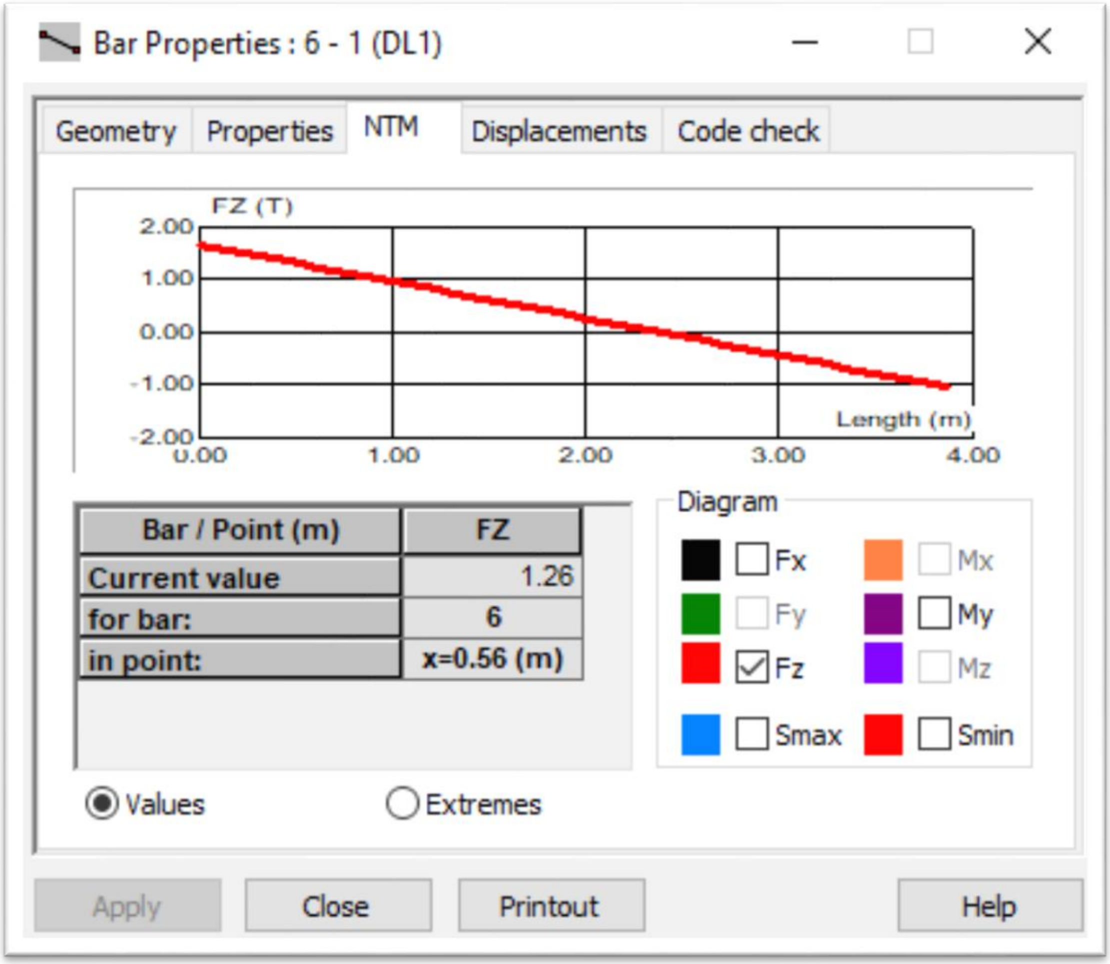
Use 4 ϕ 6 mm U-stirrups per meter run are to be used to carry the bottom flexural reinforcement.

Since critical shear section can be taken at distance d from faces of support (beam).

the previous of $V_{u\ (max.)}$ (1.65 ton) form the center of the support.

so we will take the distance $= 0.5 \times \text{beam width} + d = 0.5 \times 75 + 19.40 = 56.9$ cm.

so the critical shear section will be at a distance 56.9 cm from the center of support (beam).



From the previous figure:

V_u(critical shear section) $= 1.26$ ton $< 1.1\phi V_c$ (1.4 ton) $\rightarrow\rightarrow$ OK

The rib shear resistance is adequate.

When $V_u > \phi V_c$???.

We have six choices:

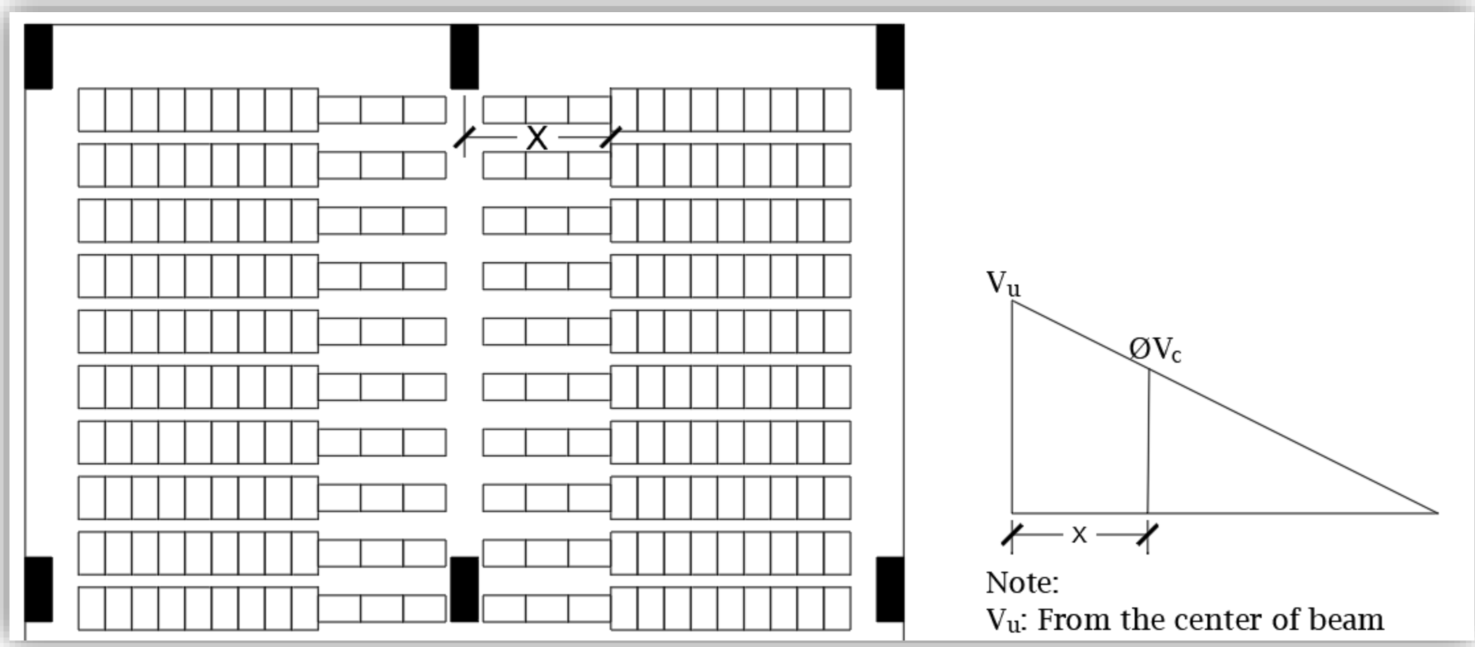
1. Increase the compressive strength of concrete $f'c$.

2. Increase the width of rib.

3. Increase the depth of the slab, which is uneconomic choice.

4. Can be used stirrups as a shear reinforcement to resist the applied force.

5. Change the direction of blocks at maximum shear area so that the width of rib is increased. the figure below describes this solution.



6. Enlarge the beam width.

2. Moment design

$M_{u\,(max.)} = 1.59\ t/m^2.$

###	A	Mid Span AB	B	Mid Span BC	C	Mid Span CD	D	Mid Span DE	E	Mid Span EF	F
Moment	**1.59**	**0.26**	**0.8**	**0.53**	**1**	**0.47**	**1**	**0.40**	**1.14**	**0.81**	**0.00**
Ro	**0.01172**	**0.00403**	**0.00546**	**0.00354**	**0.00694**	**0.00403**	**0.00694**	**0.00403**	**0.008**	**0.00553**	**0.00403**
Check	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Ro used	**0.01172**	**0.00403**	**0.00546**	**0.00354**	**0.00694**	**0.00403**	**0.00694**	**0.00403**	**0.008**	**0.00553**	**0.00403**
As	**2.602**	**0.89518**	**1.21216**	**0.78661**	**1.54001**	**0.89518**	**1.54001**	**0.89518**	**1.7766**	**1.22829**	**0.89518**
Db (mm)	**14**	**10**	**10**	**10**	**10**	**10**	**10**	**10**	**12**	**10**	**10**
d (cm)	**18.5**	**18.7**	**18.7**	**18.7**	**18.7**	**18.7**	**18.7**	**18.7**	**18.6**	**18.7**	**18.7**
No. Bars	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**
Sc	**3.6**	**4.4**	**4.4**	**4.4**	**4.4**	**4.4**	**4.4**	**4.4**	**4**	**4.4**	**4.4**
Check Sc	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

$$\rho = \frac{0.85\, f'c}{fy}\left[1 - \sqrt{1 - \frac{2.353 \times 10^5 \times M_u}{0.9 \times b_w \times d^2 \times f'c}}\right]$$

$$\rho_{min.} = \max.\left(\frac{0.80 \times f'c}{fy}, \frac{14}{fy}\right) = \max.\left(\frac{0.80 \times 200}{4200}, \frac{14}{4200}\right) = 0.0033$$

$$\rho_{max.} = \frac{0.31875 \times 0.85 \times f'c}{fy} = \frac{0.31875 \times 0.85 \times 200}{4200} = 0.0129$$

$$\rho = \frac{0.85\,(200)}{4200}\left[1 - \sqrt{1 - \frac{2.353 \times 10^5 \times 1.59}{0.9 \times 12 \times 18.80 \times 200}}\right] = 0.0117$$

$\rho_{min.} < \rho < \rho_{max.} \rightarrow\rightarrow$ OK $\rho_{used} = 0.0117$

$d = 22 - 2 - 0.6 - 0.6 = 18.80\ cm$

$A_s = \rho_{used} \times b \times d = 0.0117 \times 12 \times 18.80 = 2.64\ cm^2$

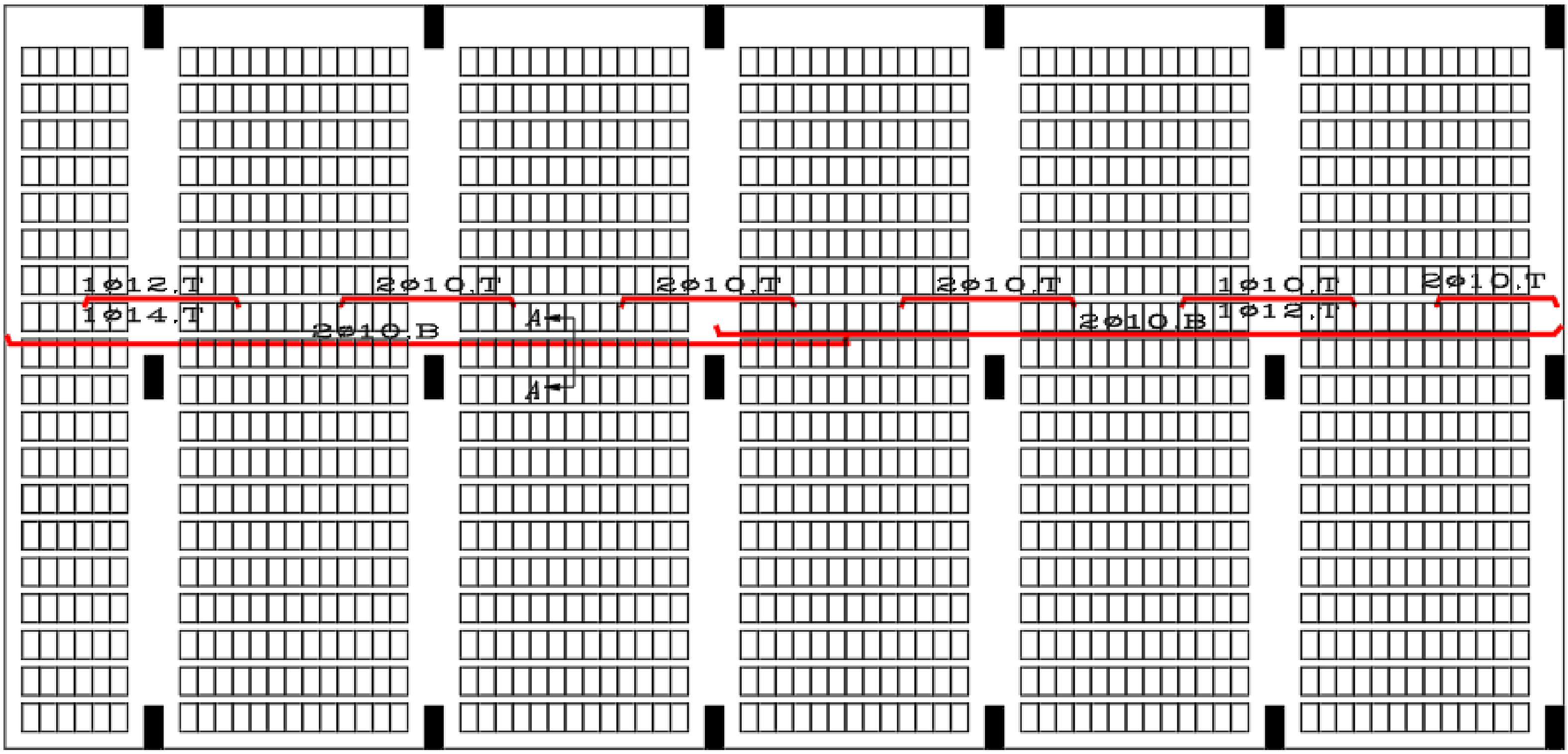
try ϕ 12 mm

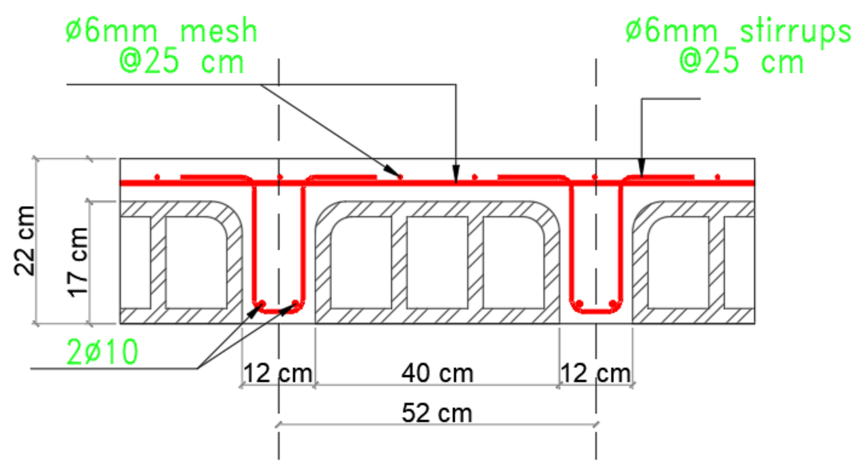
$$\#\text{ of bars} = \frac{A_s}{\text{Area of one bar}} = \frac{2.64}{0.7854 \times {d_b}^2} = \frac{2.64}{0.7854 \times 1.2^2} = 2.33$$

try ϕ 14 mm

$$\#\text{ of bars} = \frac{A_s}{\text{Area of one bar}} = \frac{2.64}{0.7854 \times {d_b}^2} = \frac{2.64}{0.7854 \times 1.4^2} = 1.71$$

use 1ϕ12 mm + 1ϕ14 mm.


1ø12.T
1ø14.T
2ø10.T
2ø10.B
A
A
2ø10.T
2ø10.T
2ø10.B
1ø10.T
1ø12.T
2ø10.T


ø6mm mesh
@25 cm
ø6mm stirrups
@25 cm
22 cm
17 cm
2ø10
12 cm
40 cm
12 cm
52 cm

Section A-A

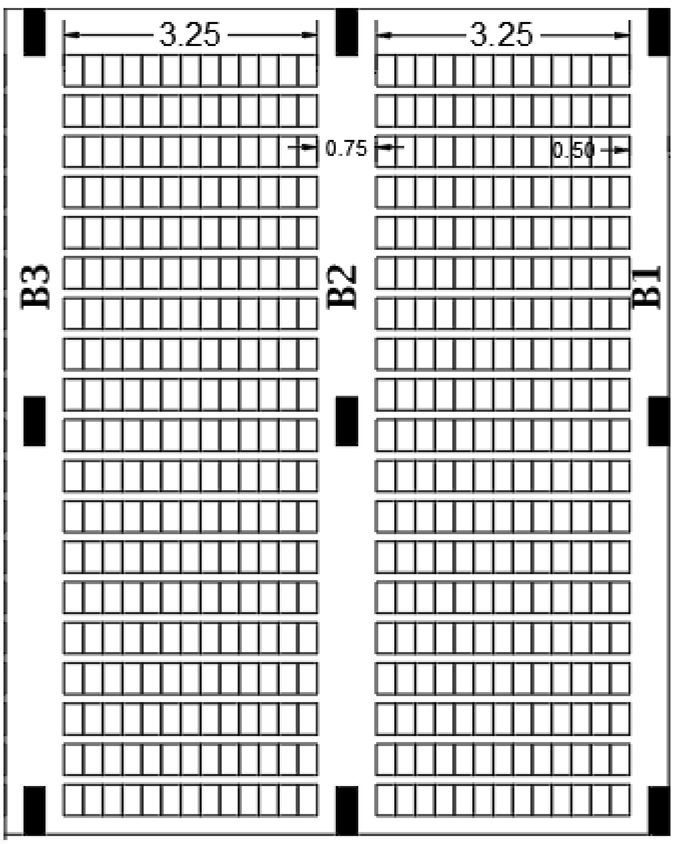
Beam design

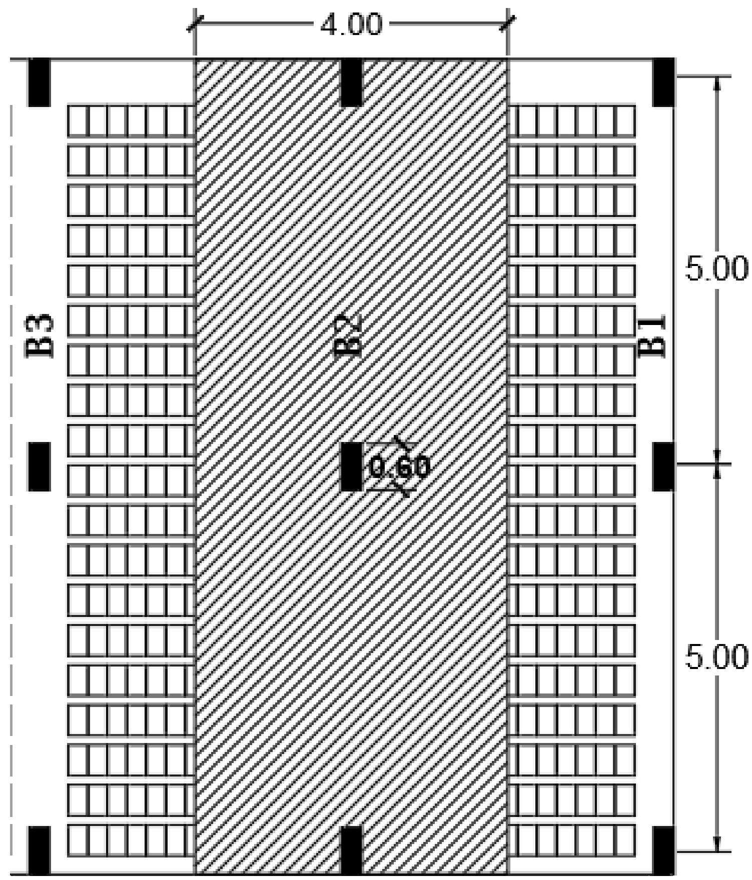
Design the continuous beam (B2) shown in the figure below.

Use $f'c = 200\ kg/cm^2$ and $fy = 4200\ kg/cm^2$.

$DL = 1.00\ t/m^2$ and $LL = 0.20\ t/m^2$.

Hint: the thickness of slab is equal 25 cm.





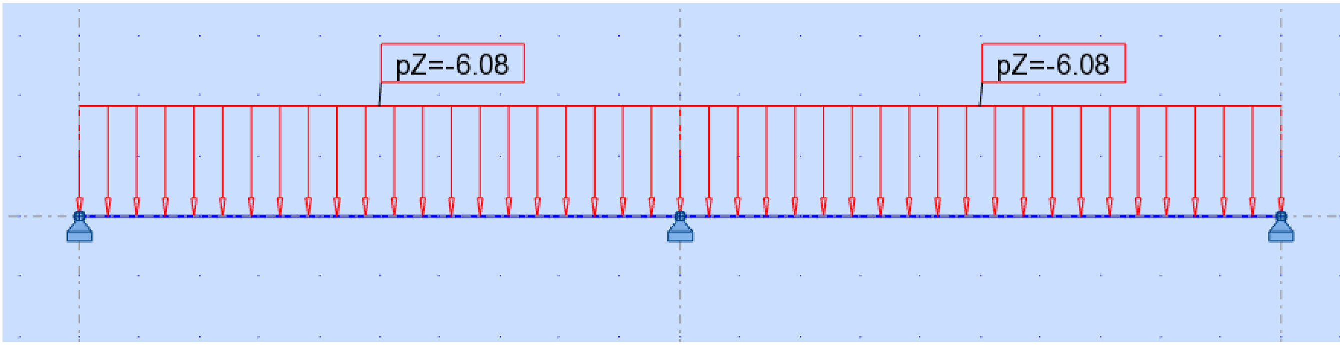
Solution:

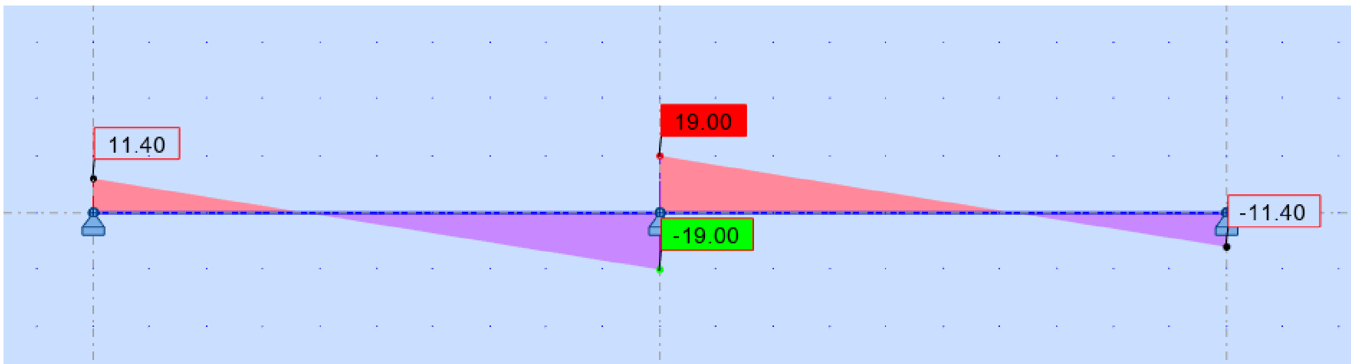
$W_u(t/m^2) = 1.2 \times 1.00 + 1.6 \times 0.20 = 1.52\ t/m^2.$

$W_u(t/m') = W_u(t/m^2) \times \text{Hatch Area} = 1.52 \times 4 = 6.08\ t/m'.$

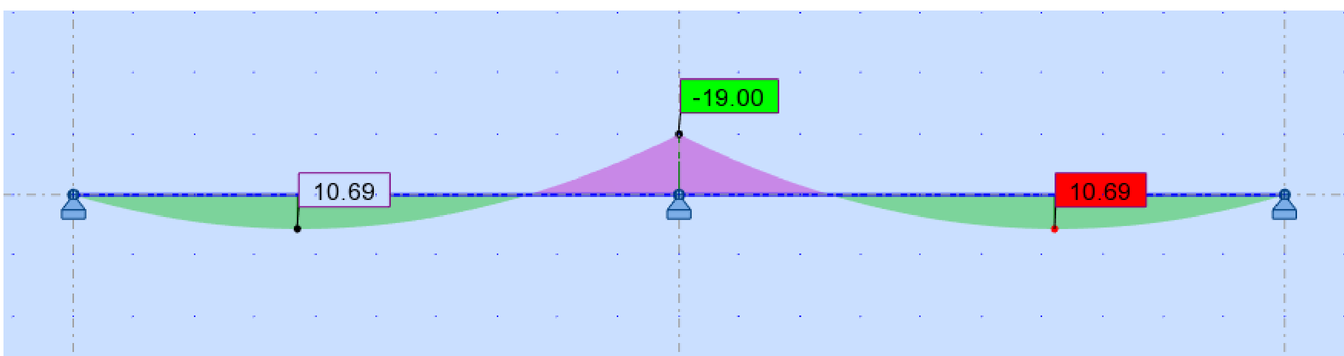
$$\text{The width of Hatch Area} = \frac{3.25}{2} + \frac{3.25}{2} + 0.75 = 4\ m$$

By using Autodesk ROPOR structural analysis software





S.F.D.



B.M.D.

$V_{u\ (max.)} = 19.00$ ton. V_u(at a critical section "d from the face of column") $= 15.4$ ton.

$M_{u\ (max.)}(+ve) = 10.69\ t/m^2.$

$M_{u\ (max.)}(-ve) = 19.00\ t/m^2.$

2. Shear design

Hidden beam

$$\phi V_c = \phi \times 0.53 \times \sqrt{f'c} \times b \times d$$

$d = 25 - 4 - 0.8 - 0.7 = 19.50$ cm. assume $d_b = 14$mm and $d_{stirrup} = 8$mm

$$\phi V_c = 0.75 \times 0.53 \times \sqrt{200} \times 75 \times \frac{19.50}{1000} = 8.22 \text{ ton}$$

$$\frac{V_u}{\phi} = V_C + V_s$$

$$V_s = \frac{V_u - \phi V_C}{\phi} = \frac{15.40 - 8.22}{0.75} = 9.6 \text{ ton.}$$

Check for ductility $\rightarrow$ Assume $d_b = \phi 14$ mm & $d_{stirrups} = \phi 8$ mm .

$$2.2\sqrt{f'c}\ b_w d = 2.2 \times \sqrt{200} \times 75 \times \frac{19.50}{1000} = 45.5 \text{ ton} \gg V_s \text{ (9.6 ton).}$$

The dimensions of the cross section are adequate for ensuring a ductile mode of failure.

Shear zones:

Zone (A)

$$V_u \leq \frac{\phi V}{2}$$

No shear reinforcement is required, but it is recommended to use minimum area of shear reinforcement

Trying two − legged $\phi 8$ mm vertical stirrups

$$\left(\frac{A_v}{S}\right)_{min.} = \max.\left(\frac{0.20 \times \sqrt{f'c} \times b_w}{f_y}, \frac{3.5 \times b_w}{f_y}\right) = \left(\frac{0.20 \times \sqrt{200} \times 75}{4200}, \frac{3.5 \times 75}{4200}\right) = 0.0625 \text{cm}^2/\text{cm}$$

$$S_{min.} = \frac{A_{v\ (min.)}}{0.0625} = \frac{\#\text{ of legges} \times \text{area of cross section for one leg}}{0.0625} = \frac{2 \times \left(\frac{\pi}{4} \times 0.8^2\right)}{0.0625} = 16 \text{ cm}$$

Hint:

1. When $V_s \leq 1.1\sqrt{f'c} \times b_w \times d$ the maximum stirrup spacing is not to exceed the smaller of d/2 or 60 cm. $S_{min.} = \max.\left(\frac{d}{2}, 60\ cm\right)$

2. When $2.2 \times \sqrt{f'c} \times b_w \times d > V_s > 1.1\sqrt{f'c} \times b_w \times d$ the maximum stirrup spacing is not to exceed the smaller of d/4 or 30 cm. $S_{min.} = \max.\left(\frac{d}{4}, 30\ cm\right)$

3. When $V_s \geq 2.2 \times \sqrt{f'c} \times b_w \times d$ must be enlarge the section dimensions.

$V_s = 9.6$ ton.

$$\sqrt{f'c} \times b_w \times d = \sqrt{200} \times 75 \times \frac{19.5}{1000} = 20.68\ ton > V_s \rightarrow S_{min.} = \max.\left(\frac{19.5}{2}, 60\ cm\right) = 9.75\ cm.$$

$$S_{used} = \min.\left(\frac{d}{2}, 60cm, S_{min.}\right) = (\ 9.75\ cm, 60\ cm, 16\ cm) \cong 10\ cm.$$

Note

When the thickness of slab is small, (d) the spacing between shear stirrups are be very small and we should be enlarge the depth of beam (drop beam).

Zone (B)

$\phi V_c > V_u > \frac{\phi V}{2}$, minimum shear reinforcement is required.

Similar to zone A

Zone (C)

$V_u \geq \phi V_c$, shear reinforcement is required.

$$\frac{V_u}{\phi} = V_C + V_s$$

$$\left(\frac{A_v}{S}\right)_{Calculated.} = \frac{V_s}{f_y \times d} = \frac{14.4 \times 1000}{4200 \times 19.5} = 0.176\ cm^2/cm$$

$$\left(\frac{A_v}{S}\right)_{Calculated.} > \left(\frac{A_v}{S}\right)_{min.} \rightarrow OK$$

$$V_s = \frac{V_u - \phi V_C}{\phi} = \frac{19.00 - 8.22}{0.75} = 14.4 \text{ ton.}$$

$$S = \frac{A_v \times f_y \times d}{V_s} = \frac{2 \times (0.5) \times 4200 \times 19.5}{9.6 \times 1000} = 8.53 \text{ cm}$$

$$S_{used} = \min.\left(\frac{d}{2}, 60\text{cm}, S_{min.}, S\right) = (\ 9.75 \text{ cm}, 60 \text{ cm}, 16 \text{ cm}, 8.5) \cong 8 \text{ cm (NOT practical).}$$

Note:

The spacing between the stirrups is small because the criteria of $S = d/2$ in minimum zones and in maximum shear zone (C) because the V_s is high.

To avoid these problems in shear should be enlarge the depth of section (Drop beams) to overcome the criteria $S = d/2$.

Design of columns

Classification of columns according to support type:

Columns are classified according to the type of support into two categories:

1. Pin supported columns

In this type of columns, the supports don't resist moment and so do the columns. This type of columns resists only axial load (Design 1).

2. Moment resisting columns

This type of columns resists both axial load and bending moment, and the design of this type of columns depends on the interaction between both forces.

The type of column is depends on the details of steel reinforcement at the joints of columns.

Classification of Frames (Sway and Non-Sway Frames):

Moment resisting frames are classified into two types according to lateral movement probability:

1- Sway frames: at which the lateral movement is permitted and possible (figure 1). This happens because the frame is not restrained against this movement.

2- Non-sway frames: at which the lateral movement is restrained either from another frame or from the secondary beams in the third direction (figure 2)

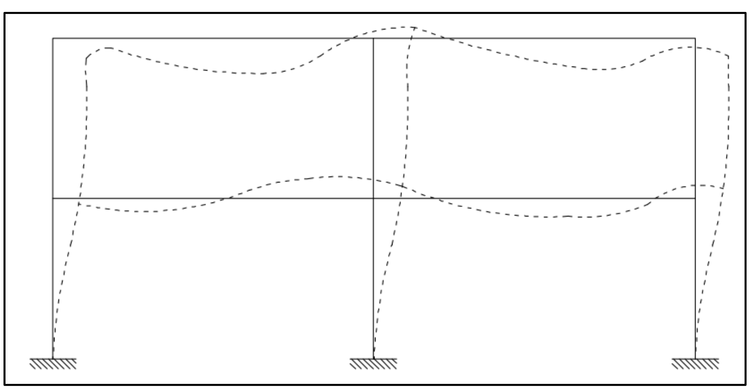


Figure 1: Sway Frame

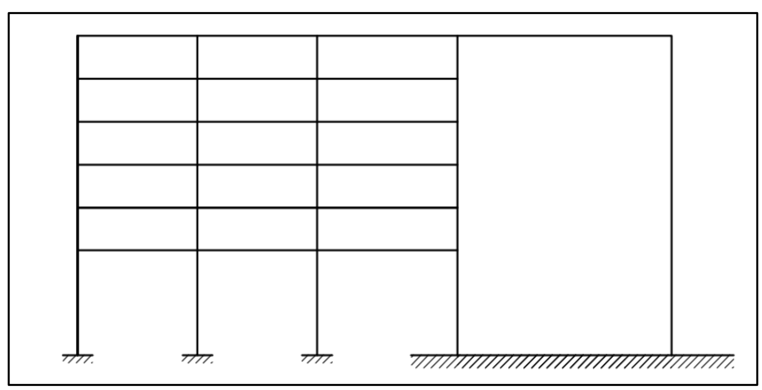


Figure 2: Non-Sway Frame

According ACI code, the frame is classified as a non-sway frame if:

$$Q = \frac{\sum P_u \Delta_\circ}{V_{us} l_c} \leq 0.05$$

Q is the stability index, which is the ratio of secondary moment due to lateral displacement and primary moment

$\sum P_u$ is the total factored vertical load in the story.

$\sum V_{us}$ is the factored horizontal story shear.

l_c is length of column measured center − to − center of the joints in the frame

$\Delta_\circ$ is the first − order relative deflection between the top and bottom of that story due to V_{us}

Hint: when we use the previous method to know the type of frame (sway or non-sway), we must be take all columns in the same story.

Classification of columns (Short and Long Columns):

Columns are classified according to design requirements into two main categories, short and long. This classification is depend on three factors, which are:

1. Length of column.

2. Cross section of column.

3. Type of support (Effective length factor).

The three factors are put together and called slenderness ratio $= \frac{KL_u}{r}$

L_u: a clear distance between floor slabs, beams, or other members capable of proiding lateral supprt.

r: radius of gyration associated with axis about which bending occurs.

For rectangular cross sections $r = 0.30\ h$, and for circular sections, $r = 0.25\ h$

$h =$ column dimension in the direction of bending.

Procedures for Classifying Short and Slender Columns

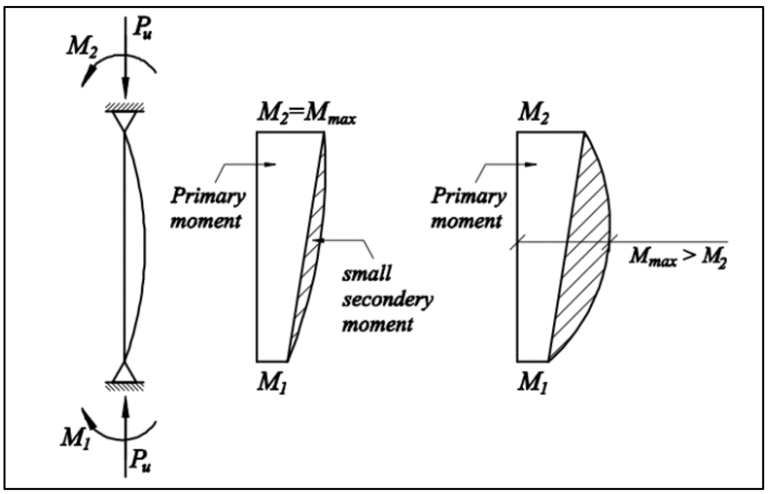
For non-sway frame

If $\frac{KL_u}{r} \leq 34 - 12\frac{M_1}{M_2} \leq 40 \rightarrow$ the column is short.

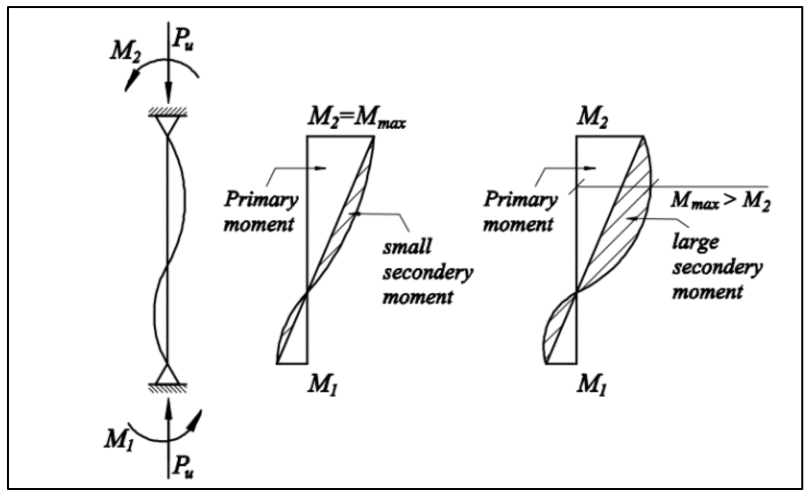
If $\frac{KL_u}{r} \geq 34 - 12\frac{M_1}{M_2} \rightarrow$ the column is long.

$M_1 =$ smaller factored end moment on column, positive if member is bent single curvature, negative if bent in double curvature.

$M_2 =$ larger factored end moment on column, always positive.



Single Curvature



Double Curvature

For sway frame

$\frac{KL_u}{r} \leq 22 \rightarrow$ the column is short.

$\frac{KL_u}{r} > 22 \rightarrow$ the column is long.

Effective length factor (K)

To estimate the effective length factor k for a column of constant cross section in a multibay frame must be used the Jackson and Moreland Alignment Charts.

Two methods to calculate the effective length factor (k):

1. from the charts.

2. from equations.

For Non-Sway frames

$$k_1 = 0.70 + 0.05(\Psi_A + \Psi_B) \leq 1.0$$

$$k_2 = 0.85 + 0.05(\Psi_{min.}) \leq 1.0$$

$$\Psi_{min.} = \min(\Psi_A, \Psi_B)$$

$$k_{used} = \min.(k_1, k_2)$$

For Sway frames

for $\Psi_m < 2.0$

$$\Psi_m = \frac{\Psi_A + \Psi_B}{2}$$

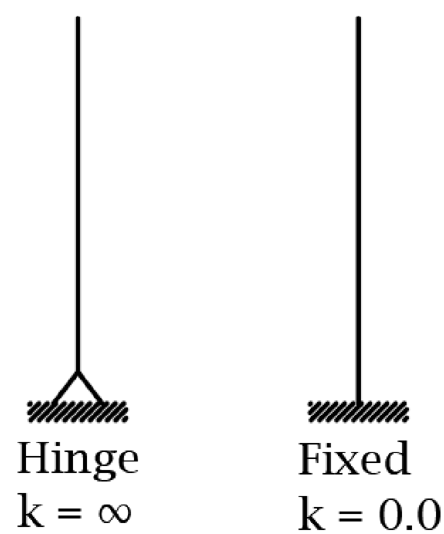
$$k = \frac{20 - \Psi_m}{20}\sqrt{1 + \Psi_m}$$

for $\Psi_m \geq 2.0$

$$k = 0.9\sqrt{1 + \Psi_m}$$

For sway frames hinged at one end, k is taken by:

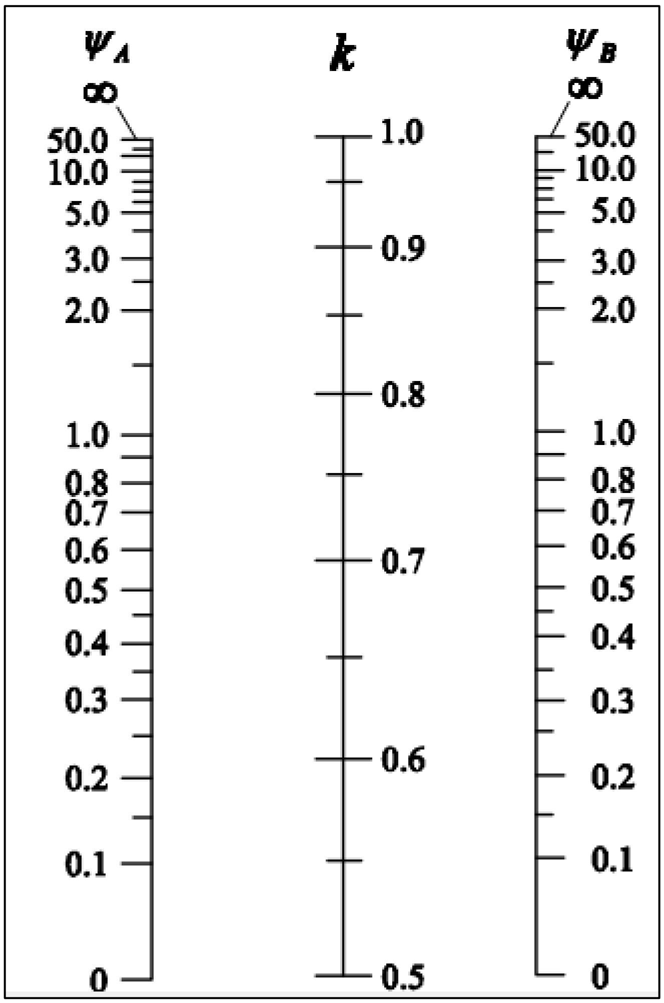
$$k = 2.0 + 0.3\Psi$$



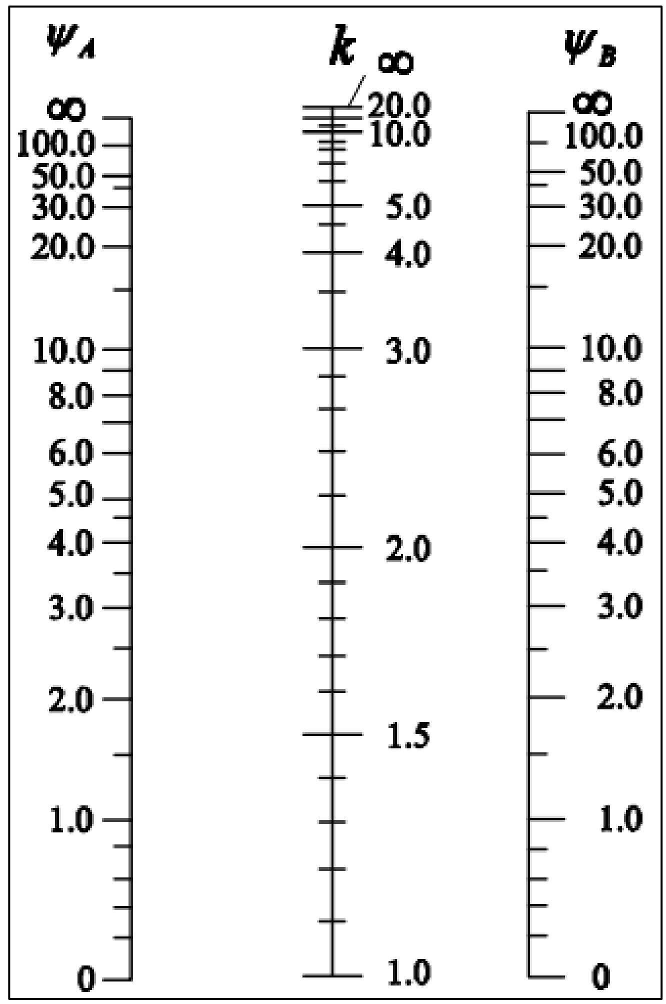
Note:

The maximum value of the effective length factor (k) is equal **1** for Non-Sway frame, and we can take this value without using the Chart (the worst case).

However, for Sway Frames the value of the effective length factor (k) must be obtained from the charts below or the previous equations.



for non-sway frame



for sway frame

The effective length factor k is a function of the relative stiffness at each end of the column. In these charts, k is determined as the intersection of a line joining the values of Ψ at the two ends of the column. The relative stiffness of the beams and columns at each end of the column Ψ is given by the following equation:

$$\Psi = \frac{\sum E_c I_c / l_c}{\sum E_b I_b / l_b}$$

l_c = length of column center-to-center of the joints

l_b = length of beam center-to-center of the joints

E_c = modulus of elasticity of column concrete

E_b = modulus of elasticity of beam concrete

I_c = moment of inertia of column cross section about an axis perpendicular to the plane of buckling being considered.

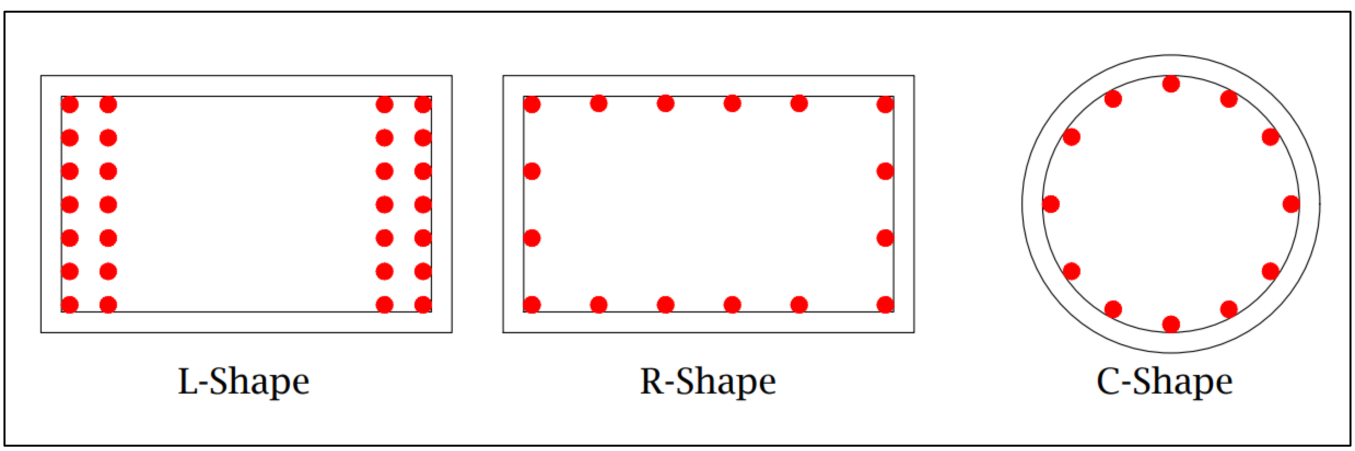
I_b = moment of inertia of beam cross section about an axis perpendicular to the plane of buckling being considered.

After the classifications, the column is possibility to be one of four possibilities:

1. Non-Sway and short column.

1. Assume suitable diminutions for the column.

2. Select the shape of column (L, R, or C)



3. Analysis the frame and find the moments and normal forces.

4. Calculate the ϒ value → $\Upsilon = \dfrac{h - 2(\text{cover}) - 2\left(d_{stirrup}\right) - d_b}{h}$

Note:

1. The ϒ value must be taken in the direction of bending

   from the farthest center of reinforcement bars to the farthest center of reinforcement bars.

2. The worst case is when we approximate the value of ϒ to the lower value because it gives

   a greater amount of reinforcement.

5. $K_n = \dfrac{P_n}{f'_c \times A_g}$

6. $R_n = \dfrac{P_n \times e}{f'_c \times A_g \times h} = \dfrac{M_n}{f'_c \times A_g \times h}$

$P_u = \phi P_n$ $\quad M_u = \phi M_n$ $\quad \phi = 0.65$ for tie columns $\quad \phi = 0.75$ for spiral columns

$$K_n = \frac{P_u}{\phi \times f'_c \times A_g}$$

$$R_n = \frac{M_u}{\phi \times f'_c \times A_g \times h}$$

7. Using strength interaction diagram. {Example (L4 − 60.8)} Inputs K_n & R_n → Output ρ

L: the shape of cross section (L, R, C)

4: the compressive strength of concrete $f'c$ in (ksi) unit.

60: yeild stress f_y in (ksi) unit.

0.8: the value of ϒ.

$$A_S = \rho \times A_g$$

$$\#\text{ of bars} = \frac{A_s}{\text{Area of one bar}}$$

Check for clear distance between bars (in each direction)

$S_c > 4$ cm and $S_c > 2.5\ d_b$

Calculate the spacing between the ties

$S = \min.\left(48 \times d_{stirrup}, 16 \times d_b, \text{least cross sectional dimension}\right).$

2. Non-sway frame and long column (Magnification Method)

$M_{design} = \delta_{ns} \times M_2 \geq \delta_{ns} \times M_{2,min.}$

$M_{2,min.} = P_u \times (15.00 + 0.03h)$, where the units within the bracket are given in millimeters.

δ_{ns} = moment magnification factor for non − sway frames , given by

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 \times P_{cr}}} \geq 1.00$$

C_m = factor relating actual moment digram to an equivalent uniform moment digram.

For members without transverse loads between the supports, C_m is taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2}$$

M_1 = smaller factored end moment on column, positive if member is bent single curvature, negative if bent in double curvature.

M_2 = larger factored end moment on column, always positive.

Notes:

1. For columns with transverse loads between supports, C_m is taken 1.00.
2. if $M_{2,min.} > M_2 \rightarrow C_m$ is taken 1.00 or $C_m = 0.6 + 0.4\frac{M_1}{M_2}$

$$P_{cr} = \frac{\pi^2 (EI)_{eff.}}{(kl_u)^2}$$

$$EI = \frac{0.4 \times E_c \times I_g}{1 + \beta_{dns}}$$

E_s = modulus of elasticity of concrete

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement.

$$\beta_{dns} = \frac{\text{Maximum factored axial sustained load}}{\text{Maximum factored axial associated with the same laod combination}} \le 1.00$$

Example

$$\beta_{dns} = \frac{1.2DL + (40\%)1.6LL}{1.2DL + 1.6LL}$$

Calculate the Υ value $\rightarrow \Upsilon = \dfrac{h - 2(\text{cover}) - 2(d_{stirrup}) - d_b}{h}$

$$K_n = \frac{P_n}{f'_c \times A_g}$$

$$R_n = \frac{P_n \times e}{f'_c \times A_g \times h} = \frac{M_n}{f'_c \times A_g \times h}$$

$P_u = \phi P_n$ $\quad M_u = \phi M_n$ $\quad \phi = 0.65$ for tie columns $\quad \phi = 0.75$ for spiral columns

$$K_n = \frac{P_u}{\phi \times f'_c \times A_g}$$

$$R_n = \frac{M_u}{\phi \times f'_c \times A_g \times h}$$

Using strength interaction diagram. {Example (L4 − 60.9)} Inputs K_n & R_n → Output ρ

$$A_S = \rho \times A_g$$

$$\#\text{ of bars} = \frac{A_s}{\text{Area of one bar}}$$

Check for clear distance between bars (in each direction). $S_c > 4$ cm and $S_c > 2.5\ d_b$

Calculate the spacing between the ties

$S = \min.(48 \times d_{stirrup}, 16 \times d_b$, least cross sectional dimension$)$.

3. Sway frame and long column (Magnification Method)

$M_1 = M_{1ns} + \delta_s \times M_{1s}$

$M_2 = M_{2ns} + \delta_s \times M_{2s}$

$$\delta_s = \frac{C_m}{1 - \frac{\sum P_u}{0.75 \times \sum P_{cr}}}$$

M_{1ns} = factored end moment at the end M_1 acts due to loads that cause no sway calculated using a first − order elastic frame analysis

M_{2ns} = factored end moment at the end M_2 acts due to loads that cause no sway calculated using a first − order elastic frame analysis

M_{1s} = factored end moment at the end M_1 acts due to loads that cause no sway calculated using a first − order elastic frame analysis

M_{2s} = factored end moment at the end M_2 acts due to loads that cause substantial sway calculated using a first − order elastic frame analysis

δ_s = moment magnification factor for sway frames to reflect lateral drift resulting from lateral and gravity loads

Note

In the project, the magnified sway moments $\delta_s M_s$ are computed by a second-order elastic frame analysis may be used.

Calculate the ϒ value → $\Upsilon = \dfrac{h - 2(\text{cover}) - 2(d_{stirrup}) - d_b}{h}$

$$K_n = \frac{P_n}{f'_c \times A_g}$$

$$R_n = \frac{P_n \times e}{f'_c \times A_g \times h} = \frac{M_n}{f'_c \times A_g \times h}$$

$P_u = \phi P_n$ $M_u = \phi M_n$ $\phi = 0.65$ for tie columns $\phi = 0.75$ for spiral columns

$$K_n = \frac{P_u}{\phi \times f'_c \times A_g}$$

$$R_n = \frac{M_u}{\phi \times f'_c \times A_g \times h}$$

Using strength interaction diagram. {Example (L4 − 60.9)} Inputs K_n& $R_n \rightarrow$ Output ρ

$A_S = \rho \times A_g$

$$\text{\# of bars} = \frac{A_s}{\text{Area of one bar}}$$

Check for clear distance between bars (in each direction). $S_c > 4$ cm and $S_c > 2.5\ d_b$

Calculate the spacing between the ties

$S = \min.(48 \times d_{stirrup}, 16 \times d_b$, least cross sectional dimension)

4. Sway frame and short column.

The same steps of sway frame and short column but the K value is different (from the chart of sway frame).

1. Assume suitable diminutions for the column.

2. Select the shape of column (L, R, or C)

3. Analysis the frame and find the moments and normal forces.

4. Calculate the Υ value $\rightarrow \Upsilon = \dfrac{h - 2(\text{cover}) - 2(d_{stirrup}) - d_b}{h}$

5. $K_n = \dfrac{P_n}{f'_c \times A_g}$

6. $R_n = \dfrac{P_n \times e}{f'_c \times A_g \times h} = \dfrac{M_n}{f'_c \times A_g \times h}$

$P_u = \phi P_n$ $M_u = \phi M_n$ $\phi = 0.65$ for tie columns $\phi = 0.75$ for spiral columns

$$K_n = \frac{P_u}{\phi \times f'_c \times A_g}$$

$$R_n = \frac{M_u}{\phi \times f'_c \times A_g \times h}$$

7. Using strength interaction diagram. {Example (L4 − 60.8)} Inputs K_n& R_n → Output ρ

$$A_S = \rho \times A_g$$

$$\# \text{ of bars} = \frac{A_s}{\text{Area of one bar}}$$

Check for clear distance between bars (in each direction). $S_c > 4$ cm and $S_c > 2.5\ d_b$

Calculate the spacing between the ties

$$S = \min.(48 \times d_{stirrup}, 16 \times d_b, \text{least cross sectional dimension})$$

Important Note:

1. In the project, we are used the Autodesk ROBOT structural analysis program to find the forces and moments.

2. The Autodesk ROBOT structural analysis program in the process of analysis is used the second order analysis (exact P-Δ analysis); therefore, in the project we are not use the magnification method and the results from the program used directly in design process.