



Civil Engineering Department

Wolkite University, College of Engineering and Technology

Structural design of G+4 Apartment + shop building

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As members of the examining board of the final B.Sc. open defense, we verify that we have read and evaluated the final BSc thesis/project prepared by **Ijara Taresa Tikesa**, **Tilahun Feyissa Gode**, **Semira Ali Mohammed**, **Eyerusalem Chernet kasaye**, **Rebka Melaku Chernet**, **Ibrahim Sadik Hassen**, entitled **Structural design of G+4 Apartment + shop building**, and recommended for acceptance as a fulfillment of the requirement of **B.Sc. in Civil Engineering**

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Abstract

A structural designer's task is mainly to predict the loads that the structure to be built will carry throughout its service period, to estimate the amount of these loads with a series of empirical formulas which are given in different codes and with the use of preliminary geometric equations.

This project deals about the structural analysis and design of Structural design of G+4 Apartment + shop building considering all the internal and external effects according to ES EN 2015.

Then the structure is exposed to a worst-case combination or existence scenario of these loads, analyzed for internal actions that will develop in the different structural parts. Finally these parts are proportioned geometrically and reinforced with steel bars in such a way that they could resist the internal actions shear, bending moment and axial force.

The roof structure is analyzed and designed under the action of own weight and the live loads. The slabs and stair is designed under the action of dead and live load. The loads that come from roof, slab and stair will be transferred to the beam then to column and footing frame together.

The frame structure is analyzed under the combined action of these load and the internal actions in the frame members is determined.

Finally, the frame members are designed in such a way that they could resist these internal actions.

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Abbreviation

A	Accidental action
A	Cross sectional area
A_c	Cross sectional area of concrete
A_p	Area of a prestressing tendon or tendons
A_s	Cross sectional area of reinforcement
$A_{s,min}$	minimum cross sectional area of reinforcement
A_{sw}	Cross sectional area of shear reinforcement
$E_{c,eff}$	Effective modulus of elasticity of concrete
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
$E_c(t)$	Tangent modulus of elasticity of normal weight concrete at a stress of
$\sigma_c = 0$	and at time t
F	Action
F_d	Design value of an action
F_k	Characteristic value of an action
G_k	Characteristic permanent action
I	Second moment of area of concrete section
L	Length
M	Bending moment
M_{Ed}	Design value of the applied internal bending moment
N	Axial force
N_{Ed}	Design value of the applied axial force (tension or compression)
P	Prestressing force
P_0	Initial force at the active end of the tendon immediately after stressing
Q_k	Characteristic variable action
R	Resistance
ULS	Ultimate limit state
V	Shear force
V_{Ed}	Design value of the applied shear force

Latin lower case letters

b	Overall width of a cross-section, or actual flange width in a T or L beam
b_w	Width of the web on T, I or L beams
d	Diameter ; Depth
d	Effective depth of a cross-section
d_g	Largest nominal maximum aggregate size
e	Eccentricity
f_c	Compressive strength of concrete
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_p	Tensile strength of prestressing steel
f_{pk}	Characteristic tensile strength of prestressing steel
f_t	Tensile strength of reinforcement

f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
f_{ywd}	Design yield of shear reinforcement
h	Height
k	Coefficient ; Factor
l	(or l or L) Length; Span
r	Radius
$1/r$	Curvature at a particular section
t	Thickness
t	Time being considered
u	Perimeter of concrete cross-section, having area A_c
u, v, w	Components of the displacement of a point
x	Neutral axis depth
x, y, z	Coordinates
z	Lever arm of internal forces

Greek lower case letters

α	Angle; ratio
β	Angle; ratio; coefficient
γ	Partial factor
γ_A	Partial factor for accidental actions A
γ_C	Partial factor for concrete
γ_G	Partial factor for permanent actions, G
γ_Q	Partial factor for variable actions, Q
γ_S	Partial factor for reinforcing or prestressing steel
δ	Increment/redistribution ratio
ϵ_c	Compressive strain in the concrete
ϵ_{c1}	Compressive strain in the concrete at the peak stress f_c
ϵ_{cu}	Ultimate compressive strain in the concrete
ϵ_u	Strain of reinforcement or prestressing steel at maximum load
ϵ_{uk}	Characteristic strain of reinforcement or prestressing steel at maximum load
θ	Angle
λ	Slenderness ratio
μ	Coefficient of friction between the tendons and their ducts
ν	Poisson's ratio
ν	Strength reduction factor for concrete cracked in shear
ρ	Oven-dry density of concrete in kg/m^3
ρ_l	Reinforcement ratio for longitudinal reinforcement
ρ_w	Reinforcement ratio for shear reinforcement
σ_c	Compressive stress in the concrete
σ_{cp}	Compressive stress in the concrete from axial load or prestressing
σ_{cu}	Compressive stress in the concrete at the ultimate compressive strain ϵ_{cu}
τ	Torsional shear stress
ϕ	Diameter of a reinforcing bar or of a prestressing duct
ϕ_h	Equivalent diameter of a bundle of reinforcing bars

$\varphi(t,t_0)$ Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days

$\varphi(\infty,t_0)$ Final value of creep coefficient

ψ Factors defining representative values of variable actions
 ψ_0 for combination values
 ψ_1 for frequent values
 ψ_2 for quasi-permanent value

1. Background

1.1. Introduction

The primary aim of all structural design is to ensure that the structure will perform satisfactorily during its design life. Specifically, the design must check that the structure can carry the loads safely and that it will not deform excessively due to the applied loads. This requires the designer to make realistic estimates of the strengths of the materials composing the structure and the loading to which it may be subjected during its design life. Furthermore, the designer will need a basic understanding of structural behavior.

The designer must assess the future likely level of loading, including self-weight, to which the structure may be subjected during its design life. Using computer methods or hand calculations the design loads acting on individual elements can be then be evaluated. The design loads are used to calculate the bending moments, shear forces and deflections at critical points along the elements. Finally, suitable dimensions for the element can be determined.

Building structure is a structure which is composed of both structural and non-structural elements. The primary purpose of structural members is to build up and maintain its shape under a considerable load action to have adequate stability strength and stiffness. Not only safety but it also fulfills the requirement of serviceability as well as it should be economical. Regarding safety, it requires that the strength of the structure should be adequate for all the loads which are reasonably considered.

When we come to the serviceability, it requires that all kinds of loads likely to occur during use, everything should be satisfactory, for example excessive deflection should be adequately small; vibration should be within the tolerable limit, the maximum width of crack also should not be greater than the specified limit etc. The structure also should be economical keeping its safety against collapse. Generally, the building structure to serve its purpose, it must be safe against collapse, economical to user and gives its serviceability in use (functional).

1.2. Architectural design review

The building height is set to be G+4 with typical floor height of 2.88m each. The building area covers total of 338 m². all floor plans are typical excluding ground floor

plan. Ground floor is designed for shop and living apartments. The remaining first up to fourth floors are fully designed for apartment living rooms. (Appendix A) Solid slabs and cantilever slabs are used in the building. To transfer loads comes from slabs to columns beams with different dimensions are used. And, columns will be arranged in accordance to transferred loads. Solid RC slab roof is used to cover the topmost of the building.

1.3. Objectives

Main objective

The main objective of this thesis project is to design safe, durable and economical building.

Sub objectives

When we do this project, we improve our skills on:

- ✓ Documentation of building project structural design
- ✓ Software related to structural design like Etabs, Sap, Auto cad, MS-excel etc...
- ✓ Reading and preparing detailed drawings of structure
- ✓ Importing design data from ES EN 2015
- ✓ Teamwork

2. Wind load analysis and design of roof

Wind produces dynamic loads on a structure at highly variable magnitudes. The variation in pressures at different locations on a building is complex to the point that pressures may become too analytically intensive for precise consideration in design. To simplify the complexity in analysis of wind load different codes provides specifications for wind load by considering basic static pressure zones on a buildings representative of peak loads that are likely to be experienced. Wind forces act directly or indirectly on the internal and external surface of structures. Wind loads fluctuate with time. A wind load produces static, dynamic and aerodynamic effects on structures. Analysis of wind load Wind loads is dynamic loads and there are two methods of analysis of dynamic loads.

Quasi- Static method: This is applied to stiff structures in which the movement of the structure with wind is negligible.

Dynamic analysis: in which the movement of structures with wind loads in considered

The choice of the two procedures is based on the height of the building and the dynamic coefficient, Cd. Static method or the simplest method of wind load analysis issued for buildings and chimneys less than 200m tall provided that the value of dynamic coefficient. In all other cases the detailed analysis considers the dynamic effect of the load namely dynamic analysis should be applied.

The effect of wind load on the structure depends on many factors such as:

- Wind velocity direction
- The height of the s structure
- Topographic location of the structure
- Shape of the structure
- Terrain category and the roughness of the surrounding.

Table 2-1 Necessary information on building for wind load analysis

Roof type	Flat roof
-----------	-----------

Structural design of G+4 Apartment + shop building B.Sc Thesis project

Location	Wolkite
Building height	14.4m
Terrain category	Category III: - for area with regular cover of vegetation or buildings with obstacles with separations of maximum 20 obstacle heights(such as villages, sub urban terrain, permanent forest)
Directional factor	1: recommended value
Seasonal factor	1: recommended value
Basic Wind velocity	22m/s: for Ethiopia
Z_0	0.3m: for terrain category III from table
$Z_{0,II}$	0.05m: for (terrain category II)
Minimum height	5m: for terrain category III from table
maximum height	200m
orography factor	1: recommended
Turbulence factor	1: recommended
Air density	1.25 kg/m ³ : recommended

b	15.44 for $\Theta=90^\circ$, 28.07 for $\Theta=0^\circ$
Reference height	14.4m : for $h < b$
Live load	0.4kN/m: recommended value for roof not accessible except for normal maintenance and repair

2.1. Wind load analysis

- determination of basic wind velocity V_b

$$V_b = C_{dir} C_{sea} V_{bo}$$

Where C_{dir} = directional factor

C_{sea} = seasonal factor

V_{bo} = fundamental basic wind velocity

$$V_b = 1 * 1 * 22 \text{ m/s} = \mathbf{22 \text{ m/s}}$$

- Roughness factor $C_r(Z)$

$$C_r(Z) = \begin{cases} K_r \ln\left(\frac{Z}{Z_0}\right) & \text{for } Z_{\min} \leq Z \leq Z_{\max} \\ C_r(Z_{\min}) & \text{for } Z \leq Z_{\min} \end{cases}$$

$$\text{Where } K_r = 0.19 \left(\frac{Z_0}{Z_{0,II}}\right)^{0.07}$$

$$Z_{0,II} = 0.05\text{m (terrain category II)}$$

Z_{\min} = minimum height

Z_{\max} = maximum height taken as 200m

$$K_r = 0.19 \left(\frac{0.3}{0.05}\right)^{0.07} = \mathbf{0.215}$$

$$C_r(Z) = K_r \ln\left(\frac{Z}{Z_0}\right) \quad \text{for } 5 \leq 14.4 \leq 200$$
$$= 0.215 \ln\left(\frac{14.4}{0.3}\right)$$

$$C_r(Z) = \mathbf{0.832}$$

➤ Mean wind velocity $V_m(Z)$

$$V_m(Z) = C_r(Z) * C_0(Z) * V_b$$

where $C_r(Z)$ = Roughness factor

$C_0(Z)$ = orography factor taken as 1

V_b = basic wind velocity

$$V_m(Z) = 0.832 * 1 * 22 \text{ m/s} = \mathbf{18.31 \text{ m/s}}$$

➤ Wind turbulence $l_v(Z)$

$$l_v(Z) = \frac{K_1}{C_0(Z) * \ln\left(\frac{Z}{Z_0}\right)} \quad \text{for } Z_{\min} \leq Z \leq Z_{\max}$$

Where K_1 = turbulence factor

$C_0(Z)$ = orography factor

$$l_v(Z) = \frac{1}{1 * \ln\left(\frac{14.4}{0.3}\right)} = \mathbf{0.258}$$

➤ Peak velocity pressure $q_p(Z)$

$$q_p(Z) = [1 + 7 * l_v(Z)] * \frac{1}{2} * \rho * V_m^2(Z)$$

where ρ = air density

$$q_p(Z) = [1 + 7 * 0.258] * \frac{1}{2} * 1.25 * 18.31^2 = \mathbf{587.96 \text{ N} = 0.588 \text{ KN}}$$

- Wind pressure
 - External pressure

$$W_e = q_p(Z) \cdot C_{pe}$$

where, $q_p(Z_e)$ peak velocity pressure

Z_e reference height

C_{pe} external pressure coefficient

- ✓ External pressure coefficient

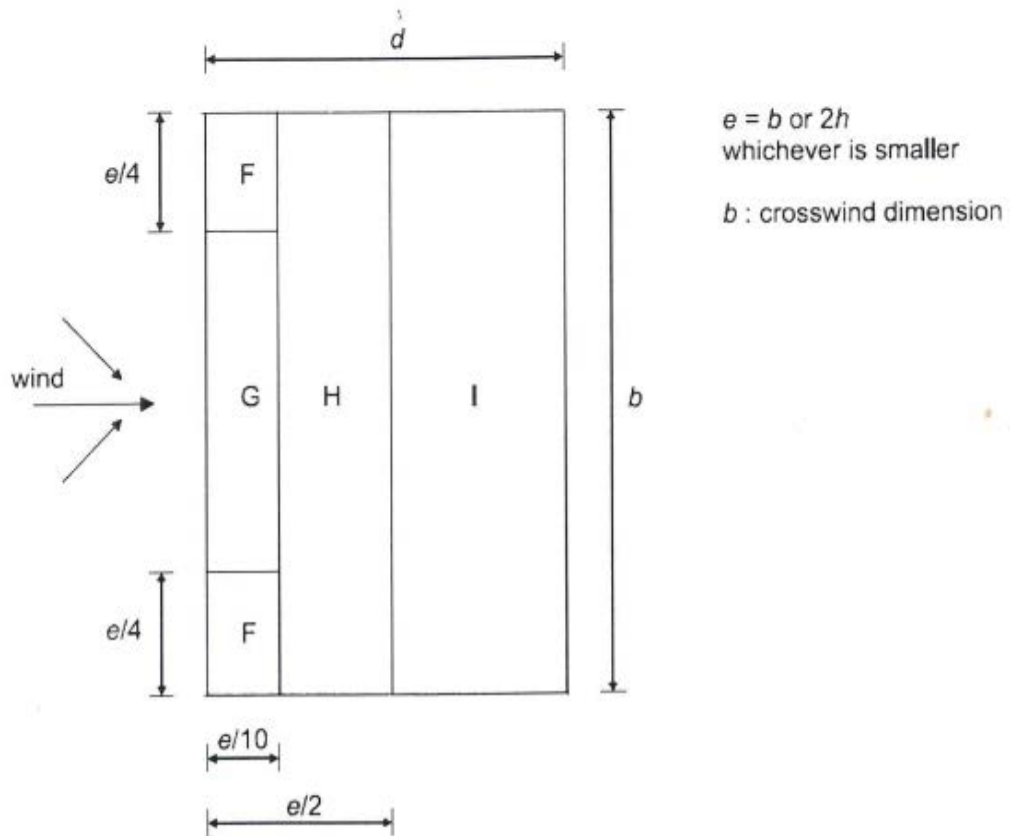
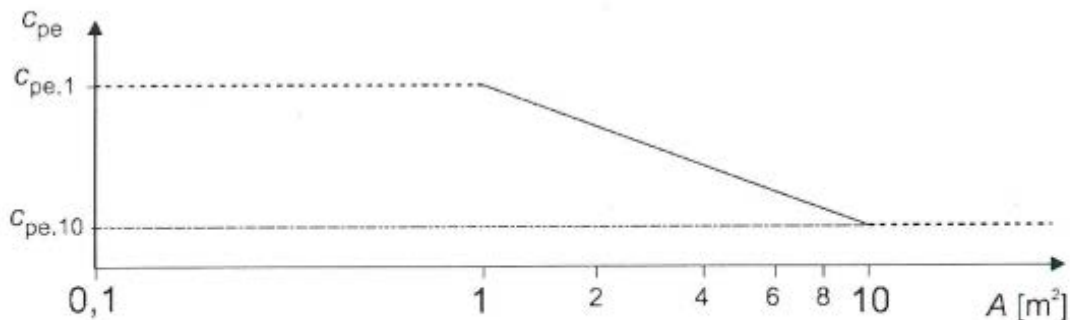


Figure 2-1key for flat roofs

Table 2-2 external pressure coefficients for flat roofs with mansard eaves angle of 60°

Roof type		Zone							
		F		G		H		I	
		C _{pe,1} 0	C _{pe} , 1	C _{pe,1} 0	C _{pe} , 1	C _{pe,1} 0	C _{pe} , 1	C _{pe,1} 0	C _{pe} , 1
Mansard Eaves	α=60°	-1.3	1.9	-1.3	-1.9	-0.5		+0.2	-0.2



The figure is based on the following:

$$\text{for } 1 \text{ m}^2 < A < 10 \text{ m}^2 \quad c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A$$

Figure 2-2 Recommended procedure for determining the external pressure coefficient *c_{pe}* for buildings with loaded area between 1m² and 10m²

$$\text{For } 1 \text{ m}^2 < A < 10 \text{ m}^2, \quad c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A$$

Case -1 $\theta=90^\circ$

$$b = 15.44, \quad d = 28.07\text{m}, \quad h=14.4$$

$$2h= 28.8$$

$$15.44 < 28.8 \text{ so, } e = 15.44 \text{ m}$$

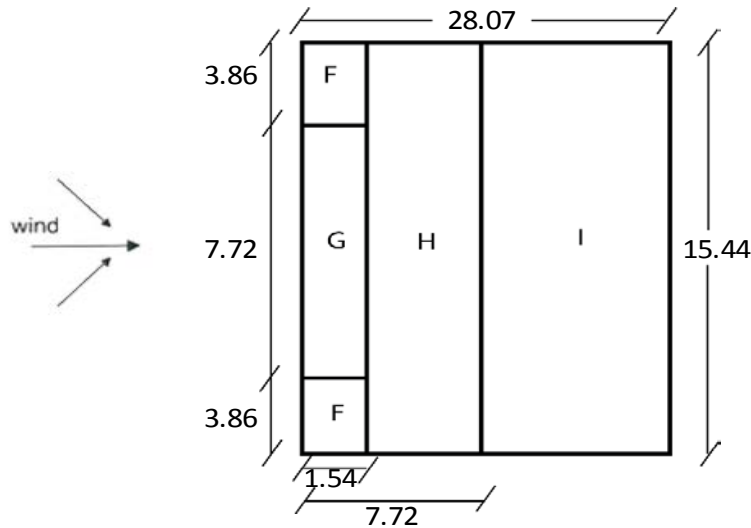


Figure 2-3 Pressure Distribution zones $\theta=90^\circ$

For $1\text{m}^2 < A < 10\text{m}^2$, $c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A$

$$W_e = q_p(Z) \cdot C_{pe}$$

Table 2-3 External wind pressure $\theta=90^\circ$

zone	Cpe values		cpe	w_e
F	cpe,10	-1.3	-1.01	-0.593
	cpe,1	1.9		
G	cpe,10	-1.3	-1.30	-0.764
	cpe,1	-1.9		
H	cpe,10	-0.5	-0.50	-0.294
	cpe,1	-0.5		
I	cpe,10	0.2	0.20	0.118
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.118
	cpe,1	0.2		

Case -2 $\theta=0^\circ$

$b = 28.07$, $d = 15.44\text{m}$, $h=14.4$

$2h = 28.8$

$28.07 < 28.8$ so, $e = 28.07 \text{ m}$

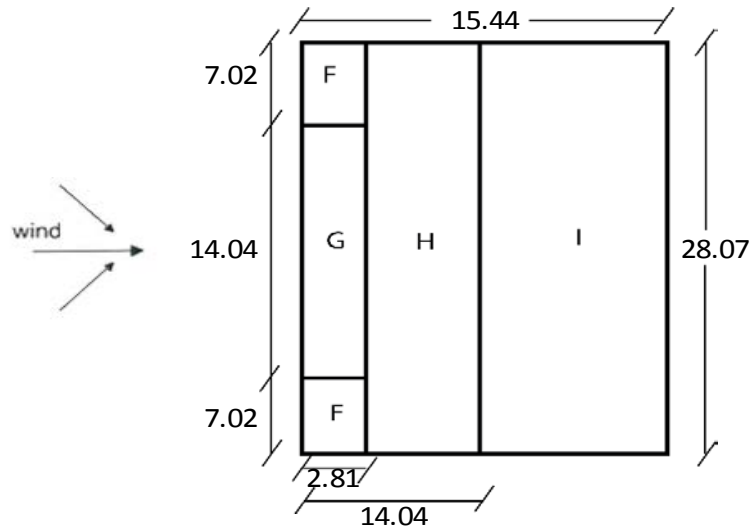


Figure 2-4 Pressure distribution zones $\Theta=0o$

Table 2-4 External wind pressure $\Theta=0o$

zone	Cpe values		cpe	w_e
F	cpe,10	-1.3	-1.30	-0.764
	cpe,1	1.9		
G	cpe,10	-1.3	-1.30	-0.764
	cpe,1	-1.9		
H	cpe,10	-0.5	-0.50	-0.294
	cpe,1	-0.5		
I	cpe,10	0.2	0.20	0.118
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.118
	cpe,1	0.2		

Critical external pressures: - pressure = **0.118** and suction = **-0.764**

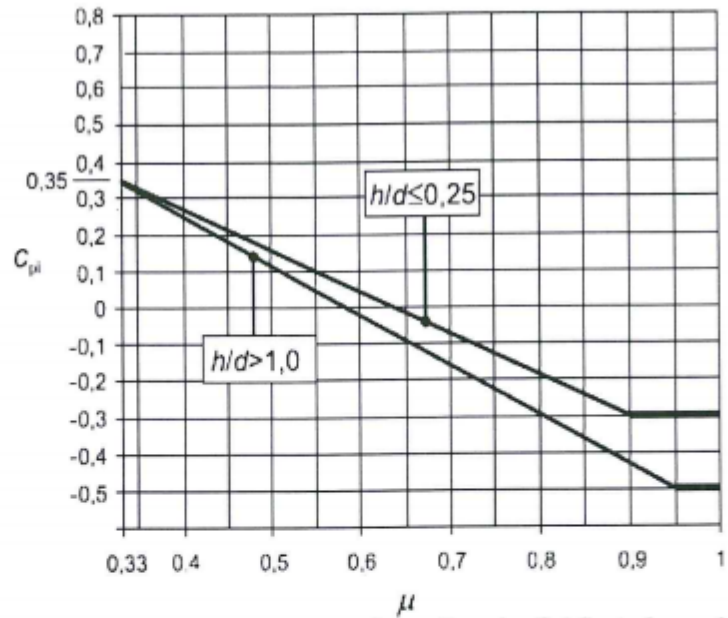
- o Internal pressure

$$W_i = q_p(Z_i) \cdot C_{pi}$$

where, $q_p(Z_i)$ peak velocity pressure

Z_i reference height

C_{pi} internal pressure coefficient



7.13 — Internal pressure coefficients for uniformly distributed openings

$$\mu = \frac{\sum \text{area of openings where } c_{pi} \text{ is negative or } -0.0}{\sum \text{area of all openings}} \quad (7.4)$$

NOTE 1 This applies to façades and roof of buildings with and without internal partitions.

NOTE 2 Where it is not possible, or not considered justified, to estimate μ for a particular case then c_{pi} should be taken as the more onerous of +0,2 and -0,3.

Figure 2-5 Internal pressure coefficients for uniformly distributed openings

As described in note 2 C_{pi} taken as 0.2 and -0.3

$$W_i = 0.588 * 0.2 = 0.118$$

$$= 0.588 * -0.3 = -0.176$$

➤ Net pressure = $W_e - W_i$

W_e	W_i	W_{net}
0.118	0.118	0
-0.764	-0.176	-0.588

3. Analysis and design of slab

A reinforced concrete slab is a broad, flat plate, usually horizontal, with top and bottom surface parallel or nearly so. It is used to provide flat surface mainly for roofs and floors of building parking lots, airfields, roadways, bridges etc. It may be supported by reinforced concrete beam, by masonry or reinforced concrete walls, by structural steel members, directly by columns, or continuously by the ground. But we are intending to design beam supported floor and roofs solid slabs of the building accordingly (*ES EN 1992-1-1*, 2015)

Beam support slab can be classified as one-way slab and two-way slab based on the longer to shorter length ratio.

A. One-way slabs: - main reinforcement in each element runs in one direction only. The slab is one way if the slab panel longer length to shorter length ratio is greater than two (i.e. $L_y/L_x > 2$).

B. Two-way slab: -main reinforcement runs in both directions where ratio of long to short span is less than or equal to two (i.e. $L_y/L_x \leq 2$).

Panel selection

Before proceeding to the analysis and design of solid slab we should select type of panel. The selection of panels is depending on:

- Boundary condition
- Shorter (L_x) and longer(L_y) length of the panel

The first step in the analysis and design of slab is concrete cover design, after we design the concrete cover, we can calculate the depth of slab. Based on depth of slab, floor finish, ceiling plaster and live load of slab the design load is computed. Individual panels have moment based on their respective design load and this moment can affect neighboring panels as the slab is continuous, to counter this effect support moment and field moment adjustment needed. When we finish adjusting the moments, we check depth of flexure, calculate shear force and design the slab for shear. Finally, we design the reinforcement for the whole slab.

3.1. Analysis and design of ground floor slab

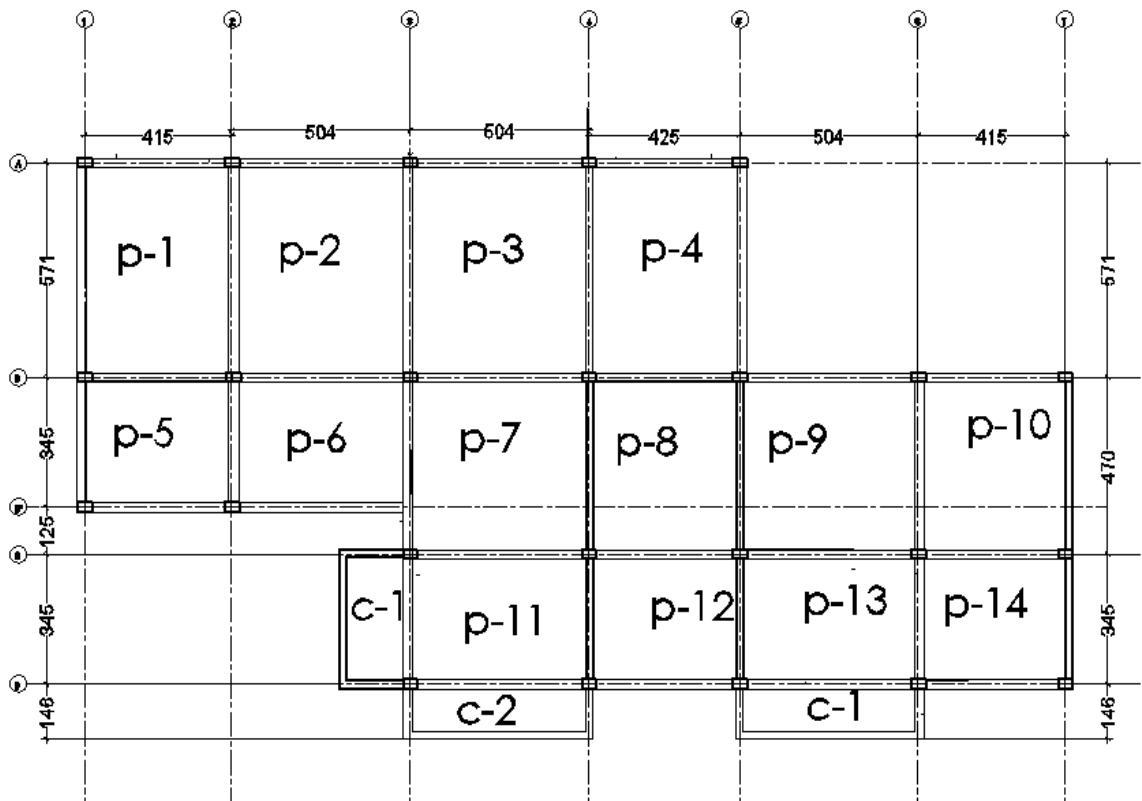


Figure 3-1 Ground floor slab layout

3.1.1. Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

- a. Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water.[Appendix B,]
- b. Minimum strength class = C20/25, for exposure class XC1[Appendix B,]
- c. Minimum concrete cover for bond/durability

$$d. C_{min} = \max \begin{Bmatrix} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{Bmatrix} = \max \begin{Bmatrix} 12mm \\ 10mm \\ 10mm \end{Bmatrix} = 12mm$$

i. $C_{min, b} = 12$ mm, assumed diameter of bar [Appendix B,]

ii. $C_{min, dur} = 10$ mm,

1. $C_{min}=12\text{mm}$

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 12\text{mm} + 10\text{mm} = 22\text{mm}$

i. $\Delta c_{dev} = 10\text{mm}$, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

g. Governing concrete cover = 22mm

3.1.2. Depth determination

We use C20/25 concrete and S400 rebar

Serviceability requirement

$$\frac{l}{d} = N * K * F1 * F2 * F3$$

$$N = 11 + 1.5\sqrt{f_{ck}} = 11 + 1.5\sqrt{20} = 17.71$$

K= Coefficient factor for different structural systems =11.3 for end span, 11.5 for interior span and 0.4 for cantilever slab.

$$F = \text{factor of safety } F1 = \frac{500}{f_{yk}} = 1.25 \quad F2=1, F3=1$$

Table 3-1 Depth determination for ground floor

Panel	ly (mm)	lx (mm)	ly/lx	Type	structural system	k	N	lx/d	d
P-1	5710	4150	1.38	two way	end span	1.3	17.71	28.78	144.20
P-2	5710	5040	1.13	two way	end span	1.3	17.71	28.78	175.13
P-3	5710	5040	1.13	two way	end span	1.3	17.71	28.78	175.13
P-4	5710	4250	1.34	two way	end span	1.3	17.71	28.78	147.68
P-5	4150	3450	1.20	two way	end span	1.3	17.71	28.78	119.88
P-6	5040	3450	1.46	two way	end span	1.3	17.71	28.78	119.88
P-7	5040	4700	1.07	two way	continous	1.5	17.71	33.21	141.54
P-8	4700	4250	1.11	two way	continous	1.5	17.71	33.21	127.99
P-9	5040	4700	1.07	two way	end span	1.3	17.71	28.78	163.31
P-10	4700	4150	1.13	two way	end span	1.3	17.71	28.78	144.20
P-11	5040	3450	1.46	two way	continous	1.5	17.71	33.21	103.90
P-12	4250	3450	1.23	two way	end span	1.3	17.71	28.78	119.88
P-13	5040	3450	1.46	two way	continous	1.5	17.71	33.21	103.90
P-14	4150	3450	1.20	two way	end span	1.3	17.71	28.78	119.88
C-1	3710	1600	2.32	cantiliver	cantiliver	0.4	17.71	8.86	180.69
C-2	5040	1460	3.45	cantiliver	cantiliver	0.4	17.71	8.86	164.88
C-3	5040	1460	3.45	cantiliver	cantiliver	0.4	17.71	8.86	164.88
Governing panel depth(mm)									180.69 mm
clear cover(mm)									25 mm
Assuming reinforcements ϕ									10 mm
D(mm)=d+c+(ϕ /2)									211 mm
use D minimum									220 mm

3.1.3. Reinforcement

- Area of steel

$$A_{st,min} = \max \begin{cases} 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \frac{2.2}{400} 1000 * 220 = 314.6 \text{ mm}^2 \\ 0.0013bd = 0.0013 * 1000 * 220 = 286 \text{ mm}^2 \end{cases}$$

$$A_{st,min} = \mathbf{314.6 \text{ mm}^2}$$

- Spacing

$$S_{max} = \max \begin{cases} 3D = 3 * 220 = 660 \text{ mm} \\ 400 \text{ mm} \end{cases}$$

$$S_{max} = \mathbf{660 \text{ mm}}$$

Let the ϕ of reinforcement be 12mm

$$a_{st} = \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = \mathbf{113.04 \text{ mm}^2}$$

$$\text{Spacing of bars} = S = \frac{ba_s}{A_s} = \frac{1000 * 113.04}{314.6} = \mathbf{359.31 \text{ mm}}$$

359.31 mm \approx 350 mm used < 630mm OK!! So ϕ 12 c/c 350 used for whole ground floor slab.

3.2. Analysis and design of typical 1st -4th floor slab

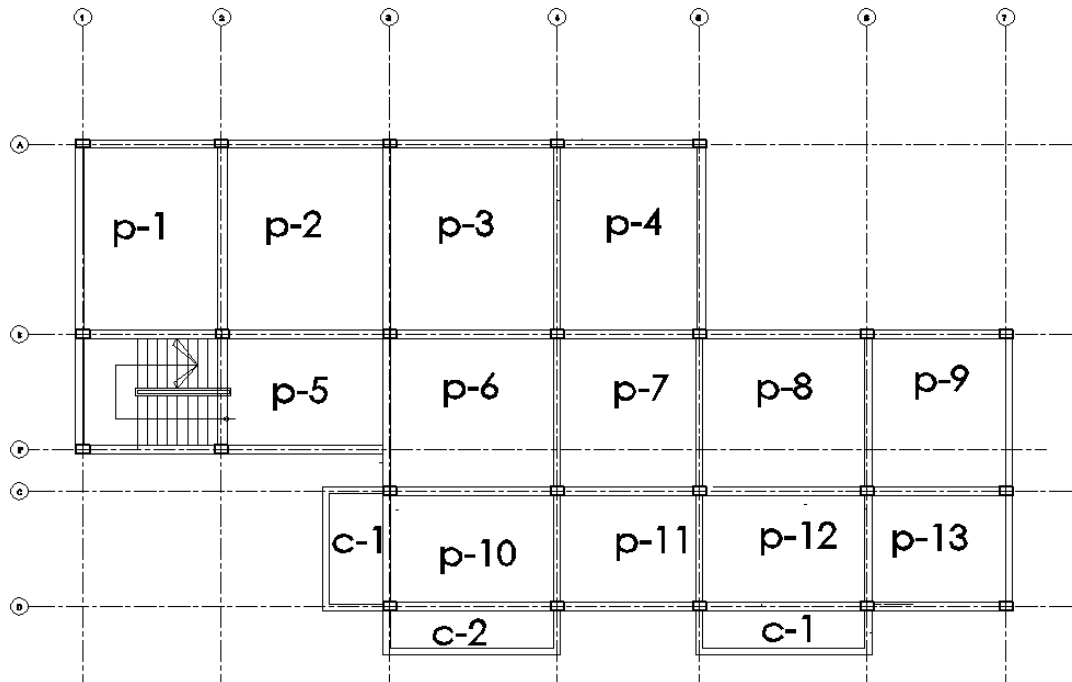


Figure 3-2 floor slab layout for typical 1st -4th floor

3.2.1. Concrete cover design

The concrete cover is the distance from the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface. The nominal cover shall be specified on the drawings. It is defined as $C_{min} C_{nom} = C_{min} + \Delta C_{dev}$

Where, C_{min} – minimum cover and ΔC_{dev} is allowance in design for deviation.

Minimum cover, shall provide in order to ensure

- Safe transmission of bond force
- Corrosion resistance/ Durability
- Fire resistance

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, b} \\ C_{min, dur} + \Delta C_{dur, \gamma} - \Delta C_{dur, st} - \Delta C_{dur, add} \\ 10mm \end{array} \right.$$

Where;

- ✚ $C_{min, b}$ - minimum cover due to bond requirement, see ES EN1992:2015 Art. 4.4.1.2 (3).
- ✚ $C_{min, dur}$ - minimum cover due to environmental conditions, see ES EN 1992:2015 Art 4.4.1.2 (5)
- ✚ $\Delta C_{dur, \gamma}$ - additive safety element, see ES EN1992:2015 Art 4.4.1.2 (6) $\Delta C_{dur, st}$ - reduction of minimum cover for use of stainless steel, see ES EN 1992:2015 Art 4.4.1.2 (7)
- ✚ $\Delta C_{dur, add}$ - reduction of minimum cover for use of additional protection, see EN1992:2015 Art 4.4.1.2 (8)

Cover is designed according to (*ES EN 1992-1-1, 2015*)

- a. Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. [Appendix B,](ES EN 1992-1-1, 2015)
- b. Minimum strength class = C20/25, for exposure class XC1[Appendix B,]
- c. Minimum concrete cover for bond/durability

$$d. C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{array} \right\} = \max \left\{ \begin{array}{l} 12mm \\ 10mm \\ 10mm \end{array} \right\} = 12mm$$

- i. $C_{min, b} = 12$ mm, assumed diameter of bar [Appendix B,]
- ii. $C_{min, dur} = 10$ mm,

1. $C_{min} = 12mm$

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 12\text{mm} + 10\text{mm} = 22\text{mm}$

i. $\Delta c_{dev} = 10\text{mm}$, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

g. Governing concrete cover = 22mm

3.2.2. Depth determination

We use C20/25 concrete from minimum strength class concrete and S400 rebar

Serviceability requirement

$$\frac{l}{d} = N * K * F1 * F2 * F3$$

$$N = 11 + 1.5\sqrt{f_{ck}} = 11 + 1.5\sqrt{20} = 17.71$$

K= Coefficient factor for different structural systems =11.3 for end span, 11.5 for interior span and 0.4 for cantilever slab.

Table 3-2 Depth determination of typical 1st -4th floor

Panel	ly (mm)	lx (mm)	ly/lx	Type	structural system	k	N	lx/d	d
P-1	5710	4150	1.38	two way	end span	1.3	17.71	28.78	144.20
P-2	5710	5040	1.13	two way	end span	1.3	17.71	28.78	175.13
P-3	5710	5040	1.13	two way	end span	1.3	17.71	28.78	175.13
P-4	5710	4250	1.34	two way	end span	1.3	17.71	28.78	147.68
P-5	5040	4700	1.07	two way	continous	1.5	17.71	33.21	141.54
P-6	4700	4250	1.11	two way	continous	1.5	17.71	33.21	127.99
P-7	5040	4700	1.07	two way	end span	1.3	17.71	28.78	163.31
P-8	4700	4150	1.13	two way	end span	1.3	17.71	28.78	144.20
P-9	5040	3450	1.46	two way	continous	1.5	17.71	33.21	103.90
P-10	4250	3450	1.23	two way	end span	1.3	17.71	28.78	119.88
P-11	5040	3450	1.46	two way	continous	1.5	17.71	33.21	103.90
P-12	4150	3450	1.20	two way	end span	1.3	17.71	28.78	119.88
P-13	5040	3450	1.46	two way	end span	1.3	17.71	28.78	119.88
C-1	3710	1600	2.32	cantiliver	cantiliver	0.4	17.71	8.86	180.69
C-2	5040	1340	3.76	cantiliver	cantiliver	0.4	17.71	8.86	151.33
C-3	5040	1340	3.76	cantiliver	cantiliver	0.4	17.71	8.86	151.33
Governing panel depth(mm)									181 mm
clear cover(mm)									22 mm
Assuming reinforcements ϕ									12 mm
D(mm)=d+c+(ϕ /2)									209 mm
use D minimum									210 mm

F=factor of safety $F1 = \frac{500}{f_{yk}} = 1.25$ $F2=1, F3=1$

3.2.3. Determination of loading

When we determine the design load we use the load combination $Pd = 1.35Gk + 1.5Qk$

The imposed or live loads on the floor based on its specific use is derived from (ES EN 1992-1-1, 2015) shown in the table below

Table 3-3 Live loads of functional categories in building

Specific use of floor	Live load KN/m ²
Bedroom	2
Kitchen	2

Living room	2
Balconies	4
Corridor	4
Stair	4
Toilet	2

Dead loads include self-weight of slab, floor finish, ceiling plaster and partition walls. The unit weight of each material shown in the table below as per (ES EN 1992-1-1, 2015)

Table 3-4 Unit weight of materials used

material	Unit weight
Reinforced Concrete	25
HCB	14
Terrazzo tiles	23
glass	25
Cement screed	23

For typical 1st -4th floor

P-1 & P-4

Panel 1 and 4 have similar loading parameters. So, we calculate design load for both panels ones.

$$P_d = 1.35DL + 1.5 LL$$

DL= self-weight of slab + floor finish + ceiling plaster + partition load

Self-weight of slab = thickness of slab*unit weight = $0.21\text{m} \times 25\text{KN/m}^3 = 5.25\text{KN/m}^2$

Floor finish = thickness * unit weight

For this panel our floor finish is cement screed with unit weight of 23KN/m^3

Floor finish = $0.03\text{m} \times 23\text{KN/m}^3 = 0.69\text{KN/m}^2$

Celling plaster = thickness * unit weight

Celling plaster = $0.03\text{m} \times 23\text{KN/m}^3 = 0.69\text{KN/m}^2$

Partition load: - the partition load on the slab acts at wall thickness but for design purpose the load will be distributed uniformly over whole panel. Steps to distribute the partition load described below

1. Calculate area of slab

$$A=L*W= 5.705\text{m} \times 4.15\text{m} = 23.655\text{m}^2$$

2. Calculate volume of partition

$$V=L*W*H= 8.9\text{m} \times 0.15\text{m} \times 2.88\text{m} = 2.56\text{m}^3$$

3. Calculate load

$$DL= \text{unit weight of HCB} * A/V = 14 \text{ KN/m} \times 23.655\text{m}^2 / 2.56\text{m}^3 = 1.51 \text{ KN/m}^2$$

$$\text{Total DL} = 5.25\text{KN/m}^2 + 0.69\text{KN/m}^2 + 0.69\text{KN/m}^2 + 1.51 \text{ KN/m}^2 = 8.15 \text{ KN/m}^2$$

Live load: - live load is selected with respect to functional use of panel

For this panel $LL=2 \text{ KN/m}^2$

$$PD = 1.35 \times 8.15 \text{ KN/m}^2 + 1.5 \times 2 \text{ KN/m}^2 = 13.998 \text{ KN/m}^2$$

Dead load calculation for other panels summarized in [Appendix C-1]

3.2.4. Analysis of individual panel moments

First, we select which method of analysis is best for each span.

- Coefficient method of analysis can be used if the span is rectangular span with no large openings, supported by beam in all edges, has uniformly distributed load, $Q_k/G_k < 1.25$ and $Q_k \leq 5$. According to those criteria without cantilever slabs all panels can be analyzed by coefficient method.

$$M_i = \alpha_i P_d L_x^2$$

Where, M_i = the design moment per unit at the point of reference

α_i = moment coefficients [Appendix B,]

P_d = design load

L_x = length of shorter span

- Strip method is a design method based on lower bound theory of plasticity approach base on satisfaction of equilibrium requirements everywhere in the slab and a moment field is first determined that fulfills equilibrium requirement, after which the reinforcement in the slab at each point is designed for the moment. For this project strip method is used for spans can't analyzed by coefficient method.

From moment analysis we can get support and field moments of each panel as shown in fig below.

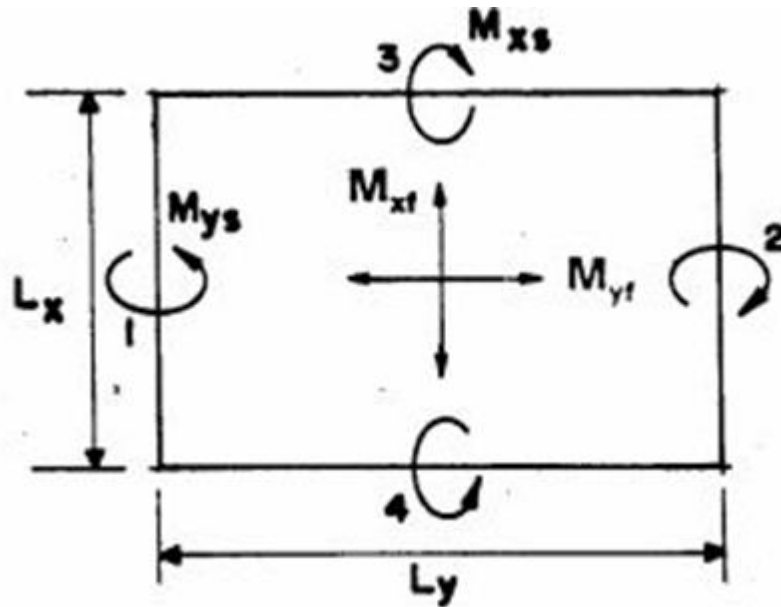


Figure 3-3 Moment direction on panels

Sample calculation

$$M_i = \alpha_i P_d L_x^2$$

$$M_i = 0.073 * 14 * 4.15^2 = 17.6$$

$$M_i = 0.054 * 14 * 4.15^2 = 13.02$$

$$M_i = 0.047 * 14 * 4.15^2 = 11.33$$

$$M_i = 0.036 * 14 * 4.15^2 = 8.68$$

The moment analysis calculation for the rest panels found in [Appendix C]

Moment analysis for cantilever slabs

C1

$$M_{xs} = WL^2/2 = 11.95 * 2^2/2 = 23.9 \text{ KN/m}^2$$

$$RA = WL = 11.95 * 2 = 23.9 \text{ KN}$$

C2 & C3

$$M_{xs} = WL^2/2 = 11.95 * 1.33^2/2 = 10.57 \text{ KN/m}^2$$

$$RA=WL=11.95*1.33= 15.89 \text{ KN}$$

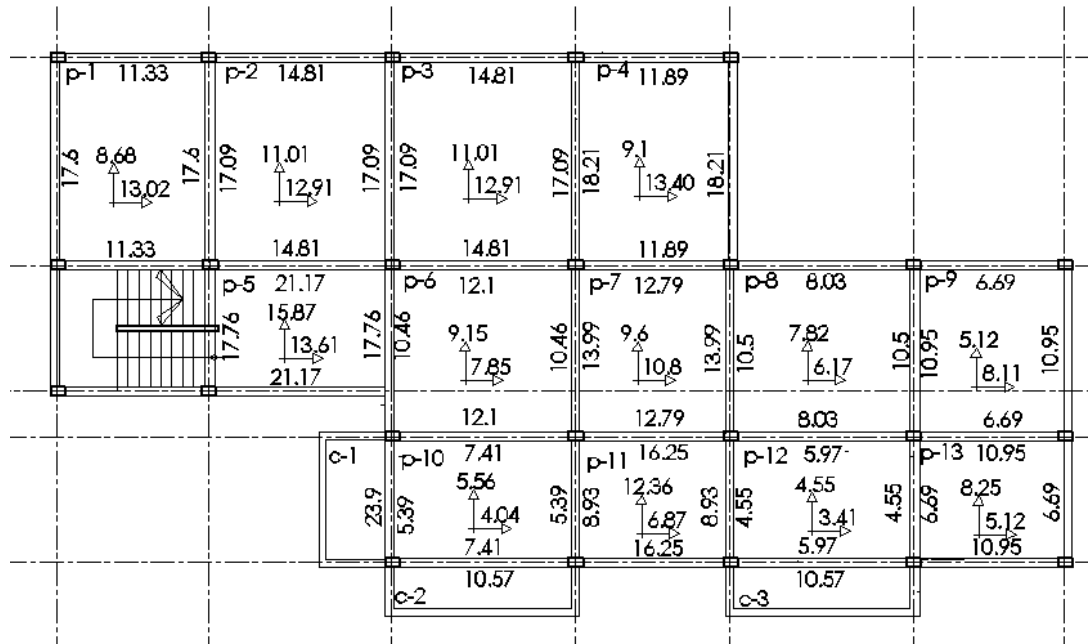


Figure 3-4 Moment analysis result for typical 1st -4th floor

Support moment adjustment

Two methods of moment adjustment are used

- i) Method-1: if $\Delta M_{sup} < 20\% M_{large}$, use simple arithmetic averaging for the design moment of the supports without further field moment adjustment.

✓ Sample calculation

Panel P-1 and P-2 –on axis 2 adjustment is required

$$\Delta M_s = 17.6 - 17.09 = 0.51 \text{ KNm/m}$$

$$0.2M_{s,max} = 0.2 \times 17.6 = 3.52 \text{ KNm/m}$$

$$\Delta M_s = 0.51 < 0.2M_{s,max} = \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (17.6 + 17.09) = 17.35 \text{ KNm/m}$$

- j) Method-2: if $\Delta M_{sup} > 20\% M_{large}$, moments are balance based on the stiffness of the slab and

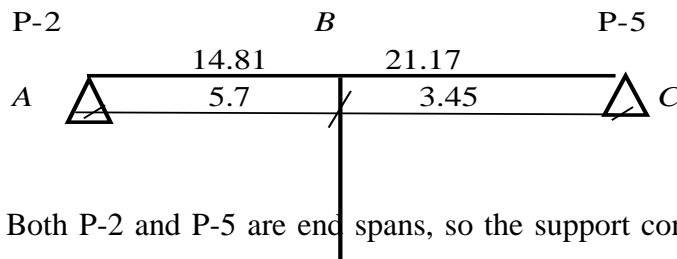
the effective length by using moment distribution method.

- **Moment distribution method:** -the method begins by assuming each joint of a structure is fixed. Then, by unlocking and locking each joint in succession, the internal moments at the joints are “distributed” and balanced until the joints have rotated to their final or nearly final positions. Generally, we distribute the analyzed support moment by multiplying with distribution factor according to support condition and span length.

✓ Sample calculation

Panel P-2 and P-5 –on axis B adjustment is required.

$$\begin{aligned} \Delta M_s &= 21.17 - 14.81 = 6.36 \\ 0.2M_{s,max} &= 0.2 \times 21.17 = 4.234 \\ \Delta M_s &= 6.36 < 0.2M_{s,max} \quad \text{FALSE} \\ &\text{moment distribution method required} \end{aligned}$$



Both P-2 and P-5 are end spans, so the support condition at far ends considered pin support.

Relative bending stiffness K

$$K = \begin{cases} \frac{I}{L} & \text{if far end of member is fixed} \\ \frac{3I}{4L} & \text{if far end of member hinged} \end{cases}$$

Distribution factor Df

$$Df = \frac{k}{\sum k} \text{ (Kassimali, 2011)}$$

$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 5.7} = 0.13 \quad I$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 3.45} = 0.22 \quad I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.377$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.623$$

$\Delta M = 21.17 - 14.81 = 6.36$, so 6.36 is distributed over two panels

For P1

$$0.377 \times 6.36 = 2.39772$$

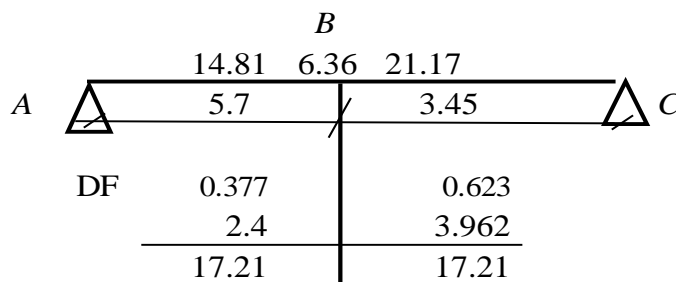
$$14.81 + 2.39772 = 17.21$$

For P2

$$0.623 \times 6.36 = 3.96228$$

$$21.17 - 3.96228 = 17.21$$

17.21 = 17.21, support moment balanced



Support moment adjustment for other panels summarized in [appendix C-2]

3.2.5. Field moment adjustment

P-1

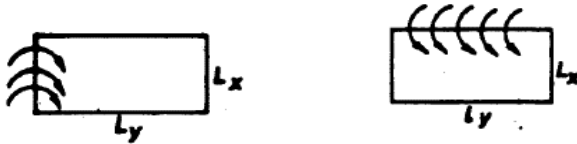
$$M_{xf} = 13.02 \quad M_{xs} = 17.6 \quad M_{xs,adj} = 17.35$$

$$M_{yf} = 8.68 \quad M_{ys} = 11.33 \quad M_{ys,adj} = 11.33$$

$$L_y/L_x = 1.37$$

$$\Delta M_{xs} = M_{xs} - M_{xs,adj} = 0.25$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 0.00 \quad \text{neglect}$$



$$C_x = 0.318 \quad C_x = 0.391$$

$$C_y = 0.118 \quad C_y = 0.337$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.098$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{13.118}$$

$$\Delta M_{yf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.084$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{8.764}$$

Value of C_x and C_y is form

Field moment adjustment for other panels is summarized in [Appendix C-3]

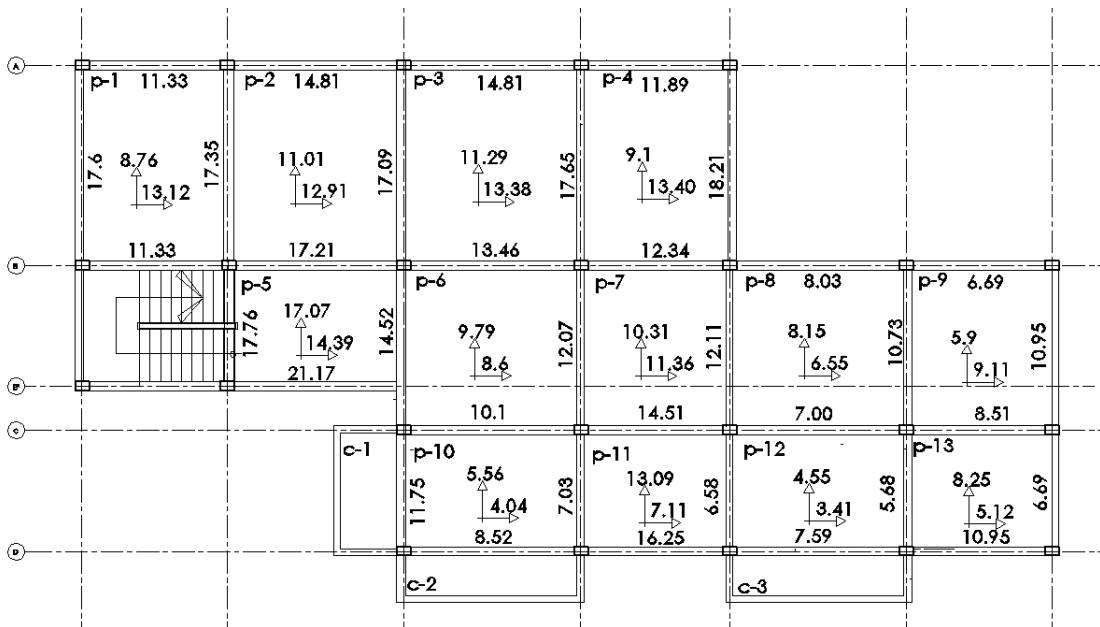


Figure 3-5 Adjusted moment for typical 1st -4th floor

3.2.6. Check depth of flexure

$$N = \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}} \left(\frac{\rho_o}{\rho} \right)^{\frac{3}{2}} \right] \text{ if } \rho \leq \rho_o$$

$$A_{st.pro} = \frac{bd}{S_{Pr0}} \quad A_{st.pro} = \frac{1000*181}{300} = 603.33$$

$$\rho = \frac{A_{st.pro}}{bd} \quad \rho = \frac{603.33}{1000*181} = 0.0033$$

$$\rho_o = \frac{\sqrt{f_{ck}}}{1000} = \frac{\sqrt{20}}{1000} = 0.00447$$

$$N = \left[11 + 1.5\sqrt{20} \frac{0.00447}{0.0033} + 3.2\sqrt{20} \left(\frac{0.00447}{0.0033} \right)^{\frac{3}{2}} \right] \text{ if } \rho \leq \rho_o$$

$$N = 42.6474$$

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$F1 = \frac{500}{\left(f_{yk} * \frac{As_{cal}}{As_{prov}} \right)} = \frac{500}{\left(400 * \frac{373.62}{603.33} \right)} = 2.0185$$

$$\frac{4700}{d} = 1.3 * 42.6474 * 2.0185 * 1 * 1$$

$$d = 41.9984$$

$$D = 41.998 + 22\text{mm} + 10/2 = 68.998\text{mm} < D \text{ used} = 210\text{mm OK !!}$$

3.2.7. Shear force calculation

Two-way slab shear

The shear force for each panel calculated by coefficient method

$$V_x = \beta_{vx} P d L_x$$

$$V_y = \beta_{vy} P d L_y$$

β_{vx} and β_{vy} are coefficients based on L_y/L_x ration and support condition [Appendix B,]

Sample calculation for P-1

$$\beta_{vxc} = 0.51 \quad \beta_{vxd} = 0.34$$

$$\beta_{vyc}=0.4 \quad \beta_{vyd}=0.26$$

$$V_{Xc} = \beta_{vxc} PdLx = 0.51 * 7.383 * 4.15 = 15.626$$

$$V_{Xd} = \beta_{vxd} PdLx = 0.34 * 7.383 * 4.15 = 10.417$$

$$V_{yc} = \beta_{vyc} PdLy = 0.4 * 7.383 * 4.15 = 12.256$$

$$V_{yd} = \beta_{vyd} PdLy = 0.26 * 7.383 * 4.15 = 7.966$$

The loads of slab transferred in to beam 90% across the entire length and 100% across 0.75 span length.

Shear force calculation for other panels found in [Appendix C]

3.2.8. Design slab for shear

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{ck})^{\frac{1}{3}} \right] bd \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d} \right)^{0.5} \leq 2$$

$$d = 181$$

$$k = 2.05 \quad \text{not} < 2, \text{so } k = 2$$

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

Taking minimum reinforcement ϕ 12
@ 430

$$\rho_1 = 0.0015$$

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

$$V_{min} = 0.443$$

$$v_{Rd,c} = 0.342 \leq V_{min} \quad \text{so, } v_{Rd,c} = 0.443$$

maximum shear force transferred to beams is

$$V_i = 25.377 \quad \text{KN/m} \\ = 0.025 \quad \text{KN/mm}$$

0.342 > 0.025, slab thickness is sufficient to support design load

3.2.9. Reinforcement

➤ Area of steel

$$A_{st,min} = \max \left\{ \begin{array}{l} 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \frac{2.2}{400} 1000 * 181 = 259 \text{ mm}^2 \\ 0.0013bd = 0.0013 * 1000 * 181 = 235 \text{ mm}^2 \end{array} \right.$$

$$A_{st,min} = \mathbf{259 \text{ mm}^2}$$

➤ Spacing

$$S_{max} = \max \left\{ \begin{array}{l} 3D = 3 * 210 = 630mm \\ 400mm \end{array} \right.$$

$$S_{max} = \mathbf{630 \text{ mm}}$$

$$A_{st} = \frac{m_{sd}}{Z * f_{yd}}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400Mpa}{1.15} = 347.83 \text{ Mpa}$$

$$Z = 0.9d = 0.9 * 181mm = 162.9 \text{ mm}$$

Example for panel 1 support moment = 17.35 KN-m

$$A_{st} = \frac{m_{sd}}{Z * f_{yd}} = \frac{17.35 * 1000 * 1000N-mm}{189mm * 347.83N/mm^2} = \mathbf{306.21 \text{ mm}^2} > \mathbf{259 \text{ mm}^2} \text{ OK!!}$$

Let the ϕ of reinforcement be 12mm

$$a_{st} = \frac{\pi d^2}{4} = \frac{3.14 * 12^2}{4} = \mathbf{113.04 \text{ mm}^2}$$

$$\text{Spacing of bars} = S = \frac{b a_s}{A_s} = \frac{1000 * 113.04}{306.21} = \mathbf{369.16 \text{ mm}}$$

369.16 mm \approx 360 mm used < 630mm OK!!

Reinforcement calculation for other panels found in [Appendix C]

➤ Lap length

The detailing of laps between bars shall be such that:

- the transmission of the forces from one bar to the next is assured.
- spalling of the concrete in the neighborhood of the joints does not occur;
- large cracks which affect the performance of the structure do not occur. The detailing of laps between bars shall be such that:

- The transmission of the forces from one bar to the next is assured;
- spalling of the concrete in the neighborhood of the joints does not occur;
- large cracks which affect the performance of the structure do not occur.
- between bars should normally be staggered and not located in areas of high moments /forces (e.g. plastic hinges).
- at any section should normally be arranged symmetrically.

The arrangement of lapped bars should comply with many laps

- the clear distance between lapped bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4ϕ or 50 mm;
- the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_0 ;
- In case of adjacent laps, the clear distance between adjacent bars should not be less than 20mm

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min} \text{ (ES EN 1992-1-1, 2015)}$$

$$l_{0,min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\phi \\ 200\text{mm} \end{cases}$$

$$\text{Where: } l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}}$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s}$$

$$f_{bd} = 2.25\eta_1\eta_2f_{ctd}$$

$$\eta_1 = 1 \text{ for good quality of bond}$$

$$\eta_1 = 0.7 \text{ for other cases}$$

$$\eta_2 = 1 \text{ for } \phi \leq 32\text{mm (ES EN 1992-1-1, 2015) Art, 8.4.2}$$

$$\eta_2 = \frac{132-\phi}{100} \text{ for } \phi > 32\text{mm}$$

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

Sample calculation for $\phi 12$ reinforcement bar

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

$$f_{ctk} = 1.5 \text{ mpa, from ESEN 1992 Table 3.1}$$

$$f_{ctd} = \frac{0.85 * 1.5}{1.5} = \mathbf{0.85}$$

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd}$$

$$\eta_1 = 1, \text{ our slab D is } < 250\text{mm, good condition of bond}$$

$$\eta_2 = 1, \text{ for } \phi \leq 32\text{mm (ES EN 1992-1-1, 2015) Art, 8.4.2}$$

$$f_{ctd} = 0.85$$

$$f_{bd} = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$$

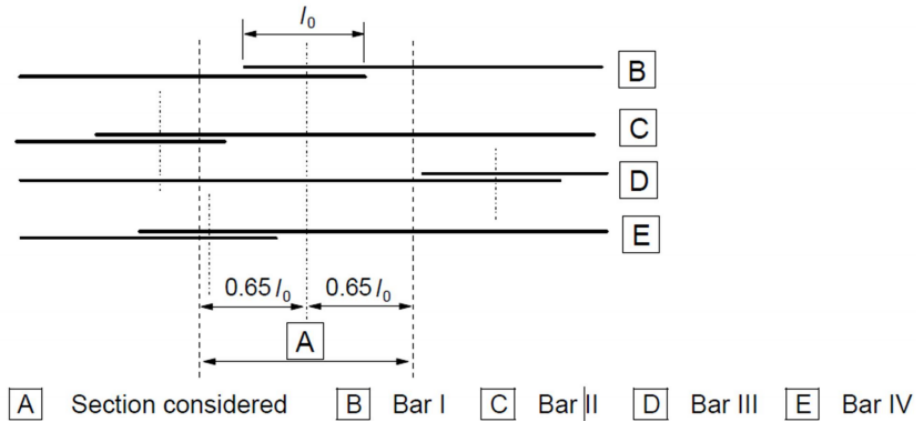
$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = \mathbf{347.83}$$

$$l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}} = \frac{12}{4} * \frac{347.83}{1.91} = \mathbf{546.33\text{mm}}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

$$\alpha_1, \alpha_2 \text{ \& } \alpha_3 = 1, \alpha_4 = 0.7, \text{ for reinforcement bar in compression}$$

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within $0.65 l_0$ from the center of the lap length considered



Example: Bars II and III are outside the section being considered: % = 50 and $\alpha_6 = 1.4$

Figure 8.8: Percentage of lapped bars in one lapped section

Let all reinforcement in slab lapped on the same center, therefore $\alpha_6 = 1.5$ from the table value for $\rho_1 > 50\%$,

$$l_{0,\min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.5 * 546.33 = 245.85\text{mm} \\ 15\phi = 15 * 12 = 180\text{mm} \\ 200\text{mm} \end{cases}$$

$$l_{0,\min} = 245.85\text{mm}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,\min}$$

$$l_0 = 1 * 1 * 1 * 1.5 * 546.33\text{mm} = 819.495 \geq 245.85\text{mm}$$

lap length, $l_0 = 820\text{mm}$

➤ Anchorage length

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,\min}$$

for anchorages in tension

$$l_{b,\min} \geq \max \begin{cases} 0.3l_{b,rqd} \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} \geq \max \begin{cases} 0.3 * 546.33 = 163.9 \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = \mathbf{163.9\text{mm}}$$

$\alpha_1 \alpha_2 \alpha_3 \alpha_5$ coefficients [appendix B]

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

$$l_{bd} = 1 * 1 * 1 * 546.33\text{mm} = \mathbf{546.33\text{mm} \geq 163.9\text{mm}}$$

for anchorages in compression

$$l_{b,min} \geq \max \begin{cases} 0.6l_{b,rqd} \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} \geq \max \begin{cases} 0.6 * 546.33 = 327.8 \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = \mathbf{327.8\text{mm}}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min} = 1 * 1 * 1 * 546.33\text{mm} = \mathbf{546.33\text{mm} \geq 327.8\text{mm}}$$

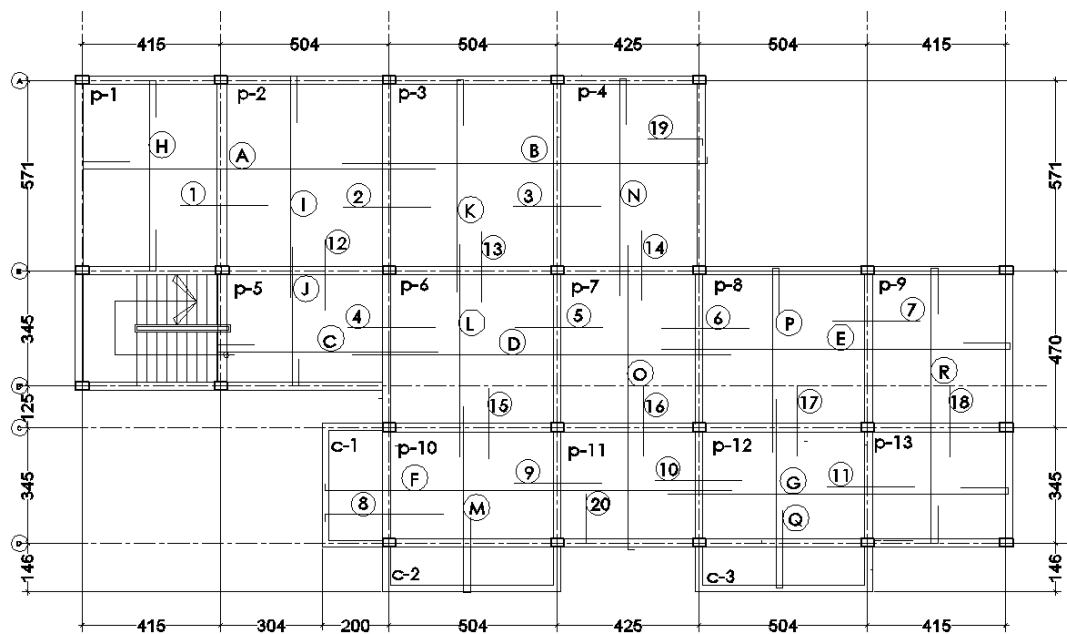


Figure 3-6 Reinforcement layout for typical 1st - 4th floor

Table 3-5 Legend for reinforcement layout of typical 1st-4th floor

label	Reinforcement	label	Reinforcement
A	L= 1158 ϕ 12 c/c 430	1	L= 307 ϕ 12 c/c 360
B	L= 1029 ϕ 12 c/c 430	2	L= 336 ϕ 12 c/c 370
C	L= 772 ϕ 12 c/c 430	3	L= 310 ϕ 12 c/c 360
D	L= 1093 ϕ 12 c/c 430	4	L= 336 ϕ 12 c/c 430
E	L= 1158 ϕ 12 c/c 430	5	L= 310 ϕ 12 c/c 430
F	L= 950 ϕ 12 c/c 430	6	L= 310 ϕ 12 c/c 430
G	L= 1158 ϕ 12 c/c 430	7	L= 306 ϕ 12 c/c 430
H	L= 987 ϕ 12 c/c 430	8	L= 386 ϕ 12 c/c 430
I	L= 862 ϕ 12 c/c 430	9	L= 310 ϕ 12 c/c 430
J	L= 560 ϕ 12 c/c 300	10	L= 310 ϕ 12 c/c 430
K	L= 862 ϕ 12 c/c 430	11	L= 306 ϕ 12 c/c 430
L	L= 634 ϕ 12 c/c 430	12	L= 305 ϕ 12 c/c 360
M	L= 852 ϕ 12 c/c 430	13	L= 347 ϕ 12 c/c 430
N	L= 862 ϕ 12 c/c 430	14	L= 347 ϕ 12 c/c 430
O	L= 915 ϕ 12 c/c 430	15	L= 272 ϕ 12 c/c 430
P	L= 727 ϕ 12 c/c 430	16	L= 272 ϕ 12 c/c 430
Q	L= 852 ϕ 12 c/c 430	17	L= 272 ϕ 12 c/c 430
R	L= 1123 ϕ 12 c/c 430	18	L= 272 ϕ 12 c/c 430
		19	L= 160 ϕ 12 c/c 350
		20	L= 133 ϕ 12 c/c 390

3.3. Analysis and design of roof slab

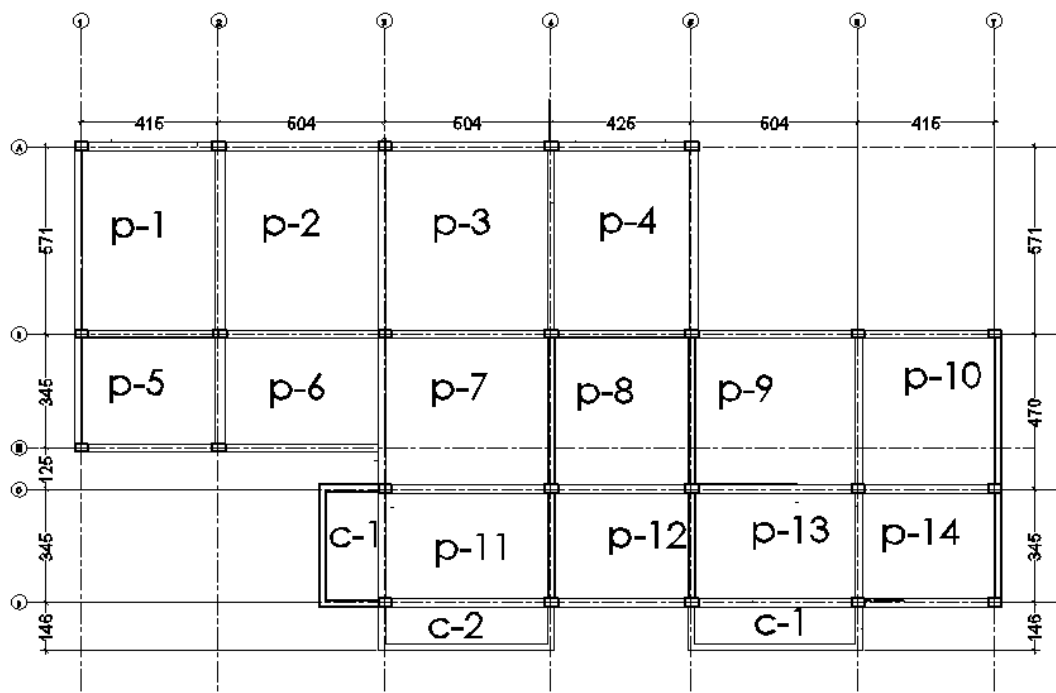


Figure 3-7 Roof slab layout

3.3.1. Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

- a. Exposure class = XC4, for cyclic wet and dry, concrete surfaces subject to water contact, not within exposure class XC2. [Appendix B,]
- b. Minimum strength class = C30/37, for exposure class XC4 [Appendix B,]
- c. Minimum concrete cover for bond/durability

$$C_{min} = \max \begin{Bmatrix} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{Bmatrix} = \max \begin{Bmatrix} 10mm \\ 15mm \\ 10mm \end{Bmatrix} = 15mm$$

$C_{min, b} = 10$ mm, assumed diameter of bar [Appendix B]

$C_{min, dur} = 15$ mm,

The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N(*ES EN 1992-1-1*, 2015). The recommended minimum Structural Class is S1

$C_{min} = 15$ mm

- d. Nominal cover

$$C_{nom} = C_{min} + \delta C_{dev} = 15mm + 10mm = 25mm$$

$\Delta C_{dev} = 10$ mm, the value of ΔC_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

- e) Minimum cover Design for Fire = 20mm
- f) Governing concrete cover = 25mm

3.3.2. Depth determination

We use C30/37 concrete and S400 rebar

Serviceability requirement

$$\frac{l}{d} = N * K * F1 * F2 * F3$$

$$N = 11 + 1.5\sqrt{f_{ck}} = 11 + 1.5\sqrt{30} = 19.22$$

K= Coefficient factor for different structural systems =1.3 for end span, 1.5 for interior span and 0.4 for cantilever slab.

Table 3-6 Depth determination for roof slab

Panel	ly (mm)	lx (mm)	ly/lx	Type	structural system	k	N	lx/d	d
P-1	5710	4150	1.38	two way	end span	1.3	19.22	31.23	132.87
P-2	5710	5040	1.13	two way	end span	1.3	19.22	31.23	161.37
P-3	5710	5040	1.13	two way	end span	1.3	19.22	31.23	161.37
P-4	5710	4250	1.34	two way	end span	1.3	19.22	31.23	136.08
P-5	4150	3450	1.20	two way	end span	1.3	19.22	31.23	110.46
P-6	5040	3450	1.46	two way	end span	1.3	19.22	31.23	110.46
P-7	5040	4700	1.07	two way	continous	1.5	19.22	36.04	130.42
P-8	4700	4250	1.11	two way	continous	1.5	19.22	36.04	117.93
P-9	5040	4700	1.07	two way	end span	1.3	19.22	31.23	150.48
P-10	4700	4150	1.13	two way	end span	1.3	19.22	31.23	132.87
P-11	5040	3450	1.46	two way	continous	1.5	19.22	36.04	95.73
P-12	4250	3450	1.23	two way	end span	1.3	19.22	31.23	110.46
P-13	5040	3450	1.46	two way	continous	1.5	19.22	36.04	95.73
P-14	4150	3450	1.20	two way	end span	1.3	19.22	31.23	110.46
C-1	3710	1600	2.32	cantiliver	cantiliver	0.4	19.22	9.61	166.49
C-2	5040	1460	3.45	cantiliver	cantiliver	0.4	19.22	9.61	151.93
C-3	5040	1460	3.45	cantiliver	cantiliver	0.4	19.22	9.61	151.93
Governing panel depth(mm)									166.49 mm
clear cover(mm)									25 mm
Assuming reinforcements ϕ									10 mm
D(mm)=d+c+(ϕ /2)									196 mm
use D minimum									200 mm

$$F = \text{factor of safety } F1 = \frac{500}{f_{yk}} = 1.25 \quad F2=1, F3=1$$

3.3.3. Determination of loading

Table 3-7 Load determination for roof slab

	Material	Unit weight	Dimension	Load KN/m ²
Slab	Reinforced concrete	25	0.200	5.00
cement screed	concrete	23	0.030	0.69
ceiling plaster	concrete	23	0.030	0.69
LIVE LOAD Q _k		0.4	KN/m ²	
WIND LOAD WL		-0.588	KN/m ²	
DEAD LOAD G _k		6.38	KN/m ²	
COMB 1	Pd = 1.35 G _k + 1.5 Q _k		9.213	KN/m ²
COMB 2	Pd = 1.35 G _k + 1.5 WL		7.731	KN/m ²
COMB 3	Pd = 1.35 G _k + 1.5 Q _k + 0.9WL		8.684	KN/m ²
COMB 4	Pd = 1.35 G _k + 1.5 Q _k - 0.9WL		9.742	KN/m ²

The worst combination is COMB 4 which is **Pd = 9.742 KN/m²**

3.3.4. Analysis of individual moments

Analysis of individual moment calculation procedures described in section 3.2.4, analysis result for roof slab summarized in [Appendix -C]

Moment analysis for cantilever slabs

C1

$$M_{xs} = WL^2/2 = 9.742 * 2^2/2 = 19.484 \text{ KN/m}^2$$

$$RA=WL=9.742*2=19.484 \text{ KN}$$

C2 & C3

$$M_{xs} = WL^2/2 = 9.742 * 1.46^2/2 = 10.38 \text{ KN/m}^2$$

$$RA=WL=9.742*1.46= 14.223 \text{ KN}$$

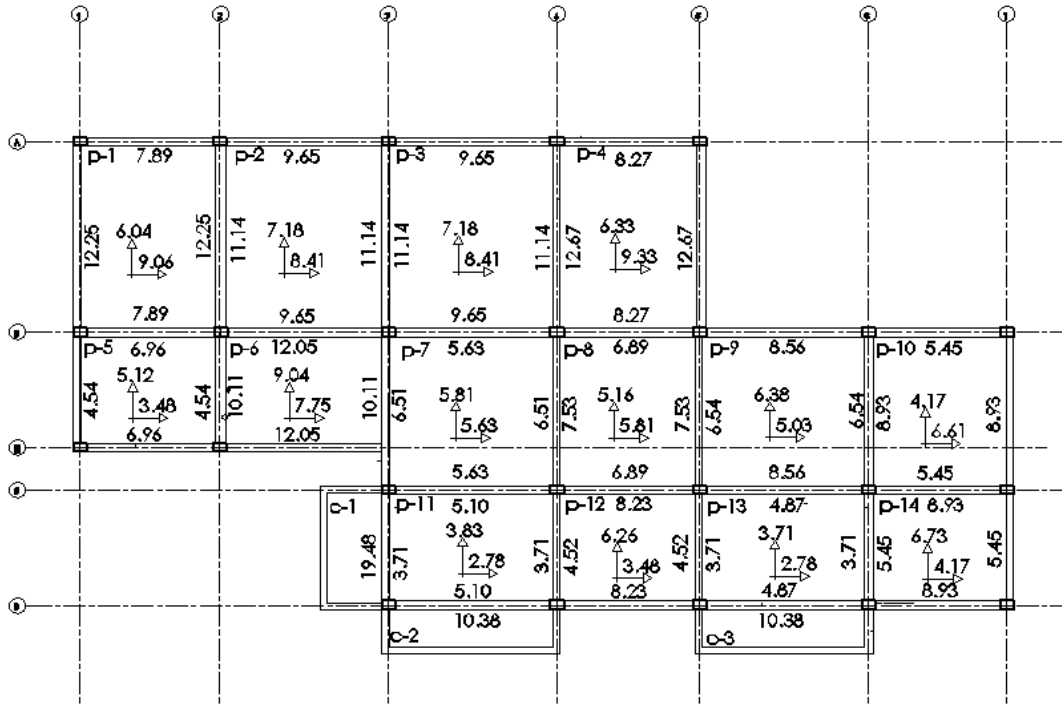


Figure 3-8 Analysis of individual moment for roof slab

3.3.5. Support moment adjustment

The procedures to adjust support moment is described in section 3.2.5. support moment adjustment for this section summarized in [Appendix C]

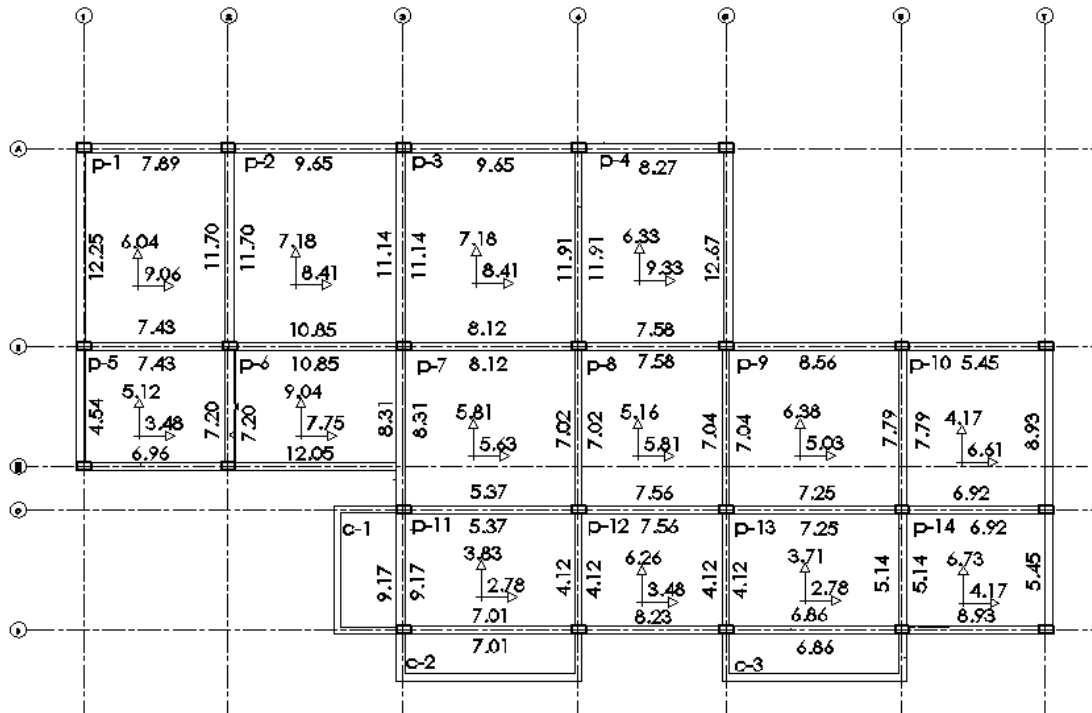


Figure 3-9 Adjusted support moment for roof slab

3.3.6. Field moment adjustment

P-1

$$\begin{aligned}
 M_{xf} &= 9.06 & M_{Xs} &= 12.25 & M_{Xs,adj} &= 12.25 \\
 M_{yf} &= 6.04 & M_{ys} &= 7.89 & M_{ys,adj} &= 7.89 \\
 Ly/Lx &= 1.46
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = 0.00 \text{ neglect}$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 0.00 \text{ neglect}$$

No adjustment of field moment required,

Field moment adjustment for other panels summarized in [Appendix C]

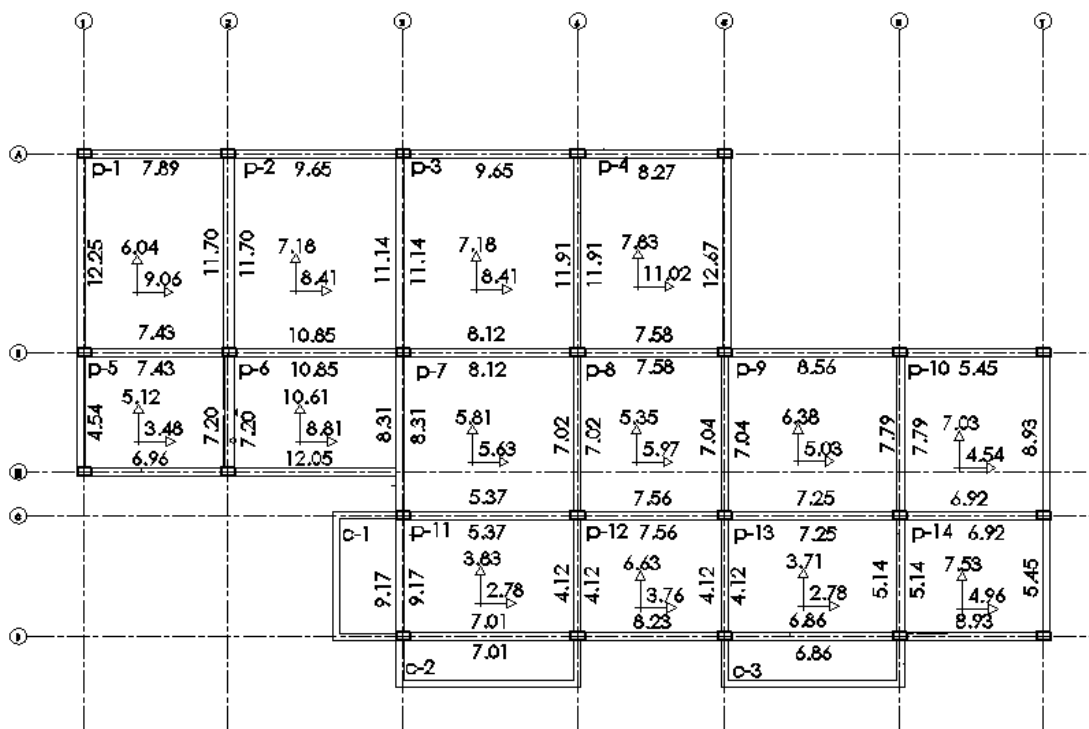


Figure 3-10 Adjusted moment for roof slab

3.3.7. Check depth of flexure

$$N = \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}} \left(\frac{\rho_o}{\rho} \right)^{\frac{3}{2}} \right] \text{ if } \rho \leq \rho_o$$

$$A_{st.pro} = \frac{bd}{S_{Pro}} \quad A_{st.pro} = \frac{1000 * 166.49}{320} = 520.28$$

$$\rho = \frac{A_{st.pro}}{bd} \quad \rho = \frac{520.28}{1000 * 166.49} = 0.0031$$

$$\rho_o = \frac{\sqrt{f_{ck}}}{1000} = \frac{\sqrt{20}}{1000} = 0.00447$$

$$N = \left[11 + 1.5\sqrt{20} \frac{0.00447}{0.0031} + 3.2\sqrt{20} \left(\frac{0.00447}{0.0031} \right)^{\frac{3}{2}} \right] \text{ if } \rho \leq \rho_o$$

$$N = 45.4518$$

$$\frac{l}{d} = K * N * F1 * F2 * F3$$

$$F1 = \frac{500}{\left(f_{yk} * \frac{As_{cal}}{As_{prov}} \right)} = \frac{500}{\left(400 * \frac{238.24}{520.28} \right)} = 2.73$$

$$\frac{4250}{d} = 1.3 * 45.452 * 2.73 * 1 * 1$$

$$d = 26.3469$$

$$D = 26.35 + 22\text{mm} + 10/2 = 53.35 \text{ mm} < D \text{ used} = 200\text{mm OK !!}$$

3.3.8. Shear force calculation

Shear force calculation procedures are described in section 3.2.7, for roof slab shear force results are found in [Appendix – C]

3.3.9. Design slab for shear

$$v_{Rd,c} = \left[0.12K(100\rho_1 f_{CK})^{\frac{1}{3}} \right] bd \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d} \right)^{0.5} \leq 2$$

$$d = 166.6$$

$$k = 2.10 \quad \text{not} < 2, \text{so } k = 2$$

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

Taking minimum reinforcement ϕ 10
@ 430

$$\rho_1 = 0.0011$$

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

$$V_{min} = 0.443$$

$$v_{Rd,c} = 0.312 \leq V_{min} \quad \text{so,} \quad v_{Rd,c} = 0.443$$

maximum shear force transfered to beams is

$$\begin{aligned} V_i &= 23.35 \quad \text{KN/m} \\ &= 0.023 \quad \text{KN/mm} \end{aligned}$$

0.312 > 0.023, slab thickness is sufficient to support design load

3.3.10. Reinforcement

Area of steel

$$A_{st,min} = \max \left\{ \begin{array}{l} 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \frac{2.2}{400} 1000 * 166.6 = 238.24 \text{ mm}^2 \\ 0.0013bd = 0.0013 * 1000 * 166.6 = 216.58 \text{ mm}^2 \end{array} \right.$$

$$A_{st,min} = 238.24 \text{ mm}^2$$

Spacing

$$S_{max} = \max \left\{ \begin{array}{l} 3D = 3 * 200 = 600 \text{ mm} \\ 400 \text{ mm} \end{array} \right.$$

$$S_{max} = 600 \text{ mm}$$

$$A_{st} = \frac{m_{sd}}{Z * f_{yd}}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400 \text{ Mpa}}{1.15} = 347.83 \text{ Mpa}$$

$$Z = 0.9d = 0.9 * 166.6 \text{ mm} = 149.94 \text{ mm}$$

Example for panel 1 support moment = 12.25 KN-m

$$A_{st} = \frac{m_{sd}}{Z * f_{yd}} = \frac{12.25 * 1000 * 1000 \text{ N-mm}}{149.94 \text{ mm} * 347.83 \text{ N/mm}^2} = \mathbf{234.88 \text{ mm}^2} < \mathbf{238.24 \text{ mm}^2}$$

$$A_{st} = 238.24 \text{ mm}^2$$

Let the ϕ of reinforcement be 10mm

$$a_{st} = \frac{\pi d^2}{4} = \frac{3.14 * 10^2}{4} = \mathbf{78.5 \text{ mm}^2}$$

$$\text{Spacing of bars} = S = \frac{b a_s}{A_s} = \frac{1000 * 78.5}{238.24} = \mathbf{329.49 \text{ mm}}$$

329.49 mm \approx 320 mm used < 600 mm OK!!

Reinforcement calculation results are found on [Appendix C]

➤ Lap length

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

$$l_{0,min} \geq \max \begin{cases} 0.3 \alpha_6 l_{b,rqd} \\ 15\phi \\ 200 \text{ mm} \end{cases}$$

$$\text{Where: } l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}}$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s}$$

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd}$$

$\eta_1 = 1$ for good quality of bond

$\eta_1 = 0.7$ for other cases

$\eta_2 = 1$ for $\phi \leq 32 \text{ mm}$

$\eta_2 = \frac{132 - \phi}{100}$ for $\phi > 32 \text{ mm}$

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

Sample calculation for $\emptyset 10$ reinforcement bar

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

$$f_{ctk} = 1. \text{ mpa, from ESEN 1992 Table 3.1}$$

$$f_{ctd} = \frac{0.85 * 1.5}{1.5} = \mathbf{0.85}$$

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd}$$

$$\eta_1 = 1, \text{ our slab D is } < 250\text{mm, good condition of bond}$$

$$\eta_2 = 1, \text{ for } \emptyset \leq 32\text{mm}$$

$$f_{ctd} = 0.85$$

$$f_{bd} = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$$

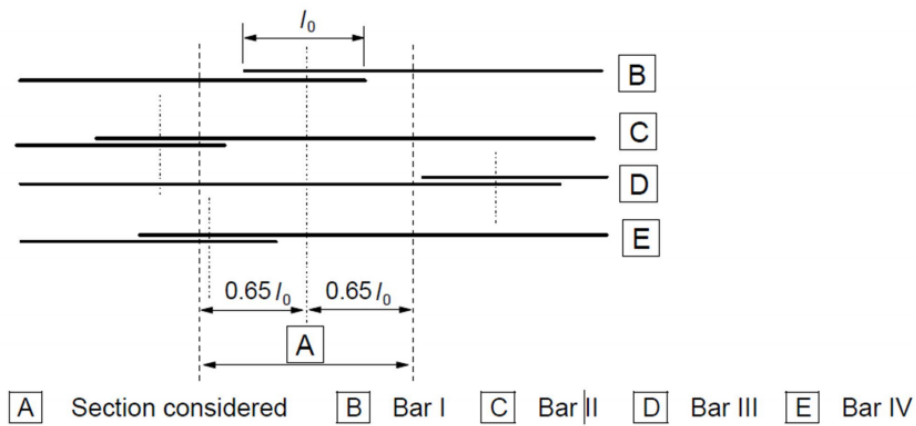
$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = \mathbf{347.83}$$

$$l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma_{sd}}{f_{bd}} = \frac{10}{4} * \frac{347.83}{1.91} = \mathbf{455.28\text{mm}}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

$$\alpha_1, \alpha_2 \ \& \ \alpha_3 = 1, \alpha_4 = 0.7, \text{ for reinforcement bar in compression}$$

$$\alpha_6 = \left(\frac{\rho_1}{25} \right)^{0.5} \quad \text{but not exceeding 1.5 nor less than 1.0, where } \rho_1 \text{ is the percentage of reinforcement lapped within } 0.65 l_0 \text{ from the center of the lap length considered}$$



Example: Bars II and III are outside the section being considered: % = 50 and $\alpha_6=1.4$

Figure 8.8: Percentage of lapped bars in one lapped section

Let all reinforcement in slab lapped on the same center ,therefore $\alpha_6 = 1.5$ from the table value for $\rho_1 > 50 \%$,

$$l_{o,min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.5 * 455.28 = 204.88\text{mm} \\ 15\phi = 15 * 10 = 150\text{mm} \\ 200\text{mm} \end{cases}$$

$$l_{o,min} = \mathbf{204.88\text{mm}}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,min}$$

$$l_0 = 1 * 1 * 1 * 1.5 * 455.28\text{mm} = 682.92 \geq 204.88\text{mm}$$

lap length, $l_0 = \mathbf{690\text{mm}}$

➤ Anchorage length

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

for anchorages in tension

$$l_{b,min} \geq \max \begin{cases} 0.3l_{b,rqd} \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} \geq \max \begin{cases} 0.3 * 455.28 = 136.58 \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 136.58 \text{ mm}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

$$l_{bd} = 1 * 1 * 1 * 455.28 \text{ mm} = 455.28 \text{ mm} \geq 136.58 \text{ mm}$$

for anchorages in compression

$$l_{b,min} \geq \max \begin{cases} 0.6 l_{b,rqd} \\ 10\phi \\ 100 \text{ mm} \end{cases}$$

$$l_{b,min} \geq \max \begin{cases} 0.6 * 455.28 = 273.17 \\ 10\phi \\ 100 \text{ mm} \end{cases}$$

$$l_{b,min} = 273.17 \text{ mm}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

$$l_{bd} = 1 * 1 * 1 * 455.28 \text{ mm} = 455.28 \text{ mm} \geq 273.17 \text{ mm}$$

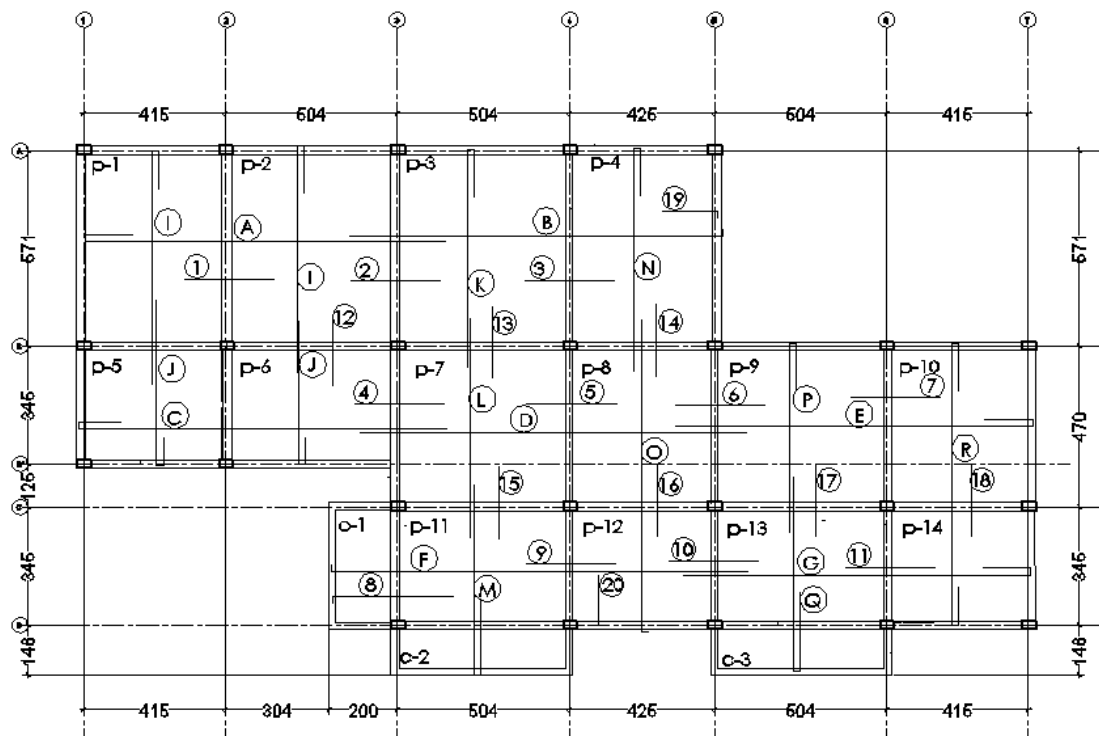


Figure 3-11 Reinforcement layout for roof slab

Table 3-8 legend for reinforcement layout of roof slab

label	reinforcement			label	reinforcement		
A	L=	1158	ø10 c/c 320	1	L=	307	ø10 c/c 320
B	L=	1029	ø10 c/c 320	2	L=	336	ø10 c/c 320
C	L=	1187	ø10 c/c 320	3	L=	310	ø10 c/c 320
D	L=	1093	ø10 c/c 320	4	L=	336	ø10 c/c 320
E	L=	1158	ø10 c/c 320	5	L=	310	ø10 c/c 320
F	L=	950	ø10 c/c 320	6	L=	310	ø10 c/c 320
G	L=	1158	ø10 c/c 320	7	L=	306	ø10 c/c 320
I	L=	862	ø10 c/c 320	8	L=	386	ø10 c/c 320
J	L=	560	ø10 c/c 320	9	L=	310	ø10 c/c 320
K	L=	862	ø10 c/c 320	10	L=	310	ø10 c/c 320
L	L=	634	ø10 c/c 320	11	L=	306	ø10 c/c 320
M	L=	852	ø10 c/c 320	12	L=	305	ø10 c/c 320
N	L=	862	ø10 c/c 320	13	L=	347	ø10 c/c 320
O	L=	915	ø10 c/c 320	14	L=	347	ø10 c/c 320
P	L=	727	ø10 c/c 320	15	L=	272	ø10 c/c 320
Q	L=	852	ø10 c/c 320	16	L=	272	ø10 c/c 320
R	L=	1123	ø10 c/c 320	17	L=	272	ø10 c/c 320
				18	L=	272	ø10 c/c 320
				19	L=	160	ø10 c/c 320
				20	L=	133	ø10 c/c 320

4. Staircase design

Stairs are structures which provide access passage to different floor levels one of the structural elements constructed with steps rising without a break from floor to floor or with steps rising to landing between floors with a series of steps rising further from the landing to floor. stairs can be straight flight with or without landing, quarter turn, half turn (also called doglegged or scissor type), open well (if well type opening exists), or it can be geometric stair (free standing stairs).

GUIDELINES AND TYPES OF STAIR IN OUR BUILDING

The type of stair and its layout is governed essentially by the available size of staircase room and positions beams and columns along the boundary of staircase.

Following are some useful guidelines in deciding the layout and type of stair:

1. The stair slab, in general are heavy compared to floor slab because of
 - i. Heavy dead load due to inclined length of slab acting over horizontal span and due to additional weight of steps,
 - ii. Greater live load in stair than on floors. Therefore, longer span for flight should be avoided as far as possible.
2. Stair flight shall be preferably being supported on beam or wall. Supporting on landing slab should be avoided as possible essentially when the span of the landing exceeds twice the width of stair, because this Cause stress concentration in the supporting landing slab at their junction.

We have one kind of stair based on their support condition

- pinned -pinned support

We have two basic ways in which stairs are planned

1. A straight flight stair, which rises from floor to floor in one direction with or without landing.

2. Open well stairs, where a space or well exists between flights

The stairs mentioned above are generally free-standing ones (simple straight flights of stairs).

Modeling the staircase

In modeling the staircase, the part that's supported on the ground and the landing part was assumed to be hinged support and the one which is monolithically casted with the slab was assumed to be fixed

DESIGN PROCEDURE

Determination of depth for deflection: which is a function of design tensile strength of steel, effective span length of the shortest span in which more load is expected to transfer and support condition

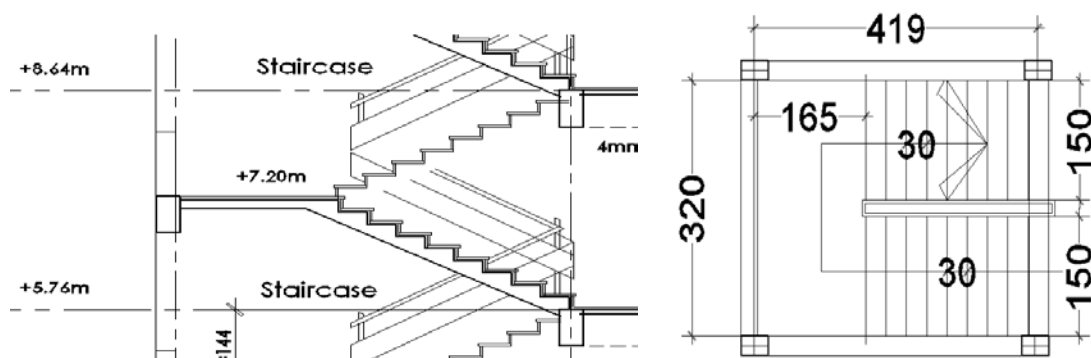
Loading: which determines the total load in the stair and landing

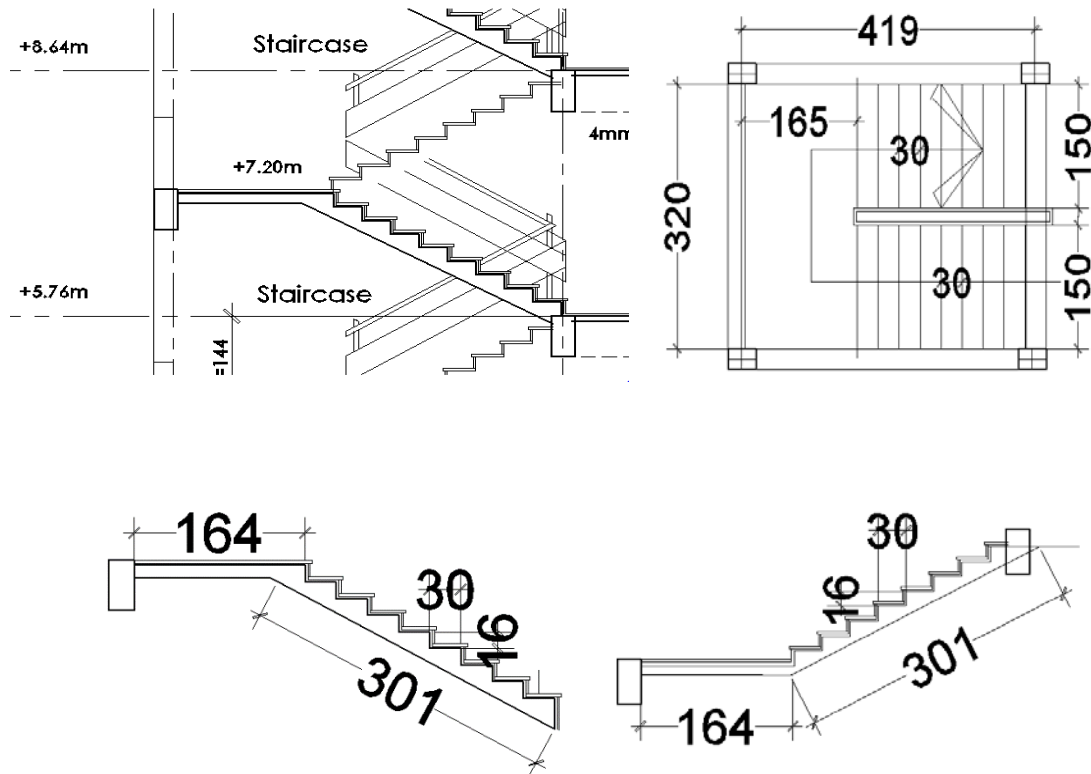
Analysis: determines moment and shear forces based on the analyzed moment

Check depth for flexure: this step helps to cross check the design depth as it is safe for flexure or not, if not revise the depth determined in step 1 and also the loads.

Reinforcement provision: using the computed moments, number and area of reinforcement bars determined.

Detailing: the arrangement of reinforcement





4.1. Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

- Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. (*ES EN 1992-1-1*, 2015)
- Minimum strength class = C20/25, for exposure class XC1
- Minimum concrete cover for bond/durability

$$d. C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{array} \right\} = \max \left\{ \begin{array}{l} 12mm \\ 10mm \\ 10mm \end{array} \right\} = 12mm$$

i. $C_{min, b} = 12 \text{ mm}$, assumed diameter of bar

ii. $C_{min, dur} = 10 \text{ mm}$,

1. $C_{min}=12\text{mm}$

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 12\text{mm} + 10\text{mm} = 22\text{mm}$

i. $\Delta c_{dev} = 10\text{mm}$, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

4.2. geometric data

height of stair in one flight (clear height between landing) = 1.44

number of riser(R) = 9

number of thread (T) = number of riser -1=9-1 =8

$$\text{height of riser} = \frac{\text{height of rasir (in one flight)}}{\text{number of raiser}} = \frac{1.44}{0.09} = 16$$

$$\text{width of thread} = \frac{\text{length of horizontal projection of inclined of one flight}}{\text{number of threads}} = 0.24\text{m} = 240\text{mm}$$

breadth of step along slope(B) G

$$B = \sqrt{(G^2 + R^2)} = \sqrt{(0.24^2 + 0.16^2)} = 0.288$$

take noising =25mm

Width of going=T-N

$$= 240 - 25 = 215\text{mm}$$

4.3. Determination of depth for deflection

According to ES EN 1992:2015; the limit state of deformation may be checked by either:

by limiting the span/depth ratio, according to 7.4.2 or

by comparing a calculated deflection, according to 7.4.3, with a limit value

$$l/d = K (11 + 1.5 \sqrt{gdl} * \rho_p / \rho + 3.2 * \sqrt{gdl} (\rho_p / \rho - 1)^{3/2}) * F1 * F2 * F3 \dots \dots \dots \text{if } \rho < \rho_p \text{ art.7.4.2. (7.16a)}$$

$$l/d = K (11 + 1.5 \sqrt{gdl} * \rho_p / (\rho - \rho') + 1/12 * \sqrt{gdl} \sqrt{(\rho_p / \rho)}) * F1 * F2 * F3 \dots \dots \dots \text{if } \rho > \rho_p \text{ art.7.4.2. (7.16b)}$$

Where, l/d - is the limit span/depth

K -is the factor to take into account the different structural systems

$$\rho_0 - \text{is the reference reinforcement ratio} = 10^{-3} \sqrt{f_{ck}}$$

ρ - is the required tension reinforcement ratio at mid -span to resist the moment due to the design loads (at support for cantilevers)

ρ' - is the required compression reinforcement ratio at mid -span to resist the moment due to design loads (at support for cantilevers) f_{yk} is in MPa

units.

$$F1 = 300 / \sigma_t = 500 / (f_{yk} * A_s, req / A_s, pro)$$

F2=0.8, for flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3. Otherwise, F2=1 for other cases.

F3=7/leff , For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections (leff in meters, see Art. 5.3.2.2 (1)). Or

$F_3=8.5/l_{eff}$, for flat slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections (leff

in meters). Otherwise, $F_3=1$ for both cases.

Basic ratios of span/effective depth for reinforced concrete members without axial compression found in [Appendix B,]

Assumption: - Slab is lightly reinforced ($\rho=0.5\%$) based on Eurocode 2 Part 1,1 - prEN 1992-1-1-2002 Section 7.4.2.

Concrete class C-25

$$f_{ck} = 20\text{MPa}$$

$$f_{cd} = 11.33\text{MPa}$$

Steel grade S-400

$$f_{yk}=347.82\text{MPa}$$

$$\rho_{s} = 10^{-3} \sqrt{f_{ck}} = 10^{-3} \sqrt{20} = 0.00447$$

For end span of one-way continuous slab

$$\frac{1}{d} = 1.3 * 11 + 1.5 * 4.47 * 0.89 = 20.26$$

$$K = 1.3 \quad \frac{1}{d} = 20.26 \text{ because we used S400 multiply the value by } \frac{385}{f_{yk}} = 0.9625$$

$$\frac{1}{d} = 20.32 * 0.9625 = 19.51, l = l_x = 3850\text{mm} \rightarrow d = 197.33 \text{ mm}$$

$$\text{Using } \varnothing 14 \text{ and cover } 15\text{mm } H = 197.33 + 15 + \frac{12}{2} = 218.33, \text{ Use } H = 250\text{mm}$$

$$d = 250 - 15 - 7 = 229\text{mm}$$

From serviceability limit state the minimum depth requirement is: -

The staircase slab is designed as one-way reinforced slab. The depth of the section for design purpose of longitudinally spanning slab is taken as the minimum thickness perpendicular to soffit of staircase. I.e. the waist (w).

For transverse staircase, there is no universal method to determine effective depth.

4.4. Loading condition on stair.

Table 4-1 Material data for stair

Material	thickness (m)	unit weight (KN/m ³)
concrete	0.21	25
Marblr	0.03	27
cement screed	0.02	23
plastering	0.02	23

Live load

The live loads for design of staircase are to be taken from table 2.10 EBCS-1 on loading standards. The value used is category A

$$q_k = 3 \text{ kN/m}^2 \text{ total live load on staircase} = 3 \text{ kN/m}^2 * 1 \text{ m} = 3 \text{ kN/m}$$

The live load given in codes assumes that it is acting along the horizontal span and this is directly used in calculation

Load computation

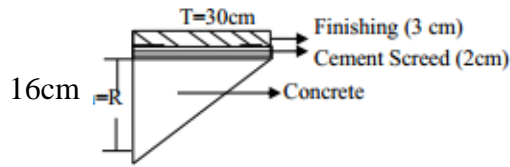
1- DL for flight (inclined slab)

The live load for staircase given in codes is the loading on horizontal projection area. Hence the dead load due to waist is calculated along the slope of the staircase and it is then converted to along the horizontal plane.

Take 1m width strip

Step dead load

Dead load on each step of length G (going) on horizontal projection is given by



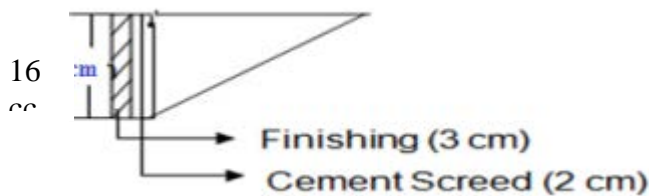
$$\text{D.L of cement screed} = t_{sc} * \delta_{sc} = 0.02 * 23 \text{ KN/m}^3 = 0.46 \text{ KN/m}$$

$$\text{D.L of finishing} = t_{fin} * \delta_{fin} = 0.03 * 27 \text{ KN/m}^3 = 0.81 \text{ KN/m}$$

$$\text{D.L of Concrete} = 1/2 * h * \delta_{conc} = 1/2 * 0.15 \text{m} * 25 \text{ KN/m}^3 = 1.875 \text{ KN/m}$$

$$\text{Therefore, DL of step} = 0.46 \text{ KN/m} + 0.81 \text{ KN/m} + 1.875 \text{ KN/m} = 3.145 \text{ KN/m}$$

Riser Dead Load

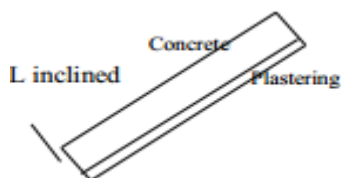


$$\begin{aligned} \text{D.L of cement screed} &= \frac{\text{Number of rasisr } (R_{cs} * t_{cs} * \gamma_{sc})}{\text{Projected length } (30 * 8 \text{cm})} \\ &= \frac{9(0.16 * 0.02 * 23 \frac{\text{KN}}{\text{m}^3})}{2.55} = 0.26 \text{ KN/m} \end{aligned}$$

$$\begin{aligned} \text{D.L of Finishing} &= \frac{\text{Number of rasisr } (b_{cs} * t_{cs} * \gamma_{sc})}{\text{Projected length } (30 * 8 \text{cm})} \\ &= \frac{9(0.16 * 0.03 * 27 \frac{\text{KN}}{\text{m}^3})}{2.55} = 0.46 \text{ KN/m} \end{aligned}$$

$$\text{Therefore D.L of riser (15cm)} = 0.26 \text{ KN/m} + 0.46 \text{ KN/m} = 0.72 \text{ KN/m}$$

Waist Dead Load





$$\tan \theta = \left(\frac{1.44}{2.63} \right) \rightarrow \theta = \tan^{-1} \left(\frac{1.44}{2.63} \right) = 28.702^\circ$$

$$\sin \theta = \left(\frac{1.44}{L_{inc}} \right) \rightarrow L_{inc} = \left(\frac{\sin}{28.702} \right) = 3\text{m}$$

$$\text{D.L of concrete} = \frac{D * (L_{inc} * \gamma_{conc})}{L_{Projected}} = \frac{0.25\text{m} * 3\text{m} * 25 \frac{\text{KN}}{\text{m}^3}}{2.63} = 7.13 \text{ KN/m}$$

$$\text{D.L of Plastering} = \frac{t_{pl} * (L_{inc} * \gamma_{pl})}{L_{Projected}} = \frac{0.03 * 3 * 23 \frac{\text{KN}}{\text{m}^3}}{2.63} = 0.525 \text{ KN/m}$$

Therefore D.L of waist = 7.13 KN/m + 0.525 KN/m = 7.655 KN/m

Total Dead Load and design load For the inclined slab (steps)

Total D.L = D.L of Step + D.L of riser + D.L of waist

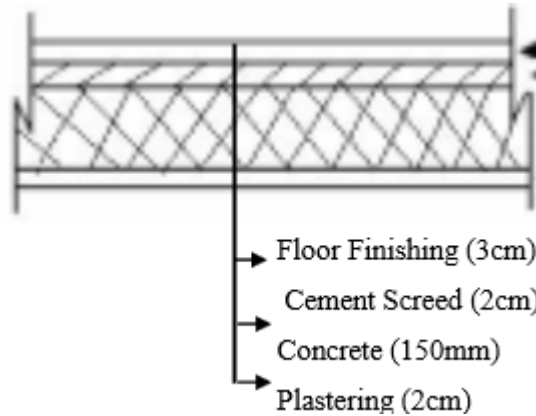
$$= 3.145 \text{ KN/m} + 0.72 \text{ KN/m} + 7.655 \text{ KN/m} = 11.52 \text{ KN/m}$$

Live load = 4 KN/m² * 1m = 3 KN/m

Design Load, Pd = 1.35 D.L + 1.5 L.L

$$Pd = 1.35(11.52) + 1.5(4)$$

21.55KN/m DL for Landing	Pd=



D.L of landing=D.L of finishing + D.L of cement screed + D.L of concrete + D.L of plastering.

$$= (t_{fin} \cdot \gamma_{fin}) + (t_{cs} \cdot \gamma_{cs}) + (t_c \cdot \gamma_c) + (t_{pl} \cdot \gamma_{pl})$$

$$= (0.03\text{m} \cdot 27 \text{ KN/m}^3) + (0.02 \cdot 23 \text{ KN/m}^3) + (0.25\text{m} \cdot 25 \text{ KN/m}^3) + (0.02\text{m} \cdot 23 \text{ KN/m}^3)$$

$$= 5.48 \text{ KN/m}$$

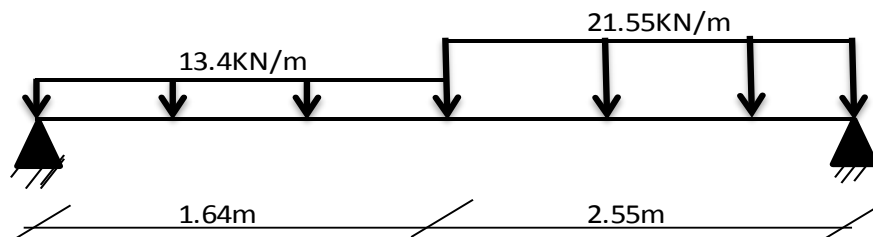
Therefore D.L of Landing=5.48 KN/m

Total Dead Load and design load for the landing

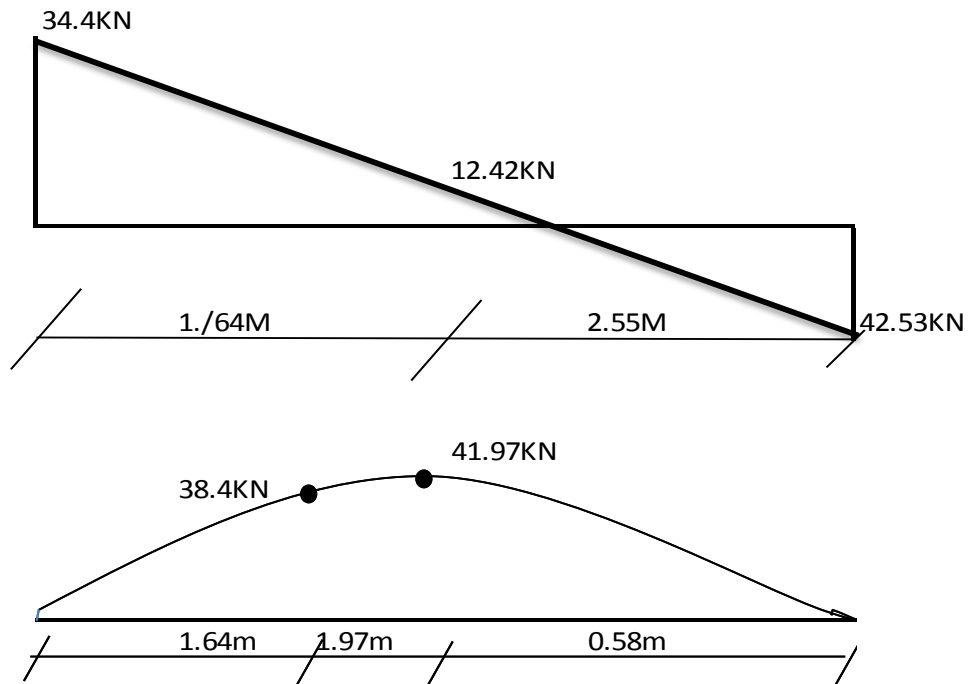
$$\text{D.L} = 5.48 \text{ KN/m} \quad \text{L.L} = 4 \text{ KN/m}^2 \cdot 1\text{m} = 3 \text{ KN/m}$$

$$\text{Design load, } P_d = 1.35(5.48) \text{ KN/m} + 1.5(3) \text{ KN/m} = 13.40 \text{ KN/m}$$

Loading



4.5. Moment Analysis



Max design moment in the span= 41.97kNm

Step-2-check depth for flexure

$$M_{rd} = 0.8kx (1 - 0.4kx) cdbd^2 = 0.8 * 0.448 (1 - 0.448 * 0.4) * 11.33 * 1000 * (229)^2$$

$$M_{rd} = 54.6 \text{ kNm/m} > 41.97 \text{ kNm/m} \dots \dots \dots \text{OK}$$

4.6. Main Reinforcement Design

For the given; Material Data, C-25, S-400

Effective depth, $d=150\text{mm}$ Width, $b=1000\text{mm}$

- Moments calculated for each panel
- Using design charts
- For span moment

Using $M_u = 41.97\text{kNm}$, calculate reinforcement, `

$$\mu_{sd} = \frac{M}{f_{cd} * b d^2} = \frac{41.77 * 10^6}{11.33 * 1000 * 229^2} = 0.071$$

Using $\mu_{sd} = 0.071$ read the $\xrightarrow{K_z} \xrightarrow{Z} \xrightarrow{A_s}$

$$k_z=0.945$$

$$Z=K_z*d =0.945*229 = 216.405\text{mm}$$

$$A_s=M/(f_yd*z) \quad A_s = 41.97*10^6/(216.405*347.82)=557.593\text{mm}^2$$

To calculate spacing by selecting diameter of bar as

$$S=1000a_s/A_s \quad \text{where: - } a_s=\text{area of single bar as}$$

$$A_s=\text{calculated area of steel} \quad S=\text{spacing}$$

$$S = 1000 * 113.04/557.573=202.74\text{mm}$$

Use, $\phi 12$ c/c200mm

Compare the above result with minimum provision given by our code

$$A_{smin} = 0.26 * \frac{2.2 * M_{pa} * 1000 * 229}{400 M_{PA}} = 327.47\text{mm}^2$$

With $d = 200 - 15 - 6 = 179\text{mm}$, for $\phi 12$ main rebar

$$A_{smin} = 327.47\text{mm}^2 < A_{smin} = 557.593\text{mm}^2 \quad \text{ok}$$

$$S_{max} \leq \begin{cases} 2D \\ 400 \\ S_{cal} \end{cases} = 2*200, 400\text{mm}, 200\text{mm}$$

$$S_{max} = 200\text{mm}$$

Provide $\phi 12$ c/c200mm

4.7. Transverse reinforcement design

According to Euro code 2 Part 1,1 - prEN 1992-1-1-2002 Section 9.3.1.1 the ratio of secondary to the main reinforcement shall be at least equal to 20% of the main reinforcement. $A_{s,ansv} \geq 20\% A_{s,cal}$

$$A_s \geq 0.2 * 557.593 \text{mm}^2 = 111.506 \text{mm}^2 < A_{smin} = 327.47 \text{mm}^2$$

Therefore, take the minimum reinforcement

$$d = 330 - 15 - 12 - 4 = 299 \text{mm, for } \varnothing 8 \text{ main transverse rebar}$$

$$A_{smin} = 0.26 * \frac{2.2 * M_{pa} * 1000 * 299}{400 M_{pA}} = 427.57 \text{mm}^2$$

$$S = 1000 * 50.26 / 427.57 = 117.55$$

Provide $\varnothing 8 \text{c}/\text{c}100 \text{mm}$

4.8. Check for shear

Shear Reinforcement

The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [C_{Rd,c} k (100 \rho_1 f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d \leq (v_{min} + k_1 \sigma_{cp}) b_w d$$

Where: f_{ck} is in MPa

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0, \text{ with } d \text{ in mm}$$

$$= 1 + \sqrt{\frac{200}{128}} = 2.25 > 2.0$$

$$\text{Take, } k=2 \quad \rho_1 = \frac{A_{s1}}{b_w * d} \leq 0.02 = \frac{557.593}{1000 * 128} = 0.00436 < 0.02$$

A_{s1} - is the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the section considered

b_w - is the smallest width of the cross-section in the tensile area (mm)

$$\sigma_{cp} = N_{Ed} / A_c < 0.2 f_{cd} \text{ [MPa]}, 0 / A_c = 0 < 0.2 * 11.33 = 2.27 \dots \text{ok}$$

N_{ED} -is the axial force in the cross-section due to loading or prestressing [in N] ($N_{ED} > 0$ for compression). The influence of imposed deformations on N_{ED} may be ignored

A_c -is the area of concrete cross section [mm²]

$V_{Rd,c}$ - is [N]

Note: For the use of $C_{RD,C}$, V_{min} and k_1 refer the National Annex. The recommended value for $C_{RD,C}$ is $\frac{0.18}{\gamma_c}$, that for v_{min} is given by Expression (6.3N) and that for k_1 is 0.15

$$V_{min} = 0.035 * k_1^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}}$$

. $f_{ck}=20\text{MPa}$

$$C_{RD,C} = \frac{0.18}{\gamma_c}, = 0.18/1.5=0.1$$

$$V_{min}=0.035*(2)^{\frac{3}{2}}*(20)^{\frac{1}{2}}=0.443$$

$$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd} \text{ [MPa]}, 0/A_c=0 < 0.2*11.33=2.27 \dots \text{ok}$$

$$V_{Rd,c} = [0.12*2*(100*0.00436*20)^{\frac{1}{3}}+0.15*0]1000*128 \leq (0.443+0.15*0)*1000*128$$

$$=72.3\text{KN/m} > 56.7\text{KN/m} \dots \text{NOT OK}$$

So, $V_{Rd,c}=56.7\text{KN/m} > V_{ED}=42.29\text{kN/m} \dots \text{OK}$

5. Lateral load analysis

In every building, the earth quake and wind load effect should be considered as a part of load in order to build a structure that can safely transfer the loads to the foundation and finally to the ground and absorb some of the energy present rather than suffering damage.

The objective of seismic design in accordance with ES EN 1998-1:2015 is explicitly stated. Its purpose is to ensure that in the event of earthquakes. For Human lives are protected, Damage is limited and Structures important for civil protection remain operational. Since our structure is “regular structure” means, we assume that our building has uniform distribution of mass and stiffness; therefore, we select static analysis (coefficient method)

Design Basis:

Ground condition: (Assume sub soil class D)

Seismic zone: (Wolkite, Zone II)

Wind load Analysis

The wind load analysis is done by Etabs software. The input data from (ES EN 1991-1-1, 2015)is as follows

From the three different procedures of determining design wind actions we select Static procedures of analysis because it is Suitable for the design of low-and mid-rise buildings

- Wind velocity=22m/s (Wind Actions,ES EN 1991:2015, 2018)
- Terrain category = 3, by assuming the building is in area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest (*ES EN 1991-1-4*, 2015).
- Orthography factor = 1(*ES EN 1991-1-4*, 2015) for $\phi < 0.05$, by assuming $Lu > 0.72m$, $\phi = H/Lu$

A. C_s & C_d (*ES EN 1991-1-4*, 2015)

- $C_s = C_d = 0.85$
- Turbulence factor = 1 the recommended value from (*ES EN 1991-1-4*, 2015) (section 4.4)
- Air density = 1.25 kg/m^3 (*ES EN 1991-1-4*, 2015) (section 4.5)

Auto Wind Load Calculation for load pattern wind Y

This calculation presents the automatically generated lateral wind loads for load pattern wind Y according to EUROCODE1 2005, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0 degrees

Basic Wind Velocity, V_b [EC 4.2(2)] $V_b = 22 \frac{\text{meter}}{\text{sec}}$

Windward Coefficient, $C_{p,\text{wind}}$ $C_{p,\text{wind}} = 0.8$

Leeward Coefficient, $C_{p,\text{lee}}$ $C_{p,\text{lee}} = 0.5$

Air Density, ρ $\rho = 1.25$

Top Story = ROOF

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, $c_s c_d$ [EC 6.2(1)] $c_s c_d = 0.85$

Elevation, z_0 $z_0 = 0.3$

Minimum Elevation, z_{min} $z_{min} = 5$

Maximum Elevation, z_{max} $z_{max} = 200$

Turbulence Factor, k_1 [EC 4.4(1)] $k_1 = 1$

Orography Factor, c_o [EC 4.3.3] $c_o = 1$

Turbulence Intensity, I_v [EC 4.4(1)] $I_v = \frac{k_1}{c_o(z) \ln(\frac{z}{z_0})}$ for $z_{min} \leq z \leq z_{max}$
 $= I_v(z_{min})$ for $z < z_{min}$

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5] $k_r = 0.19 \left(\frac{z_0}{0.05}\right)^{0.5}$ $k_r = 0.215389$

Roughness Factor, $c_r(z)$ [EC 4.3.2(1) Eq. 4.4] $c_r(z) = k_r \ln(\frac{z}{z_0})$ for $z_{min} \leq z \leq z_{max}$
 $= c_r(z_{min})$ for $z < z_{min}$

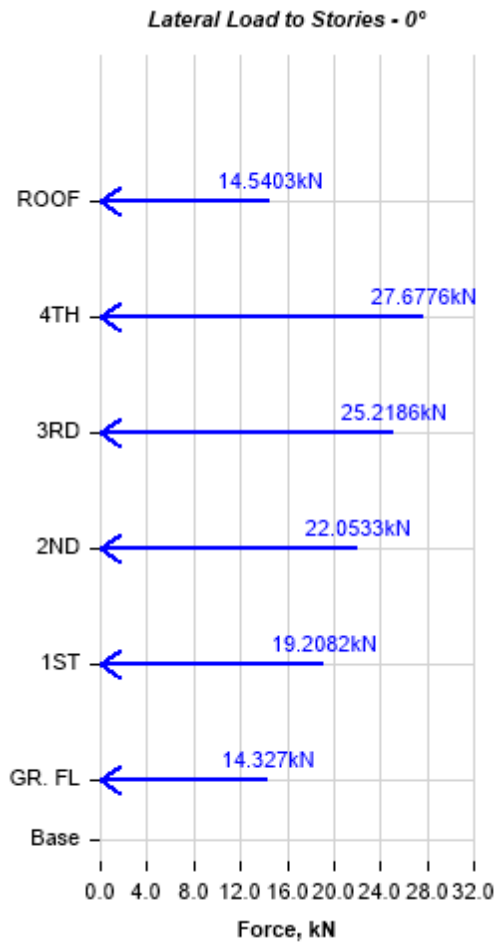
Lateral Loading

Peak Velocity Pressure, $\langle q_p \rangle(z)$ [EC 4.5(1) Eq. 4.8]

$$q_p(z) = [1 + 7I_v(z)] \frac{1}{2} \rho [c_r(z) c_o(z) v_b]^2$$

Wind Pressure, w [EC 5.2(1) Eq. 5.1]

$$w = q_p(z) c_s c_d (c_{p,wind} + c_{p,lee})$$



Story	Elevation m	X-Dir kN	Y-Dir kN
ROOF	14.4	14.5403	0
4TH	11.52	27.6776	0
3RD	8.64	25.2186	0
2ND	5.76	22.0533	0
1ST	2.88	19.2082	0
GR. FL	0	14.327	0
Base	-1.5	0	0

Auto Wind Load Calculation or load pattern wind Y

This calculation presents the automatically generated lateral wind loads for load pattern wind X according to EUROCODE1 2005, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0;90 degrees

Basic Wind Velocity, V_b [EC 4.2(2)] $V_b = 22 \frac{\text{meter}}{\text{sec}}$

Windward Coefficient, $C_{p,\text{wind}}$ $C_{p,\text{wind}} = 0.8$

Leeward Coefficient, $C_{p,\text{lee}}$ $C_{p,\text{lee}} = 0.5$

Air Density, ρ $\rho = 1.25$

Top Story = ROOF

Bottom Story = Base

Include Parapet = No

Factors and Coefficients

Structural Factor, $c_s c_d$ [EC 6.2(1)] $c_s c_d = 0.85$

Elevation, z_0 $z_0 = 0.3$

Minimum Elevation, z_{min} $z_{\text{min}} = 5$

Maximum Elevation, z_{max} $z_{\text{max}} = 200$

Turbulence Factor, k_1 [EC 4.4(1)] $k_1 = 1$

Orography Factor, c_o [EC 4.3.3] $c_o = 1$

Turbulence Intensity, I_v [EC 4.4(1)] $I_v = \frac{k_1}{c_o(z) \ln\left(\frac{z}{z_0}\right)}$ for $z_{\min} \leq z \leq z_{\max}$
 $= I_v(z_{\min})$ for $z < z_{\min}$

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5] $k_r = 0.19 \left(\frac{z_0}{0.05}\right)^{0.05}$ $k_r = 0.215389$

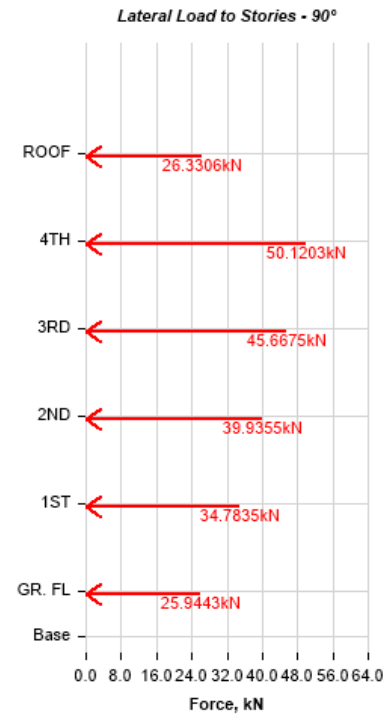
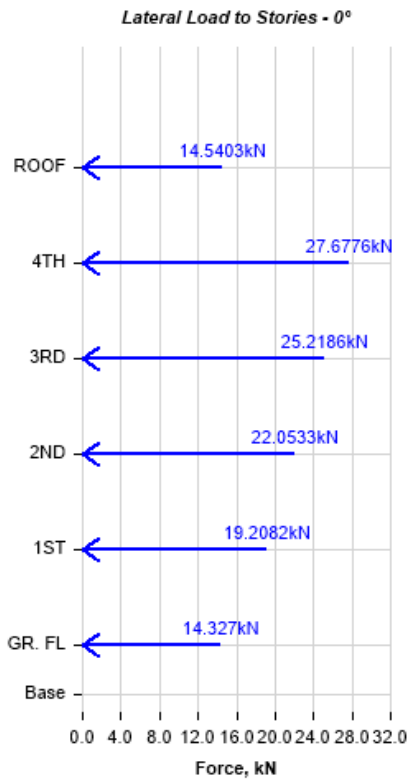
Roughness Factor, $\langle c_r \rangle(z)$ [EC 4.3.2(1) Eq. 4.4] $c_r(z) = k_r \ln\left(\frac{z}{z_0}\right)$ for $z_{\min} \leq z \leq z_{\max}$
 $= c_r(z_{\min})$ for $z < z_{\min}$

Lateral Loading

Peak Velocity Pressure, $\langle q_p \rangle(z)$ [EC 4.5(1) Eq. 4.8] $q_p(z) = [1 + 7I_v(z)] \frac{1}{2} \rho [c_r(z) c_o(z) v_b]^2$

Wind Pressure, w [EC 5.2(1) Eq. 5.1] $w = q_p(z) c_s c_d (c_{p,wind} + c_{p,lee})$

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	14.5403	0
4TH	11.52	27.6776	0
3RD	8.64	25.2186	0
2ND	5.76	22.0533	0
1ST	2.88	19.2082	0
GR. FL	0	14.327	0
Base	-1.5	0	0

Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	0	26.3306
4TH	11.52	0	50.1203
3RD	8.64	0	45.6675
2ND	5.76	0	39.9355
1ST	2.88	0	34.7835
GR. FL	0	0	25.9443
Base	-1.5	0	0

Earthquake load analysis

An earthquake is the vibration of earth produced by the rapid release of accumulated energy in elastically strained rock. The building is to be constructed in Wolkite city, which is located in. it is categorized seismic zone three.

Analysis of earthquake loads on building on ES EN 2015

The objective of seismic design in accordance with ES EN 1998-1:2015 is explicitly stated. Its purpose is to ensure that in the event of earthquakes. For human lives are protected, Damage is limited and Structures important for civil protection remain operational.

Distribution of the base shear over the height of the building

The horizontal story shear force determined above shall be distributed to the lateral load resisting system based on the assumption that the slab floor at every level are rigid enough. The storey shear force will be distributed to each frame system according to their stiffness,

Seismic load analysis

Seismic load analysis (earthquake load analysis) also analyzed by Etabs software, the required parameters derived from ES EN 1998-1:2015, Design of Structures for Earthquake Resistance - Part 1: General rules - seismic actions and rules for buildings

- A. Ground type = D, by assuming stratigraphic profile of deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil. (Appendix D)
- B. Spectrum type = 2, . If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s , not greater than 5.5, it is recommended that the Type 2 spectrum is adopted.(ES EN 1998-1, 2015)
- C. Seismic zone = 2 (ES EN 1998-1, 2015) (Annex D)
- D. Ground acceleration = 0.07 ,for zone 2 (Appendix D)
 - Soil factor = 1.35 , for spectrum type 2 & ground type D (ES EN 1998-1, 2015)
 - Spectrum period, for spectrum type 2 & ground type D (Appendix D)
 - $T_b = 0.2$

- $T_c = 0.8$
- $T_d = 2$
- Behavioral factor = 4.9, for frame system structures $3.0\alpha u/\alpha_1$, $\alpha u/\alpha_1 = 1.3$ for multistory building. (ES EN 1998-1, 2015) (Appendix D)
- Correction factor = 1, damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping (ES EN 1998-1, 2015)
- Lower bound factor = 0.2, recommended value from (ES EN 1998-1, 2015)

5.3.1 Base shear force

(1) The seismic base shear force F_b , for each horizontal direction in which the building is analysed, shall be determined using the following expression:

$$F_b = S_d(T_1) \cdot m \quad (4.5)$$

where

$S_d(T_1)$ is the ordinate of the design spectrum (see 3.2.2.5) at period T_1 ;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the total mass of the building, above the foundation or above the top of a rigid basement, computed in accordance with 3.2.4(2);

λ is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 \leq 2 T_c$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise.

NOTE The factor λ accounts for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller, on average by 15%, than the total building mass.

(2) For the determination of the fundamental period of vibration T_1 of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used.

(3) For buildings with heights of up to 40 m the value of T_1 (in s) may be approximated by the following expression:

$$T_1 = C_t \cdot H^{(3/4)}$$

where

C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

For concrete moment resisting frame $C_t=0.075$, $H=34.94\text{m}$

Ground type	S	$T_B(\text{s})$	$T_C(\text{s})$	$T_D(\text{s})$
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

$$T_1 = 0.075 \cdot 34.94^{0.75} = 1.078 \text{sec}$$

$$T_1 \leq \min(4T_C(\text{s}) = 4 \cdot 0.8 = 3.2 \text{sec}, 2 \text{sec})$$

$$T_1 = 3.2 \text{sec} < 1 \text{sec} \dots \text{not ok}$$

Take $T_1 = 1 \text{sec}$

Table 5-1 Importance class of buildings

Importance class	Buildings

I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EBCS EN 1990:2013, Annex B.

Distribution of the horizontal seismic forces

(1) The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

(2)P The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys.

$$F_i = \frac{(F_b - F_t)W_i h_i}{\sum_{j=1}^n W_j h_j}$$

where

F_t the concentrated force at the top which is in addition to F_n shall be determined by the equation

$$F_t = 0.007T_l F_b$$

F_i is the horizontal force acting on storey i ;

F_b is the seismic base shear in accordance with expression

$$F_b = F_t + \sum_{j=1}^n F_j$$

s_i, s_j are the displacements of masses m_i, m_j in the fundamental mode shape;

m_i, m_j are the storey masses computed in accordance with **3.2.4(2)**.

(3) When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as being given by:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \quad (4.11)$$

where

z_i, z_j are the heights of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement).

P The horizontal forces F_i determined in accordance with this clause shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane.

For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \quad (3.13)$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \eta \cdot \frac{2.5}{q} \quad (3.14)$$

$$T_B \leq T \leq T_C : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.15)$$

$$T_D \leq T : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.16)$$

where

a_g, S, T_B, T_C and T_D are as defined in **3.2.2.2**;

$S_d(T)$ is the design spectrum;

q is the behaviour factor;

β is the lower bound factor for the horizontal design spectrum.

NOTE The value to be ascribed to β for use is found in the National Annex. The recommended value for β is 0,2. (ES EN 1998-1, 2015)

$$S_d(T) = \text{Max} \{ \mathbf{0.2638}, \mathbf{0.1962} \}$$

$$S_d(T) = 0.2638$$

Story	Dia phr ag m	Mass X kg	Mass Y kg	XCM m	YCM m	Cumulati ve X kg	Cumulati ve Y kg	XCCM m	YCC M m
ROOF	D1	47749.7	47749.7	14.087	6.225	47749.73	47749.73	14.087	6.225
		3	3	2	6			2	6
4TH	D1	65746.8	65746.8	13.354	6.275	113496.5	113496.5	13.662	6.254
		7	7	1	8	9	9	5	7
3RD	D1	66950.3	66950.3	13.183	6.262	180446.9	180446.9	13.484	6.257
		5	5		6	4	4	6	6
2ND	D1	66950.3	66950.3	13.183	6.262	247397.2	247397.2	13.403	6.259
		5	5		6	9	9		
1ST	D1	66950.3	66950.3	13.183	6.262	314347.6	314347.6	13.356	6.259
		5	5		6	4	4	2	7
GR. FL	D1	59569.3	59569.3	13.275	6.261	373916.9	373916.9	13.343	6.26
		3	3	9	4	7	7	4	

$$F_b = S_d(T_1) \cdot m = 0.2638 \cdot 373916.97 = 98639.3 \text{ KN}$$

$$F_t = 0.07 T_1 F_b = 0.07 \cdot 1 \cdot 98639.3 = 6904.75 \text{ KN}$$

Table base shear

level	remark	wi	hi	wihi	Fb-ft	Fi
Roof	w1	47749.7	16	763996	91734.6	26067.1
4th	w2	65746.9	12.8	841560	91734.6	28713.5
3rd	w3	66950.4	9.6	642723	91734.6	21929.3
2nd	w4	66950.4	6.4	428482	91734.6	14619.6
1st	w5	66950.4	3.2	214241	91734.6	7309.78
GR.FL	w6	59569.3	0	0	91734.6	0
			\sum mihi	2891002		

Auto Seismic Load Calculation for load pattern EQXL

This calculation presents the automatically generated lateral seismic loads for load pattern EQXL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = X - Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075m$
 Structure Height Above Base, H $H = 15.9 \text{ m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g $a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3] $S = 1.35$
 Constant Acceleration Period Limit, T_B [EC Table 3.3] $T_B = 0.2 \text{ sec}$
 Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8\text{sec}$
 Constant Displacement Period Limit, T_D [EC Table 3.3] $T_D = 2 \text{ sec}$
 Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$
 Behavior Factor, q [EC 3.2.2.5(3)] $q = 4.9$

Seismic Response

Spectral Response Acceleration, $S_d(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \text{ for } T \leq T_B$$

$$= a_g S \frac{2.5}{q} \text{ for } T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C}{T} \right] \geq \beta a_g \text{ for } T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C T_D}{T^2} \right] \geq \beta a_g \text{ for } T_D \leq T$$

Equivalent Lateral Forces

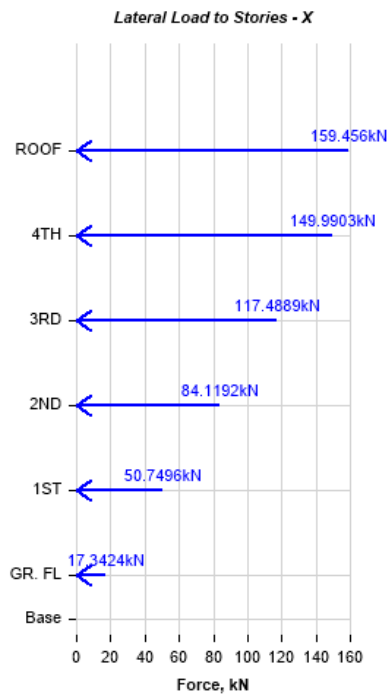
Seismic Base Shear Coefficient

$$V_{\text{coeff}} = S_d(T_1)\lambda$$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
X - Ecc. Y	0.395	12758.304	615.1325

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	159.456	0
4TH	11.52	149.9903	0
3RD	8.64	117.4889	0
2ND	5.76	84.1192	0
1ST	2.88	50.7496	0
GR. FL	0	17.3424	0
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQXR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = X + Eccentricity Y

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075m$

Structure Height Above Base, H $H = 15.9 \text{ m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g $a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3] $S = 1.35$

Constant Acceleration Period Limit, T_B [EC Table 3.3] $T_B = 0.2 \text{ sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8 \text{ sec}$

Constant Displacement Period Limit, T_D [EC Table 3.3] $T_D = 2 \text{ sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 4.9$

Seismic Response

Spectral Response Acceleration,
 $S_d(T_1)$ [EC 3.2.2.5(4) Eq. 3.13]

$$S_d(T_1) = a_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right] \text{ for } T \leq T_B$$
$$= a_g S \frac{2.5}{q} \text{ for } T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C}{T} \right] \geq \beta a_g \text{ for } T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C T_D}{T^2} \right] \geq \beta a_g \text{ for } T_D \leq T$$

Equivalent Lateral Forces

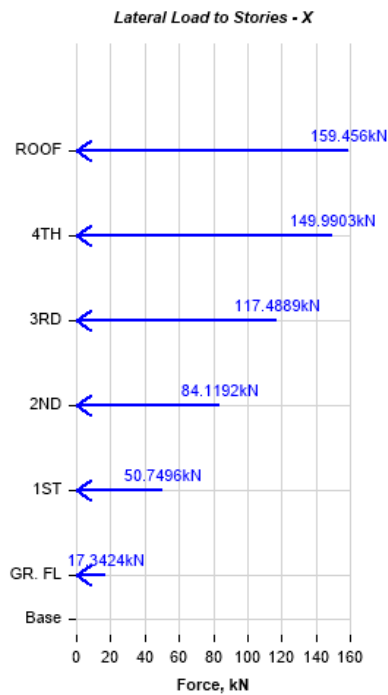
Seismic Base Shear Coefficient

$$V_{\text{coeff}} = S_d(T_1)\lambda$$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
X + Ecc. Y	0.369	12011.9257	579.1464

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	159.456	0
4TH	11.52	149.9903	0
3RD	8.64	117.4889	0
2ND	5.76	84.1192	0
1ST	2.88	50.7496	0
GR. FL	0	17.3424	0
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQYL according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Y - Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075m$

Structure Height Above Base, H $H = 15.9 \text{ m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g $a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3] $S = 1.35$

Constant Acceleration Period Limit, T_B [EC Table 3.3] $T_B = 0.2 \text{ sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8 \text{ sec}$

Constant Displacement Period Limit, T_D [EC Table 3.3] $T_D = 2 \text{ sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 4.9$

Seismic Response

Spectral Response Acceleration, $S_d(T_1)$ [EC 3.2.2.5(4) Eq. 3.13] $S_d(T_1) = a_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$ for $T \leq T_B$

$$= a_g S \frac{2.5}{q} \text{ for } T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C}{T} \right] \geq \beta a_g \text{ for } T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C T_D}{T^2} \right] \geq \beta a_g \text{ for } T_D \leq T$$

Equivalent Lateral Forces

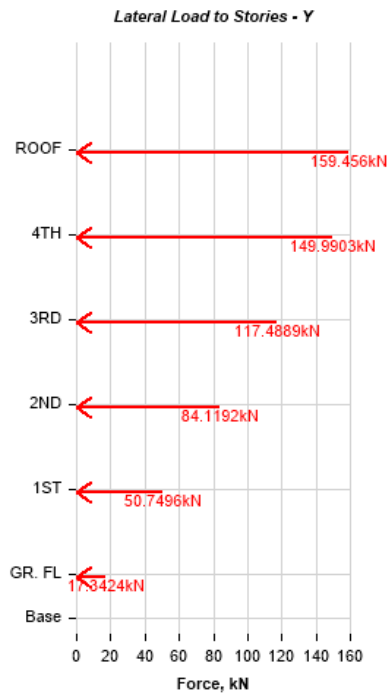
Seismic Base Shear Coefficient

$$V_{\text{coeff}} = S_d(T_1) \lambda$$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
Y - Ecc. X	0.658	12011.9257	579.1464

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	0	159.456
4TH	11.52	0	149.9903
3RD	8.64	0	117.4889
2ND	5.76	0	84.1192
1ST	2.88	0	50.7496
GR. FL	0	0	17.3424
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQYR according to EUROCODE8 2004, as calculated by ETABS.

Direction and Eccentricity

Direction = Y + Eccentricity X

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, C_t [EC 4.3.3.2.2] $C_t = 0.075m$

Structure Height Above Base, H $H = 15.9 \text{ m}$

Factors and Coefficients

Country =

Design Ground Acceleration, a_g $a_g = 0.07g$

Ground Type [EC Table 3.1] = D

Soil Factor, S [EC Table 3.3] $S = 1.35$

Constant Acceleration Period Limit, T_B [EC Table 3.3] $T_B = 0.2 \text{ sec}$

Constant Acceleration Period Limit, T_C [EC Table 3.3] $T_C = 0.8 \text{ sec}$

Constant Displacement Period Limit, T_D [EC Table 3.3] $T_D = 2 \text{ sec}$

Lower Bound Factor, β [EC 3.2.2.5(4)] $\beta_0 = 0.2$

Behavior Factor, q [EC 3.2.2.5(3)] $q = 4.9$

Seismic Response

Spectral Response Acceleration, $S_d(T_1)$ [EC 3.2.2.5(4) Eq. 3.13] $S_d(T_1) = a_g S \left[\frac{2}{3} + \frac{T}{T_B} \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$ for $T \leq T_B$

$$= a_g S \frac{2.5}{q} \text{ for } T_B \leq T \leq T_C$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C}{T} \right] \geq \beta a_g \text{ for } T_C \leq T \leq T_D$$

$$= a_g S \frac{2.5}{q} \left[\frac{T_C T_D}{T^2} \right] \geq \beta a_g \text{ for } T_D \leq T$$

Equivalent Lateral Forces

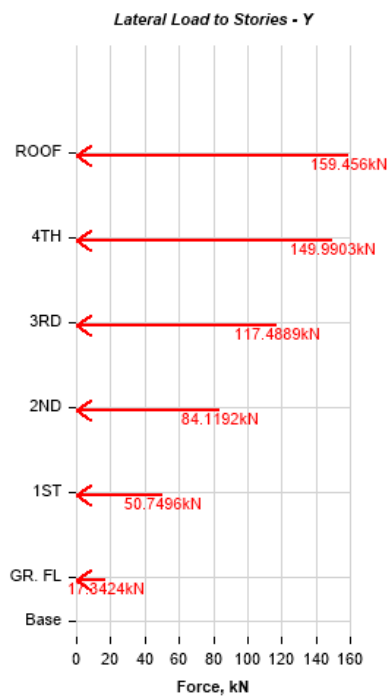
Seismic Base Shear Coefficient

$$V_{\text{coeff}} = S_d(T_1) \lambda$$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
Y + Ecc. X	0.658	12011.9257	579.1464

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
ROOF	14.4	0	159.456
4TH	11.52	0	149.9903
3RD	8.64	0	117.4889
2ND	5.76	0	84.1192
1ST	2.88	0	50.7496
GR. FL	0	0	17.3424
Base	-1.5	0	0

6. Frame analysis

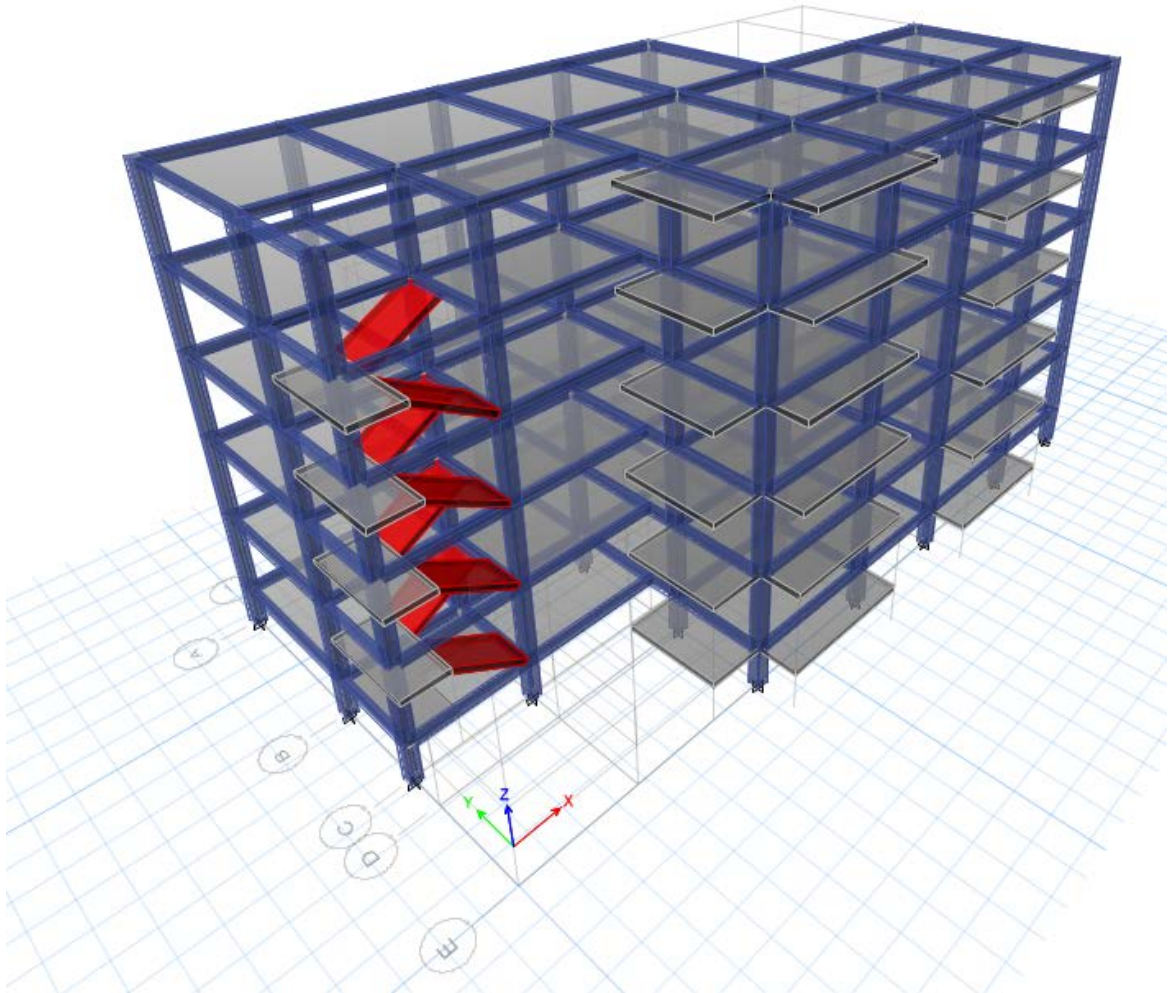


Figure 6-13D model of project

Frame is analysed to get maximum shear and moments in the structure by using ETABS 2016 software.

The steps to analyze the structure described below.

1. Select the Base Units and Design Codes :- we select the base unit to metric SI and design codes of Euro Code
2. Set up Grid Lines:- set up number and spacing of grid lines in both x and y direction by using Arctehtural drawing data.
3. Define Story Levels:- define story height and label them.

4. Define material Properties: - define material properties that we used in over all design process. For initial we define C-25 concrete and S-400 rebar according to material data's on (ES EN 1992-1-1, 2015)

A. Concrete:-

Grade - C25

Modules of elasticity – 31 GPa

Weight per unit volume- 25kg/m³.

Poisson's ratio, ν – is the ratio between lateral strains and longitudinal strains in a material subjected to loading, Poisson's ratio of concrete varies between 0.1 for high strength mix and 0.2 for weak mixes. It is normally taken as 0.15 for strength design and 0.2 for serviceability criteria.

Coefficient of thermal expansion: - when free to deform concrete will expand and contract due to fluctuations in temperature. An average value for coefficient of the coefficient of thermal expansion of concrete is about 10 millionths per degree Celsius ($10 \times 10^{-6} / ^\circ\text{C}$).

The software itself calculates shear modulus, the formula for shear modulus= $E/2(1+\nu)$.

B. Rebar

Grade – S400

Mass per unit volume = 78.6KN/m³

Modulus of elasticity =200GPa=200000MPa

Coefficient of Thermal expansion=0.0000117

5. Define Section properties:- for initial we define column , beam and slab sections.
 - a. Beam:- Cross section 250x400
 - b. Column:- Cross section 250x400
 - c. Slab :- depth of 210mm
6. Draw Structural Objects :- draw the structural objects included in architectural drawing
7. Define Load Patterns :- define the loads on the structure

Name	Type	Self Weight Multiplier	Auto Load
Dead	Dead	1	
Live	Live	0	
Super dead load	Superimposed Dead	0	
Partition load	Superimposed Dead	0	
EQXL	Seismic	0	EUROCODE8 2004
EQXR	Seismic	0	EUROCODE8 2004
EQYL	Seismic	0	EUROCODE8 2004
EQYR	Seismic	0	EUROCODE8 2004
wind Y	Wind	0	EUROCODE1 2005
wind X	Wind	0	EUROCODE1 2005

8. Define load combinations.

Table 6-1 Load combination

comb 1	1.35DL	1.5LL			linear add
comb 2	DL	LL			linear add
comb 3	DL	0.6LL	EQXL	0.3EQYL	linear add
comb 4	DL	0.6LL	EQXR	0.3EQYL	linear add
comb 5	DL	0.6LL	-EQXL	0.3EQYL	linear add
comb 6	DL	0.6LL	-EQXR	0.3EQYL	linear add
comb 7	DL	0.6LL	EQXL	0.3EQYR	linear add
comb 8	DL	0.6LL	EQXR	0.3EQYR	linear add
comb 9	DL	0.6LL	-EQXL	0.3EQYR	linear add
comb 10	DL	0.6LL	-EQXR	0.3EQYR	linear add
comb 11	DL	0.6LL	EQXL	-0.3EQYL	linear add
comb 12	DL	0.6LL	EQXR	-0.3EQYL	linear add
comb 13	DL	0.6LL	-EQXL	-0.3EQYL	linear add
comb 14	DL	0.6LL	-EQXR	-0.3EQYL	linear add
comb 15	DL	0.6LL	EQXL	-0.3EQYR	linear add
comb 16	DL	0.6LL	EQXR	-0.3EQYR	linear add
comb 17	DL	0.6LL	-EQXL	-0.3EQYR	linear add
comb 18	DL	0.6LL	-EQXR	-0.3EQYR	linear add
comb 19	DL	0.6LL	EQYL	0.3EQXL	linear add
comb 20	DL	0.6LL	EQYR	0.3EQXL	linear add
comb 21	DL	0.6LL	-EQYL	0.3EQXL	linear add
comb 22	DL	0.6LL	-EQYR	0.3EQXL	linear add
comb 23	DL	0.6LL	EQYL	0.3EQXR	linear add
comb 24	DL	0.6LL	EQYR	0.3EQXR	linear add
comb 25	DL	0.6LL	-EQYL	0.3EQXR	linear add
comb 26	DL	0.6LL	-EQYR	0.3EQXR	linear add
comb 27	DL	0.6LL	EQYL	-0.3EQXL	linear add
comb 28	DL	0.6LL	EQYR	-0.3EQXL	linear add
comb 29	DL	0.6LL	-EQYL	-0.3EQXL	linear add
comb 30	DL	0.6LL	-EQYR	-0.3EQXL	linear add
comb 31	DL	0.6LL	EQYL	-0.3EQXR	linear add
comb 32	DL	0.6LL	EQYR	-0.3EQXR	linear add
comb 33	DL	0.6LL	-EQYL	-0.3EQXR	linear add
comb 34	DL	0.6LL	-EQYR	-0.3EQXR	linear add
comb 35	1.35DL	1.5LL	0.9WL		linear add
comb 36	1.35DL	1.5LL	-0.9WL		linear add
comb 37	1.35DL	1.5WL			linear add
comb 38	comb 3-18				envelope
comb 39	comb 19-34				envelope
comb 40	comb 1-37				envelope

9. Load the structural model

- calculate the units of load for all defined load patters by hand and find other loading properties from the code like live load

- Example on shell loads(uniform)
 - For super dead load (floor finish and ceiling plaster)

Assume marble floor finish with unit weight of 23KN/m^3 and thickness of 20mm

$$\text{Load of floor finish} = 23 * 0.02 = 0.46 \text{ KN/m}^2$$

Assume ceiling plaster thickness=30mm and unit weight of 23 KN/m^3

$$\text{Load of ceiling plaster} = 23 * 0.03 = 0.69 \text{ KN/m}^2$$

$$\text{Super dead load} = 0.46 + 0.69 = \underline{\underline{0.115 \text{ KN/m}^2}}$$

- Example on frame loads (distributed)
 - For beam partition load
 - Assume partition thickness 15cm, height 3m, made of HCB unit weight 14 KN/m^3

$$\text{Partition load} = 0.15\text{m} * 3\text{m} * 14\text{KN/m}^3 = \underline{\underline{6.3 \text{ KN/m}}}$$

10. Assign Joint restraint :- assign joint on the bottom of the foundation column to fixed support.
11. Assign Joint diaphragm :- assign joint diaphragm for the whole structure to calculate center of mass.
12. Analyze the model:- analyse the model to get moment and shear results.

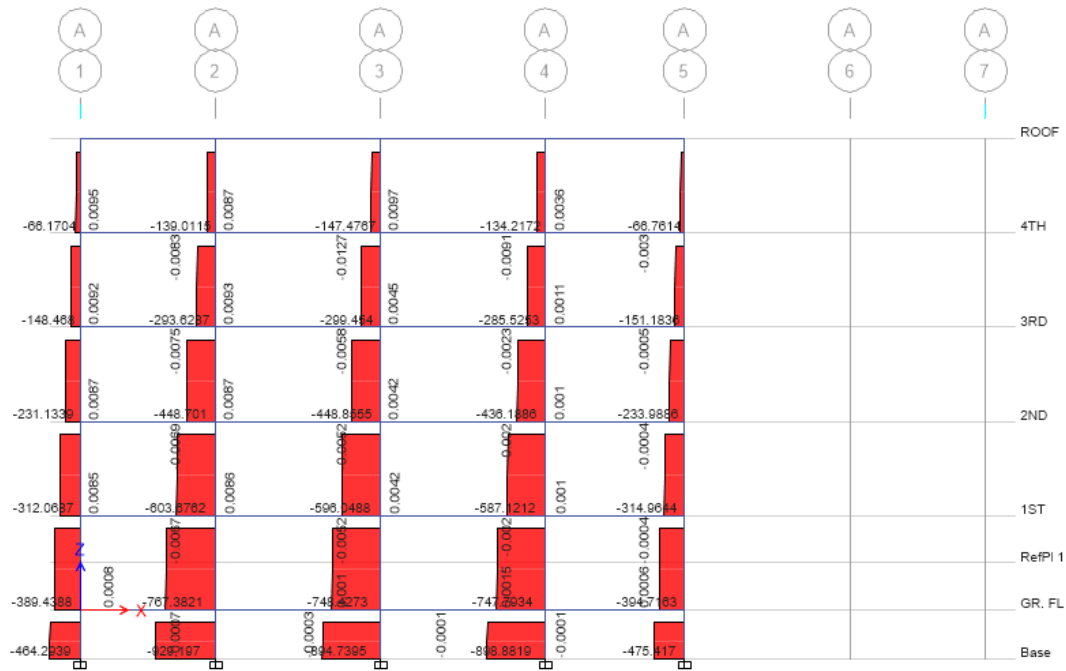


Figure 6-2 Axial load result sample generated from ETABS

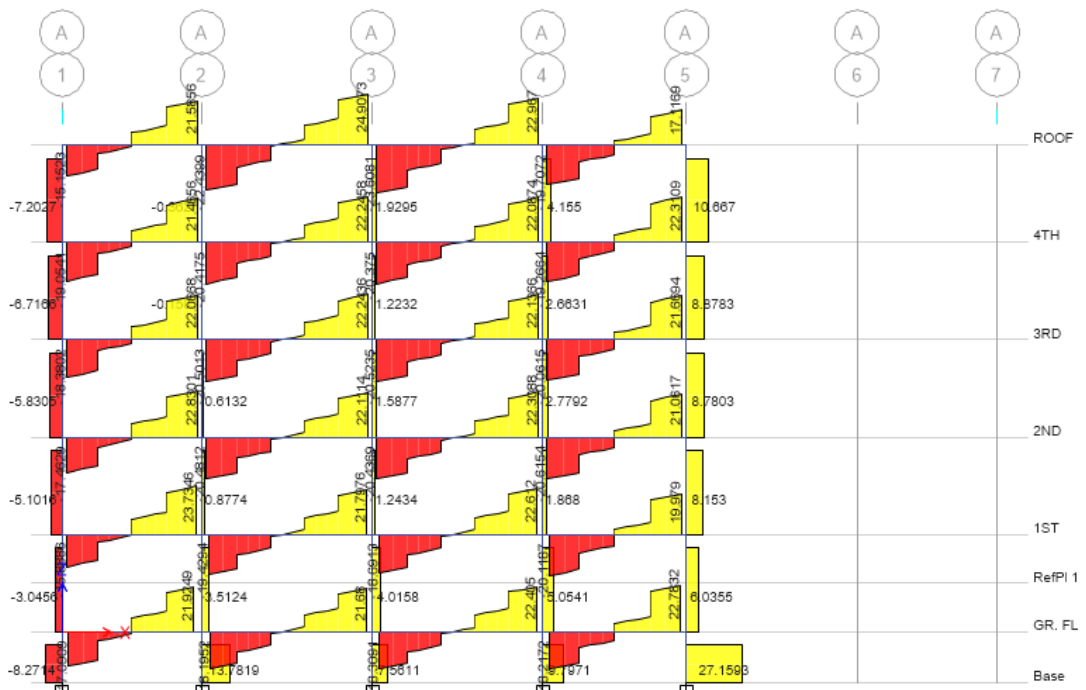


Figure 6-3 Shear force result sample generated from ETABS

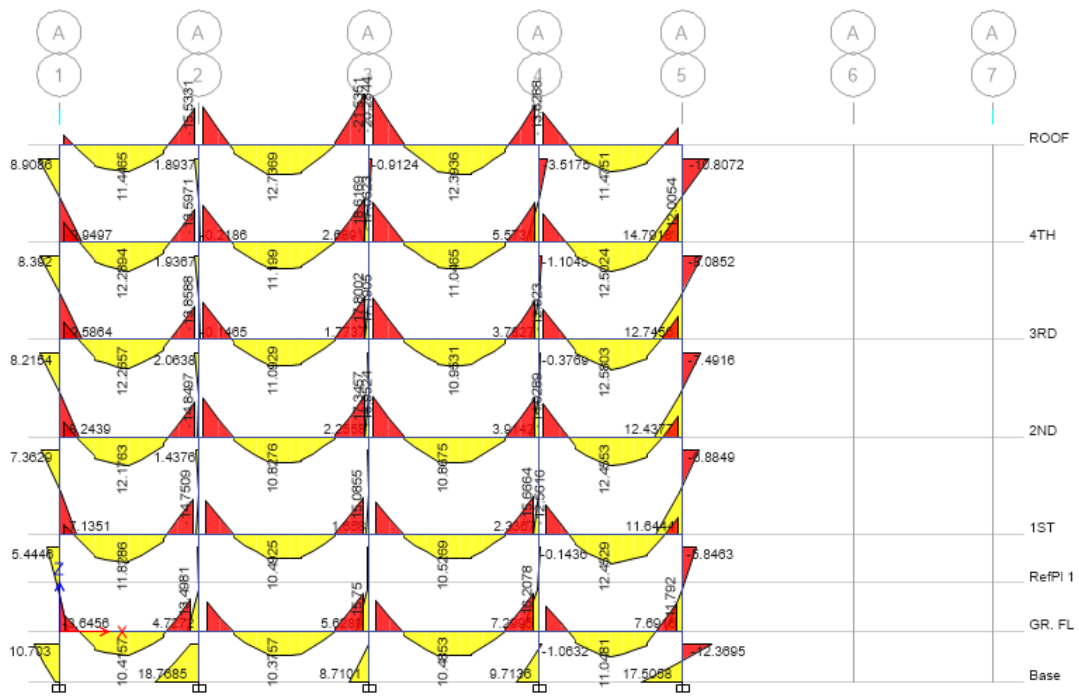


Figure 6-4 Moment result sample generated from ETABS

7. Beam design

Beam is a horizontal structural member which resist bending due to applied loads. As any members, beam is also subjected to different kinds of loadings. Its mode of deflection is primarily by bending. The loads applied to the beam result in reaction forces at the beam's support points. The total effect of all the forces acting on the beam is to produce shear forces and bending moments within the beam, that in turn induce internal stresses, strains and deflections of the beam. Beams are characterized by their manner of support, profile (shape of cross-section), length, and their material.

There are two types of beam sections, singly reinforced and doubly reinforced sections.

- Singly reinforced beam is reinforced with steel bars only at the tension zone.
- Doubly reinforced beam is reinforced in both zones (tension & compression zone).

If a section of a beam is limited in depth, it cannot develop the compressive force required to resist the applied bending moment. If it's a small increase in moment, over reinforced beam section can be used (not recommended in design). In such case, providing a reinforcement in the compression zone assists the concrete in resisting compressive force. This kind of beam section is called doubly reinforced beam. If the required depth is also unacceptable, doubly reinforced beam is provided.

Sample calculation

Design Data

Dimension – 250x400mm

Material used

A -Concrete

Grade of concrete C-25

Concrete cube strength, $f_{cu}=25\text{MPa}$

Concrete characteristic strength= $0.8f_{cu}=0.8*25=20\text{MPa}$

Concrete design strength, $f_{cd} = \alpha_{cc} * \frac{f_{ck}}{\gamma_c} = 0.85 * \frac{25}{1.5} = 11.33\text{MPa}$

B -Steel S-400

Steel tensile strength, 400MPa

Steel tensile design strength, $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = 347.83\text{MPa}$

Elastic modulus of steel, $E_s=200\text{Gpa}$

7.1. Concrete Cover design

The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

$C_{nom} = C_{min} + \Delta C_{dev}$

Where, C_{min} , Minimum concrete cover

ΔC_{dev} , is allowance in design for deviation.

Minimum concrete cover

$$C_{min} = \max \left\{ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur,add} \right. \\ \left. \begin{matrix} C_{min,b} \\ 10\text{mm} \end{matrix} \right\}$$

Where;

$C_{min,b}$ -minimum cover due to bond requirement, see (ES EN 1992-1-1, 2015)Art. 4.4.1.2 (3).

$C_{min,dur}$ -minimum cover due to environmental conditions, see (ES EN 1992-1-1, 2015)Art 4.4.1.2(5)

$\Delta C_{dur,\gamma}$ - additive safety element, see (ES EN 1992-1-1, 2015)Art 4.4.1.2 (6)

$\Delta C_{dur,st}$ -reduction of minimum cover for use of stainless steel, see (ES EN 1992-1-1, 2015) Art 4.4.1.2 (7)

$\Delta C_{dur,add}$ -reduction of minimum cover for use of additional protection, see (ES EN 1992-1-1, 2015) Art 4.4.1.2 (8)

But; the recommended value of $\Delta C_{dur,Y^-}$, $\Delta C_{dur,st}$, and $\Delta C_{dur,add}$ -is zero see (ES EN 1992-1-1, 2015) Art. 4.4.1.2 (6, 7, and 8).

➤ $C_{min,b}$ minimum cover due to bond requirement [[Appendix B](#)]

Therefore, As the bars are arranged separately, the minimum cover due to bond requirement, and let the longitudinal bar diameter be 20mm

$C_{min} = 20\text{mm}$

➤ $C_{min,dur}$ minimum cover due to environmental conditions [[Appendix B](#)]

Therefore, As the structural class is S1 & Exposure class is XC1, the minimum cover due to

environmental requirement, $C_{min, dur}$ is 10mm.

Finally, $C_{min} = \max \left\{ \begin{array}{l} C_{min, b} \\ C_{min, b, dur} + \Delta C_{dur, Y^-} - \Delta C_{dur, st} - \Delta C_{dur, add} \\ 100 \end{array} \right.$

$$C_{min} = \max \left\{ \begin{array}{l} 20\text{mm} \\ 10 + 0 + 0 = 10\text{mm} \\ 10\text{mm} \end{array} \right.$$

$C_{min} = 20\text{mm}$

➤ Allowance in design for deviation

To calculate the nominal cover, C_{nom} an addition to the minimum cover shall be made in design to allow for the deviation (ΔC_{dev}). The required minimum cover shall be increased by the absolute value of the accepted negative deviation.

Note: The value of ΔC_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

In certain situations, the accepted deviation and hence allowance, (ΔC_{dev}) may be reduced.

Note: The reduction in ΔC_{dev} in such circumstances for use in a Country may be found in its National Annex. The recommended values are:

Where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation ΔC_{dev} may be reduced: $10 \text{ mm} \geq \Delta c_{dev} \geq 5 \text{ mm}$

Where it can be assured that a very accurate measurement device is used for monitoring and non-conforming members are rejected (e.g. precast elements), the allowance in design for deviation ΔC_{dev} may be reduced: $10 \text{ mm} \geq \Delta c_{dev} \geq 0 \text{ mm}$ (ES EN 1992-1-1, 2015)

Therefore, we take $\Delta C_{dev} = 10 \text{ mm}$

$$C_{nom} = C_{min} + \Delta C_{dev} = 20 \text{ mm} + 10 \text{ mm} = 30 \text{ mm}$$

7.2. Check depth for deflection

$$\frac{l}{d} = k \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_o}{\rho} + 3.2 \sqrt{f_{ck}} \left(\frac{\rho_o}{\rho} - \sqrt{\frac{\rho_o}{\rho}} \right) \right]$$
$$\frac{l}{d} = k \left[11 + 1.5 \sqrt{f_{ck}} \frac{\rho_o}{\rho} + 3.2 \sqrt{f_{ck}} \sqrt{\frac{\rho_o}{\rho}} \right] \text{ if } \rho > \rho_o$$

(ES EN 1992-1-1, 2015) Art 7.4.2

Taking $L/d=26$ for end span $L/d = 30$ for interior span from [Appendix B]

Where L = effective length of the beam

d = effective depth but those value is for steel grade 500,

We must have to modify it. In our case, Modification factor = $500/400=1.25$

End span, $26 * 1.25 = 32.5$

Interior span, $30 * 1.25 = 37.5$

Cantilever, $8 * 1.25 = 10$

7.3. check the whether the beam single or double reinforced

A beam should be treated as singly reinforced if $K < 0.167$

A beam should be treated as doubly reinforced if $K > 0.167$

$$\text{Where } K = \frac{M_{sd}}{bd^2f_{ck}} = \frac{21.53}{250 * 350 * 350 * 20} = 0.04$$

7.4. Calculate reinforcement

Let calculate for axis A roof slab reinforcement on support location axis c

$M_{sd} = 21.53$

check the minimum and maximum reinforcement

$$A_{st, \min} = \max \begin{cases} 0.26 \frac{f_{ctm}}{f_{yk}} bd = 0.26 \frac{2.2}{400} 250 * 350 = 125.125 \text{ mm}^2 \\ 0.0013bd = 0.0013 * 250 * 350 = 113.75 \text{ mm}^2 \end{cases}$$

$$A_{st, \max} = 0.04bd = 0.04 * 250 * 350 = 3500$$

$$A_{st, \text{cal}} = \frac{M_{sd}}{Zf_{yd}}$$

$$Z = \frac{d}{2} (1 + \sqrt{1 - 3.53K})$$

$$Z = \frac{350}{2} (1 + \sqrt{1 - 3.53 * 0.04}) = 338.78$$

$$A_{st, \text{cal}} = \frac{M_{sd}}{Zf_{yd}} = \frac{21.53}{338.78 * 347.82} = 182.71$$

Therefore $A_{st, \min} \leq A_{st, \text{cal}} \leq A_{st, \max}$, $A_{st} = 182.71 \text{ mm}^2$

If we use $\phi 14$

$$a_{st} = 3.14 * 14^2 / 4 = 153.56$$

$$\text{no of bars} = A_{st}/a_{st} = 182.71 \text{mm}^2 / 153.56 = 1.2$$

so we can use 2 $\phi 14$ bars

reinforcement calculation for other beams summarized in table on [Appendix D-]

Lap length

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,\min} \text{ (ES EN 1992-1-1, 2015)}$$

$$l_{0,\min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\phi \\ 200\text{mm} \end{cases}$$

$$\text{Where: } l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}}$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s}$$

$$f_{bd} = 2.25\eta_1\eta_2 f_{ctd}$$

$$\eta_1 = 1 \text{ for good quality of bond}$$

$$\eta_1 = 0.7 \text{ for other cases (ES EN 1992-1-1, 2015) Art, 8.4.2}$$

$$\eta_2 = 1 \text{ for } \phi \leq 32\text{mm}$$

$$\eta_2 = \frac{132-\phi}{100} \text{ for } \phi > 32\text{mm}$$

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

Sample calculation for $\phi 14$ reinforcement bar

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

$$f_{ctk} = 1. \text{ mpa, from ESEN 1992 Table 3.1}$$

$$f_{ctd} = \frac{0.85 \cdot 1.5}{1.5} = \mathbf{0.85}$$

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd}$$

$\eta_1 = 1$, our slab D is < 250mm, good condition of bond

$\eta_2 = 1$, for $\varnothing \leq 32\text{mm}$

$$f_{ctd} = 0.85$$

$$f_{bd} = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$$

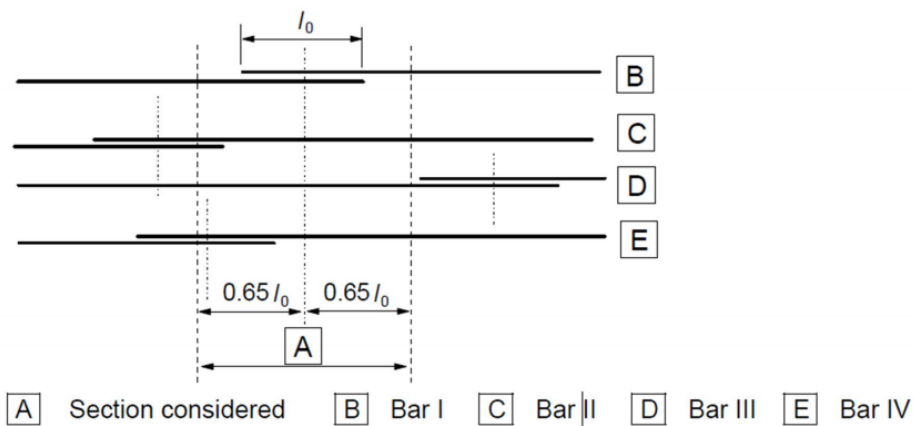
$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = \mathbf{347.83}$$

$$l_{b,rqd} = \frac{\varnothing}{4} * \frac{\sigma_{sd}}{f_{bd}} = \frac{14}{4} * \frac{347.83}{1.91} = \mathbf{637.38\text{mm}}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

α_1, α_2 & $\alpha_3 = 1$, $\alpha_4 = 0.7$, for reinforcement bar in compression

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within $0.65 l_0$ from the center of the lap length considered



Example: Bars II and III are outside the section being considered: $\rho = 50$ and $\alpha_6 = 1.4$

Figure 8.8: Percentage of lapped bars in one lapped section

Let all reinforcement in slab lapped on the same center ,therefore $\alpha_6 = 1.5$ from the table value for $\rho_1 > 50 \%$,

$$l_{o,\min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.5 * 637.38 = 286.82\text{mm} \\ 15\phi = 15 * 14 = 210\text{mm} \\ 200\text{mm} \end{cases}$$

$$l_{o,\min} = 286.82\text{mm}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,\min}$$

$$l_0 = 1 * 1 * 1 * 1.5 * 637.38\text{mm} = 956.07 \geq 286.82 \text{ mm}$$

lap length, $l_0 = \mathbf{1000 \text{ mm}}$

Anchorage length

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,\min}$$

for anchorages in tension

$$l_{b,\min} \geq \max \begin{cases} 0.3l_{b,rqd} \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,\min} \geq \max \begin{cases} 0.3 * 637.38 = 191.24 \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,\min} = \mathbf{191.24\text{mm}}$$

$\alpha_1 \alpha_2 \alpha_3 \alpha_5$ coefficients [appendix B]

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,\min}$$

$$l_{bd} = 1 * 1 * 1 * \mathbf{637.38\text{mm}} = \mathbf{637.38\text{mm}} \geq 191.24 \text{ mm}$$

for anchorages in compression

$$l_{b,\min} \geq \max \begin{cases} 0.6l_{b,rqd} \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} \geq \max \begin{cases} 0.6 * 637.38 = 382.43 \\ 10\phi \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = \mathbf{327.8\text{mm}}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5 l_{b,rqd} \geq l_{b,min}$$

$$l_{bd} = 1 * 1 * 1 * 637.38\text{mm} = \mathbf{637.38\text{mm}} \geq \mathbf{327.8\text{mm}}$$

7.5. shear design

For the verification of the shear resistance the following symbols are defined:

- $V_{Rd,c}$ is the design shear resistance of the member without shear reinforcement.
- $V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement.
- $V_{Rd,max}$ is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.

$$V_{RD} = V_{RD,S}$$

In regions of members where $V_{ED} \leq V_{RD,C}$, no calculated shear reinforcement is necessary. V_{ED} is design shear force in the section considered resulting from the external loading.

In regions where, $V_{Ed} > V_{Rd,C}$ sufficient shear reinforcement should be provided in order that $V_{Ed} \leq V_{Rd}$

The design shear force should not exceed the permitted maximum value $V_{RD,MAX}$, anywhere in the member.

For members subject to predominantly uniformly distributed loading, the design shear force need not be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{RD,max}$

Members not requiring design shear reinforcement

The design value for the shear resistance $V_{Rd,c}$ is given by:

$$v_{Rd,c} = \left[0.12K(100\rho_1f_{CK}) \right]^{\frac{1}{3}}$$

With minimum of

$$v_{Rd,c} = [V_{min} + K_1\sigma_{cp}]bwd$$

Where:

f_{CK} is in MPa

$$K = 1 + \left(\frac{200}{d} \right)^{0.5} \leq 2$$

with d in mm

$$\rho_1 = \frac{A_{sl}}{bwd}$$

A_{sl} is the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the section considered.

bw is the smallest width of the cross-section in the tensile area (mm)

$$\sigma_{cp} = \frac{N_{ED}}{A_c} < 0.2f_{cd} \text{ [MPa]}$$

N_{ED} is the axial force in the cross-section due to loading or prestressing in newtons ($N_{ED} > 0$ for compression). The influence of imposed deformations on N_E may be ignored.

A_c Is the area of concrete cross section [mm²]

$V_{Rd,c}$ is in newtons

The values of $C_{Rd,c}$, V_{min} and k_1 for use in a country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0.18/\gamma_c$, that for V_{min} is $0.035 k^{3/2} f_{ck}^{1/2}$ and that for k_1 is 0.15. The design of members with shear reinforcement is based on a truss model.

The angle θ should be limited. The limiting values of $\cot\theta$ for use in a country may be found in its National Annex.

The recommended limits are $1 \leq \cot\theta \leq 2.5$.

Members requiring design shear reinforcement

For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of

$$V_{RD,S} = \frac{A_{sw}}{s} Z f_{ywd} \cot\theta \quad , \text{ and}$$

$$V_{RD,max} = \alpha_c b_w Z V f_{cd} / (\cot\theta + \tan\theta)$$

Where

A_{sw} is the cross-sectional area of the shear reinforcement

s is the spacing of the stirrups

f_{ywd} is the design yield strength of the shear reinforcement

V follows from the expression below

For reinforced and prestressed members, if the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk} , V may be taken as:

$$V=0.6$$

$$V=0.9-f_{ck}/200 \quad 0.5 \quad \text{for } f_{ck} \geq 60\text{MPa}$$

The value of $\tan\alpha_c$ for use in a Country may be found in its National Annex. The recommended value is 1 for non-prestressed structures.

Minimum area and maximum spacing of shear reinforcement

The ratio of shear reinforcement is given by

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin\alpha)$$

Where

ρ_w is the shear reinforcement ratio ρ_w should not be less than ρ_{wmin}

A_{sw} is the area of shear reinforcement within length s

S is the spacing of the shear reinforcement measured along the longitudinal axis of

the member

b_w is the breadth of the web of the member

α is the angle between shear reinforcement and the longitudinal axis

When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided. The minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed or hollow core slabs) where transverse redistribution of loads is possible. Minimum reinforcement may also be omitted in members of minor importance which do not contribute significantly to the overall resistance and stability of the structure.

The value of $\rho_{w,min}$ for beams for use in a Country may be found in its National

Annex. The recommended value is $\rho_{w,min} = (0.08\sqrt{f_{ck}})/f_{yk}$

The maximum longitudinal spacing between shear assemblies should not exceed $S_{l,max}$

The value of $S_{l,max}$ for use in a country may be found in its National Annex. The

recommended value is $S_{max} = 0.75d(1 + \cot\alpha)$

where α is the inclination of the shear reinforcement to the longitudinal axis of the beam.

Procedure for design

- Minimum area and maximum spacing of shear reinforcement

Step 1: Determine maximum applied shear force at the face of support, ved

Step 2: Determine $v_{rd,max}$ with $\cot \theta=2.5$

Step 3: If $V_{RD,max} > v_{ed} \cot \theta=2.5$, go to step 6 and calculate required shear reinforcement

Step 4 If $v_{rd,max} < v_{ed}$ calculate required strut angle:

$$\Theta = 0.5 \sin^{-1} \left[\left(\frac{2V_{ED}}{\alpha_{cb} w Z v_{fcd}} \right) \right]$$

Step 5: If $\cot \theta$ is less than 1, re-size element, otherwise go to step 6

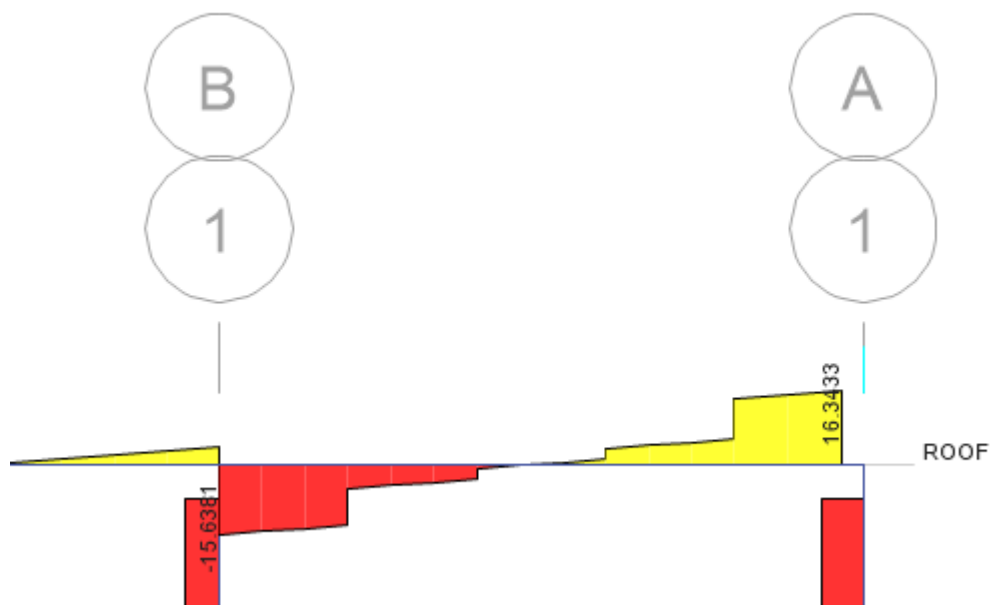
Step 6: Calculate $V_{Rd,c}$

Step 7: If $v_{rd,c} > v_{ed}$, Minimum shear reinforcement is sufficient

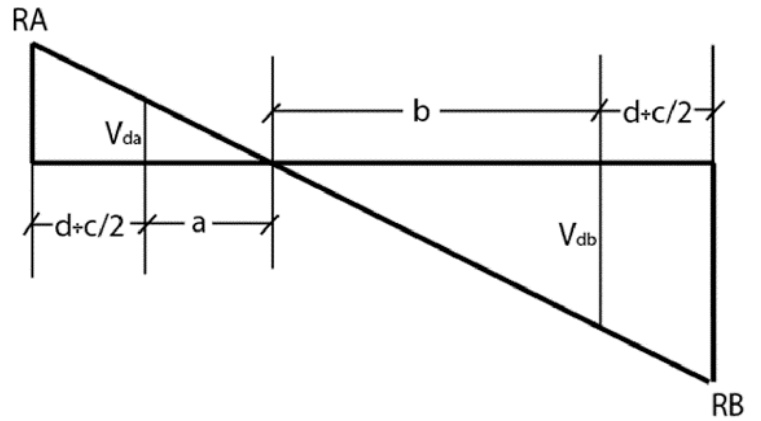
Step 8: If $v_{rd,c} < v_{ed}$, Calculate amount of shear reinforcement required

Step 9: Check min shear reinforcement and maximum spacing

Sample calculation for shear design



RA= 15.63 KN
BB= 16.34 KN
L= 5.7 m
d= 367 mm
C= 300 mm
D+c/2 0.517 m
a= 2.269706 m
b= 2.396294 m
VEda= 12.73026 KN
VEdb= 13.44026 KN
VEd= 13.44026 KN



Concrete capacity check

$$v_{Rd,c} = \left[0.12K(100\rho_1f_{ck})^{\frac{1}{3}} \right] bd \geq V_{min}$$

$$K = 1 + \left(\frac{200}{d} \right)^{0.5} \leq 2$$

$$d = 367$$

$$b = 250$$

$$A_{sl} = 200.96$$

$$k = 1.74 \quad 0$$

$$\rho_1 = \frac{A_{sl}}{bd} \leq 0.02$$

taking minimum reinforcement ϕ 8
@ 260

$$\rho_1 = 0.0022$$

$$v_{Rd,c} = 31.314$$

$$v_{Rd,c} = VR_{dc} > V_{Ed}, \text{ use min spacing, } 15^* \text{ long. Bar dia.}$$

longitudinal bar diam. = 16

minimum spacing = 240

diagonal compression check

$$V_{Rd,max} = \alpha_{cw} b_w Z V_1 f_{cd} \left(\frac{1}{\cot\theta + \tan\theta} \right)$$

$$v_{Rd,c} = \left[0.12K(100\rho_1f_{ck})^{\frac{1}{3}} \right] bd \geq V_{min}$$

$$V_1 = 0.6 \left(1 - \frac{f_{ck}}{250} \right)$$

$$V_1 = 0.552$$

$$f_{cd} = 11.33$$

$$z = 0.9d$$

$$Z = 330.3$$

$$\cot\theta = 2.5$$

$$\tan\theta = 0.4$$

$$V_{Rd,max} = 178.0818$$

$V_{Rd,max} > V_{Ed}$, so this section does not require shear reinforcement for diagonal compression

Calculate the required shear reinforcement

The ratio of shear reinforcement is given by;

$$\rho_w = \frac{A_{sw}}{(s * b_w - \sin\alpha)}$$

Where

ρ_w is the shear reinforcement ratio

ρ_w should not be less than $\rho_{w_{min}}$

A_{sw} is the area of shear reinforcement within length s

s is the spacing of the shear reinforcement measured along the longitudinal axis of

the member

b_w is the breadth of the web of the member

α is the angle between shear reinforcement and the longitudinal axis.

$$\rho_{w_{min}} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} = \frac{0.08\sqrt{20}}{400} = 0.00089$$

➤ The maximum longitudinal spacing of bent-up bars should not exceed $S_{b,max}$

$$S_{b,max} = 0.75d(1 + \cot\alpha) = 0.6 * 367(1 + \cot90) = 262.5\text{mm}$$

$$S_{min} = \frac{A_{sw}}{\rho_w b_w \sin\alpha} = \frac{50.24}{0.00089 \times 250 \times 1} = 225.8$$

Required shear reinforcement

$$\frac{A_{sw}}{s} = \frac{V_{ED}}{0.9df_{yk}\cot\theta}$$

$$S_{calc} = \frac{A_{sw}0.78df_{yk}\cot\theta}{V_{ED}} = \frac{50.24*0.9*367*400}{13.44} = 549.4\text{mm}$$

use $s = 260\text{mm}$

TOP TIE BEAMS ON AXES - 5 (250x400mm)

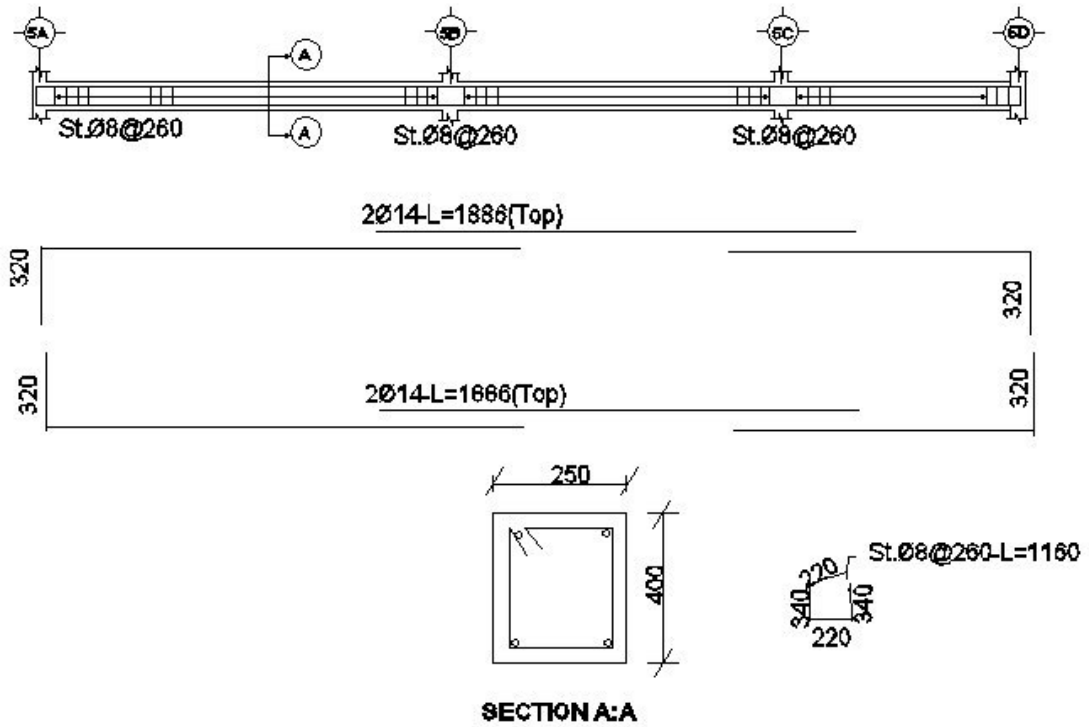


Figure 7-1 sample reinforcement layout for beam

8. Column design

Columns are defined as members that carry loads chiefly in compression, even though the bending action may produce tensile forces over part of their cross section. Columns are designed to transfer the load from beams and slabs (in flat slab) down to the foundation without buckling or crashing. To determine the nature of the frame, we substitute the beams and columns by one substitute frame.

The value of the axial force on each substitute frame column obtained by adding the axial load of each column for the story including self-weight. A column is special case of a compression member that is vertical. Almost all compression members in RC structure subjected to moment in addition to axial loads these may be due to misalignment of load on column, may result from column resisting portion of the unbalanced moment at the ends of the beams supported by the columns/ unbalanced moments from beams.

The bending moments can be converted in to un-equivalent axial load applied at eccentricity . Therefore, columns should be designed for the design moment obtained from the total eccentricity.

Column may be classified based the following criteria

- On the basis of lateral reinforcement; tied columns, spiral columns
- On the basis of manner by which lateral stability is provided to the structure as a whole; braced and unbraced columns
- On the basis of sensitivity to second order effect due to lateral displacements; sway and non-sway columns
- On the basis of degree of slenderness; short and slender columns
- On the basis of loading; axially loaded column, column under uni axial bending, column under bi axial bending.

Axial column; column subjected to axial loads accompanied by bending about one axis whereas biaxial, for column support axial force and bending about two perpendicular axes. A braced structure is one which contain bracing elements. These are vertical elements usually walls, which are so stiff relative to other vertical elements

that may be assumed to be attract all horizontal forces. Braced structure may be defined as one where the bracing elements attract and transmits to the foundations, at least 90% of all horizontal forces applied to the structure.

Braced structure is one where side sway of the whole structure is unlikely to be significant while in unbraced structure sideways likely to be significant, and defined as lateral displacement of the ends of the column increasing the critical bending moment by more than 10% above calculated by ignoring the displacement.

Design procedure

8.1. Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

a. Exposure class = XC1, for Dry or permanently wet Environment like Concrete inside buildings with low air humidity and Concrete permanently submerged in water. [Appendix B](*ES EN 1992-1-1*, 2015)

b. Minimum strength class = C20/25, for exposure class XC1[Appendix B]

c. Minimum concrete cover for bond/durability

d.
$$C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{array} \right\} = \max \left\{ \begin{array}{l} 20mm \\ 10mm \\ 10mm \end{array} \right\} = 12mm$$

i. $C_{min, b} = 20$ mm, assumed diameter of bar [Appendix B]

ii. $C_{min, dur} = 10$ mm,

1. $C_{min} = 20$ mm

iii. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N. The recommended minimum Structural Class is S1

e. Nominal cover

a. $C_{nom} = C_{min} + \delta c_{dev} = 20\text{mm} + 10\text{mm} = 30\text{mm}$

i. $\Delta c_{dev} = 10\text{mm}$, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

f. Minimum cover Design for Fire = 20mm

g. Governing concrete cover = 30mm

8.2. Effective length determination

Effective length is used to account for the shape of the deflection curve: it can also be defined as buckling length i.e. The length of pin-ended column with constant normal force, having the same cross section and buckling load.

Effective length (l_o) for braced member

From (ES EN 1992-1-1, 2015) Art, 5.8.

$$l_o = 0.5l \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) \cdot \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

Where, K_1 , K_2 -relative flexibilities of rotational restraint at both ends 1,2

l -length of column

K_1 -stiffness at end 1

K_2 -stiffness at end 2

Stiffness at each end (K) =column stiffness/ Σ beam stiffness

$$K = \frac{\text{Column stiffness}}{\Sigma \text{Beam stiffness}}$$

$$K_i = \frac{\left(\frac{EI}{l}\right)_c}{\left(\frac{2 * EI}{l}\right)_b}$$

8.3. Check slenderness limit

Slenderness ratio $\lambda = \frac{l_0}{i}$ Where l_0 -effective length of column

i Is radius of gyration of the uncracked concrete In both directions

$$i = \sqrt{\frac{I_c}{A}}$$

The minimum limiting value of slenderness is

$$\lambda_{lim} = \frac{20 * ABC}{\sqrt{n}}$$

where:

$A = 1 / (1 + 0.2 \varphi_{ef})$ (if φ_{ef} is not known, $A = 0.7$ may be used)

$B = \sqrt{1 + 2\omega}$ (if ω is not known, $B = 1.1$ may be used)

$C = 1.7 - r_m$ (if r_m is not known $C = 0.7$ may be used)

φ_{ef} effective creep ratio; see 5.8.4:

$\omega = A_s f_{yd} / (A_c f_{cd})$; mechanical reinforcement ratio;

A_s is the total area of longitudinal reinforcement

$n = N_{Ed} / (A_c f_{cd})$; relative normal force

$r_m = M_{01} / M_{02}$; moment ratio

M_{01}, M_{02} are the first order end moments, $|M_{02}| \geq |M_{01}|$

If the end moments M_{01} and M_{02} give tension on the same side, r_m should be taken positive (i.e. $C \leq 1.7$), otherwise negative (i.e. $C > 1.7$).

In the following cases, r_m should be taken as 1.0 (i.e. $C = 0.7$):

- for braced members in which the first order moments arise only from or predominantly due to imperfections or transverse loading
- for unbraced members in general (ES EN 1992-1-1, 2015) Art 5.8.3.1

$$M_{01} = \min\{|M_{top}|, |M_{bottom}|\} + e_i + NED$$

$$M_{02} = \max\{|M_{top}|, |M_{bottom}|\} + e_i + NED$$

Accidental eccentricity

$$e_i = \max \left\{ \begin{array}{l} \frac{l_o}{400} \\ \frac{h}{30} \\ 20\text{mm} \end{array} \right\} \text{ (ES EN 1992-1-1, 2015) Art 6.1.4}$$

In the both direction

8.4. Calculate reinforcement

$$A_{s,tot} = \frac{\omega * A_c * f_{cd}}{f_{yd}}$$

ω , is taken design chart by using, V_{sd} , $\mu_{sd,x}$, $\mu_{sd,y}$ from (ES EN 1992-1-1, 2015)

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}}$$

$$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h}$$

$$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h}$$

If the column is slender column

$$M_{ED} = \max\{M_{02}; M_{0e} + M_2; M_{01} + 0.5M_2; e_i N_{ED}\}$$

If the column is not slender column(short column)

$$M_{ED} = \max\{M_{02}; e_i N_{ED}\}$$

$$M_{0e} = 0.6M_{02} + 0.4M_{01} \geq 0.4M_{02}$$

Minimum and maximum area of reinforcement

$$A_{s,min} = \max \left\{ \left(0.1 * \frac{N_{ED}}{f_{yd}} \right) \right. \\ \left. 0.002 * A_c \right\}$$

$$A_{s,max} = 0.04A_c$$

For shear reinforcement

$$\text{Spacing} = \min \left\{ \begin{array}{l} 20 * \phi_{\text{long}} \\ \text{lesser dimension of column} \\ 400\text{mm} \end{array} \right\}$$

8.5. Sample calculation

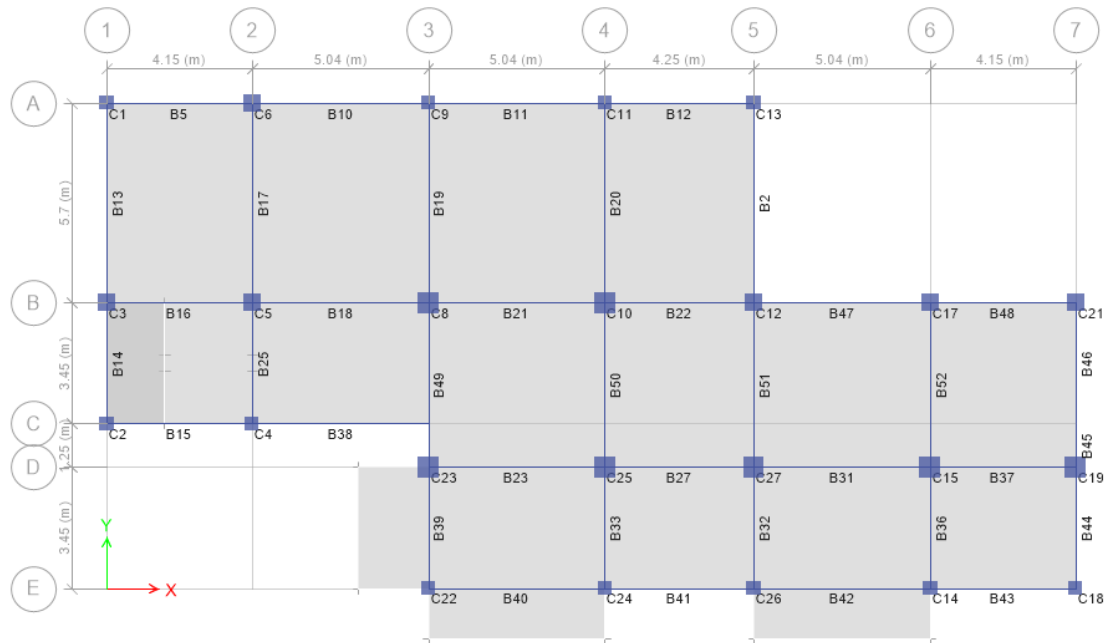


Figure 8-1 column label layout

Effective length determination

Let calculate for C-6 on roof elevation

Given data

$$l_{bxL} = 4150\text{mm}$$

$$l_{bxR} = 5050\text{mm}$$

$$l_{byL} = 0$$

$$l_{bxR} = 5700$$

$$l_c = 2800$$

Beam = 250 X 400

Column = 250 x 400

$$l_o = 0.5l \sqrt{\left(1 + \frac{k_1}{0.45 + k_1}\right) \cdot \left(1 + \frac{k_2}{0.45 + k_2}\right)}$$

$$K_{1x} = \frac{\left(\frac{250 * 400^3}{12}\right)}{\left(\frac{250 * 400^3}{4150}\right) + \left(\frac{250 * 400^3}{5050}\right)} = \frac{476190.48}{321285.14 + 264026.40} = \mathbf{0.81}$$

Same for $K_{2x} = \mathbf{0.81}$

$$K_{1y} = \frac{\left(\frac{250 * 400^3}{12}\right)}{\left(\frac{250 * 400^3}{0}\right) + \left(\frac{250 * 400^3}{5700}\right)} = \frac{476190.48}{233918.13} = \mathbf{2.04}$$

Same for $K_{2y} = \mathbf{2.04}$

$$l_{ox} = 0.5 * 2800 \sqrt{\left(1 + \frac{0.81}{0.45+0.81}\right) \cdot \left(1 + \frac{0.81}{0.45+0.81}\right)} = \mathbf{2301.41}$$

$$l_{oy} = 0.5 * 2800 \sqrt{\left(1 + \frac{2.04}{0.45+2.04}\right) \cdot \left(1 + \frac{2.04}{0.45+2.04}\right)} = \mathbf{2546.55}$$

Check slenderness limit

Slenderness ratio $\lambda = \frac{l_o}{i}$

$$i = \sqrt{\frac{I_c}{A}} = \sqrt{\frac{250*400^3}{12}} = 115.47$$

$$\lambda_x = \frac{l_{ox}}{i} = \frac{2304.41}{115.47} = 19.93$$

$$\lambda_y = \frac{l_{oy}}{i} = \frac{2546.55}{115.47} = 22.05$$

The minimum limiting value of slenderness is

$$\lambda_{lim} = \frac{20 * ABC}{\sqrt{n}}$$

φ_{ef} is not known

$$A=0.7$$

$$B=1.1$$

$$C= 1.7 - rm$$

$$rm = \frac{M_{01}}{M_{02}}$$

$$M_{01} = \min\{|M_{top}|, |M_{bottom}|\} + e_i * NED$$

$$M_{02} = \max\{|M_{top}|, |M_{bottom}|\} + e_i * NED$$

Accidental eccentricity

$$e_{ix} = \max \left\{ \begin{array}{l} \frac{2301.41}{400} = 5.75 \\ \frac{400}{30} = 13.33 \\ 20\text{mm} \end{array} \right\} = \mathbf{20 \text{ mm}}$$

$$e_{iy} = \max \left\{ \begin{array}{l} \frac{2546.55}{400} = 6.37 \\ \frac{400}{30} = 13.33 \\ 20\text{mm} \end{array} \right\} = \mathbf{20 \text{ mm}}$$

$$M_{01x} = \min\{-38.03, |41.63|\} + 0.02 * 139.01 = 40.81$$

$$M_{02x} = \max\{-38.03, |41.63|\} + 0.02 * 139.01 = 44.41$$

$$M_{01y} = \min\{1.89, |-0.218|\} + 0.02 * 139.01 = 2.99$$

$$M_{02y} = \min\{1.89, |-0.218|\} + 0.02 * 139.01 = 4.67$$

$$n = \frac{NED}{A_c f_{cd}} = \frac{139.01}{250 * 400 * 11.33} = 0.123$$

$$\lambda_{x\text{lim}} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{40.81}{44.41})}{\sqrt{0.123}}$$

$$\lambda_{x\text{lim}} = \mathbf{34.32}$$

$$\lambda_{y\text{lim}} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{2.99}{4.67})}{\sqrt{0.123}}$$

$$\lambda_{y\text{lim}} = \mathbf{46.64}$$

19.93 < 34.32, column is not slender

22.05 < 46.64, column is not slender

Calculate reinforcement

$$A_{s,\text{tot}} = \frac{\omega * A_c * f_{cd}}{f_{yd}}$$

ω , is taken design chart by using, V_{sd} , $\mu_{sd,x}$, $\mu_{sd,y}$ from (ES EN 1992-1-1, 2015)

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{139.01 * 1000}{250 * 400 * 11.33} = 0.123$$

$$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h} = \frac{44.41 * 1000000}{11.33 * 250 * 400} = 0.01$$

$$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h} = \frac{4.67 * 1000000}{11.33 * 250 * 400 * 400} = 0.1$$

ω , is taken from biaxial chart on [appendix F] for this sample $\omega = 0.15$

Minimum and maximum area of reinforcement

$$A_{s,\text{min}} = \max \left\{ \left(0.1 * \frac{NED}{f_{yd}} \right), \left(0.002 * A_c \right) \right\}$$

$$A_{s,min} = \max \left\{ \left(0.1 * \frac{139.01}{400} * 1000 \right) = 30.21 \right. \\ \left. 0.002 * 250 * 400 = 200 \right\}$$

$$A_{s,min} = 200$$

$$A_{s,max} = 0.04A_c = 0.04 * 250 * 400 = 4000$$

$$A_{s,tot} = \frac{\omega * A_c * f_{cd}}{f_{yd}} = \frac{0.15 * 250 * 400 * \frac{20}{1.5}}{\frac{400}{1.15}} = 575$$

Let us use ϕ 16 bar

$$\underline{N_o} \text{ of bar} = \frac{A_{st}}{a_{st}} = \frac{575}{\frac{3.14 * 16^2}{4}} = 2.86$$

Use 4 ϕ 16 bar

For shear reinforcement

Take ϕ 8mm

$$\text{Spacing} = \min \left\{ \begin{array}{l} 20 * \phi_{long} = 20 * 16 = 320 \\ \text{lesser dimension of column} = 250 \\ 400\text{mm} \end{array} \right\}$$

Use ϕ 8 c/c 250

Lap Length

The design lap length is:

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,min}$$

Where;

$l_{b,rqd}$ is basic anchorage length

$$l_{o,min} \geq \max \{ 0.3 \alpha_6 l_{b,rqd}; 15\phi; 200 \text{ mm} \}$$

Values of $\alpha_1 \alpha_2 \alpha_3$ and α_5 may be taken from table and α_6 are given in the table

$$l_{o,\min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} \\ 15\phi \\ 200\text{mm} \end{cases}$$

$$\text{Where: } l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}}$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s}$$

$$f_{bd} = 2.25\eta_1\eta_2f_{ctd}$$

$$\eta_1 = 1 \text{ for good quality of bond}$$

$$\eta_1 = 0.7 \text{ for other cases}$$

$$\eta_2 = 1 \text{ for } \phi \leq 32\text{mm}$$

$$\eta_2 = \frac{132-\phi}{100} \text{ for } \phi > 32\text{mm}$$

$$f_{ctd} = \frac{\alpha_{ct}f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

Sample calculation for $\phi 24$ reinforcement bar

$$f_{ctd} = \frac{\alpha_{ct}f_{ctk,0.05}}{\gamma_c}$$

$$\alpha_{ct} = 0.85, \text{ recommended value}$$

$$f_{ctk} = 1.5 \text{ mpa, from ESEN 1992 Table 3.1}$$

$$f_{ctd} = \frac{0.85 * 1.5}{1.5} = \mathbf{0.85}$$

$$f_{bd} = 2.25\eta_1\eta_2f_{ctd}$$

$$\eta_1 = 1, \text{ our Slab D is } < 250\text{mm, good condition of bond}$$

$$\eta_2 = 1, \text{ for } \emptyset \leq 32\text{mm}$$

$$f_{ctd} = 0.85$$

$$f_{bd} = 2.25 * 1 * 1 * 0.85 = \mathbf{1.91}$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = \mathbf{347.83}$$

$$l_{b,rqd} = \frac{\emptyset}{4} * \frac{\sigma_{sd}}{f_{bd}} = \frac{24}{4} * \frac{347.83}{1.91} = \mathbf{1092.66}$$

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{o,min}$$

α_1, α_2 & $\alpha_3 = 1, \alpha_4 = 0.7$, for reinforcement bar in compression

$\alpha_6 = \left(\frac{\rho_1}{25}\right)^{0.5}$ but not exceeding 1.5 nor less than 1.0, where ρ_1 is the percentage of reinforcement lapped within 0.65 l_o from the center of the lap length considered

Let all reinforcement in slab lapped on the same center, therefore $\alpha_6 = 1.4$ from the table value for $\rho_1 > 50\%$,

$$l_{o,min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} = 0.3 * 1.4 * \mathbf{1092.66} = \mathbf{458.92mm} \\ 15\emptyset = 15 * 12 = 180mm \\ 200mm \end{cases}$$

$$l_{o,min} = \mathbf{458.92}$$

$$l_o = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_{b,rqd} = 1 * 0.7 * 0.7 * 0.7 * 1.4 * 1092.66 = \mathbf{524.7mm} \approx \mathbf{600mm}$$

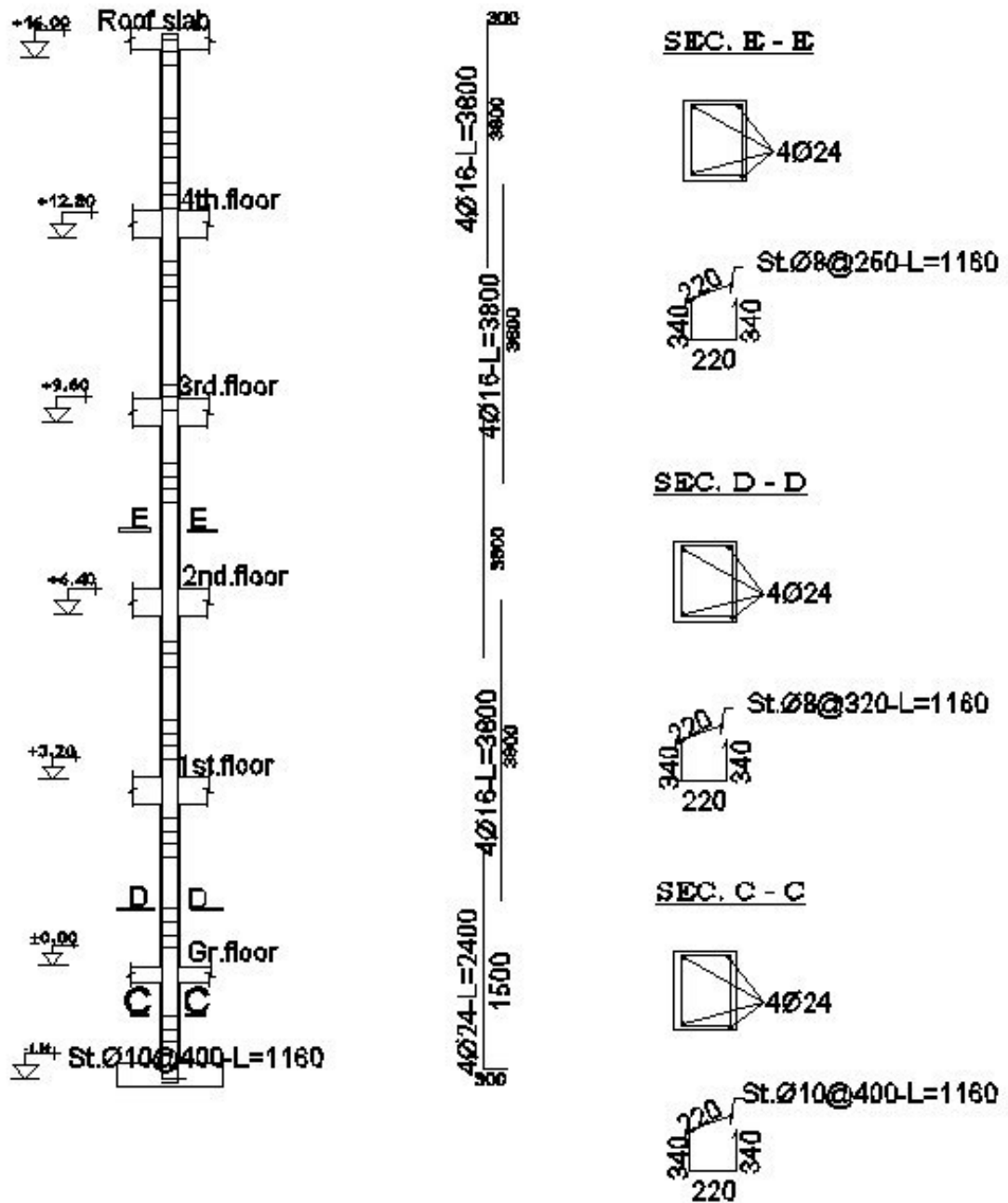


Figure 8-2 sample reinforcement layout for column

9. Foundation design

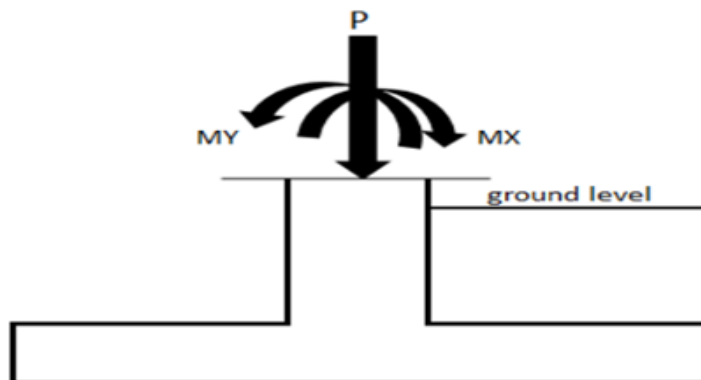
The foundation is the part of an engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and its self-weight.

Purposes of foundations to distribute the load of the structure over a large bearing area so as to bring intensity of loading within the safe bearing capacity of the soil lying underneath. To load the bearing surface at a uniform rate so as to prevent unequal settlement.

- ✓ To prevent the lateral movement of the supporting material.
- ✓ To secure a level and firm bed for building operations.
- ✓ To increase the stability of the structure as a whole.

The foundation should be design such that

- ✓ The soil below doesn't fail in shear
- ✓ The settlement is within the safe limit



Concrete cover design

Cover is designed according to (*ES EN 1992-1-1*, 2015)

- e. Exposure class = XC4, for cyclic wet and dry, concrete surfaces subject to water contact, not within exposure class XC2. [Appendix B]

f. Minimum strength class = C30/37, for exposure class XC4 [Appendix B]

g. Minimum concrete cover for bond/durability

$$C_{min} = \max \begin{Bmatrix} C_{min, B} \\ C_{min, Dur} \\ 10mm \end{Bmatrix} = \max \begin{Bmatrix} 32mm \\ 15mm \\ 10mm \end{Bmatrix} = 32mm$$

$C_{min, b} = 32$ mm, assumed diameter of bar [Appendix B]

$C_{min, dur} = 15$ mm,

The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths given in Annex E and the recommended modifications to the structural class is given in Table 4.3N(ES EN 1992-1-1, 2015). The recommended minimum Structural Class is S1

$C_{min} = 32$ mm

h. Nominal cover

$$C_{nom} = C_{min} + \delta c_{dev} = 32mm + 10mm = 42mm$$

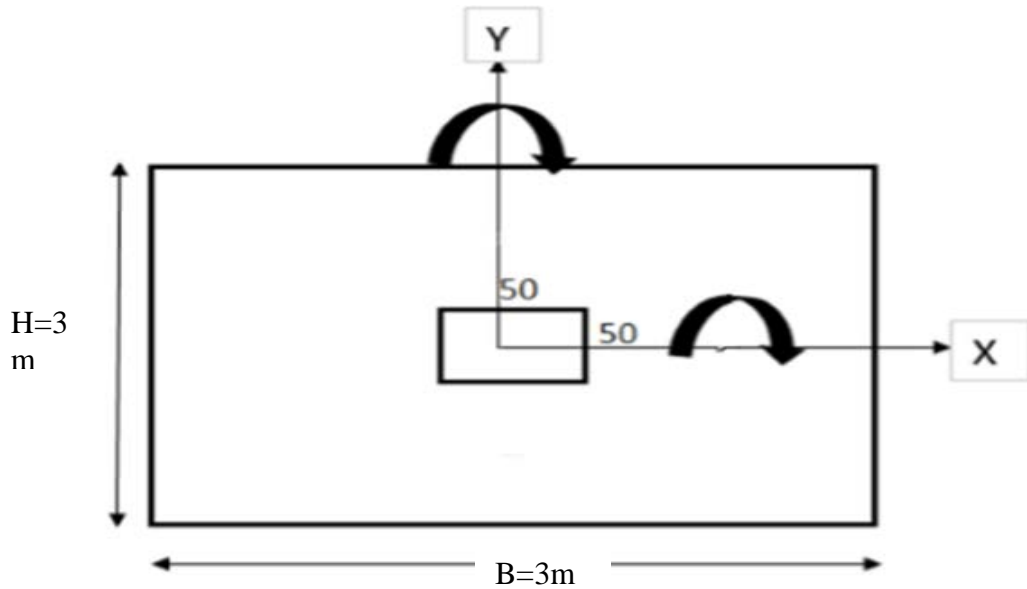
$\Delta c_{dev} = 10$ mm, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.

g) Minimum cover Design for Fire = 20mm

h) Governing concrete cover = 42mm \approx 50mm

Area proportioning

First let us assume that the footing pad is square



Dimension of column is 500x500mm

From bearing capacity of the soil using the theoretical method

Service axial, $N=p=2050\text{KN}/1.4=1464.29\text{kN}$

Service moment= 50Kn/m

Assumed the self-weight= 10% of the service load or axial= $205\text{Kn}/1.4=146.429\text{kN}$

Area of footing required = $\left(\frac{N+W}{\gamma_{soil}}\right)$, $\gamma_{soil} = 150\text{N/mm}^2$

$$A = \left(\frac{1464.29 + 146.429}{150}\right) = 10.74\text{m}^2$$

❖ Try square footing size , $B*H*h=3*3*0.65$

$$A = 3*3 = 9\text{m}^2$$

Self-weight, $W=A*h*\text{unit weight of concrete}$

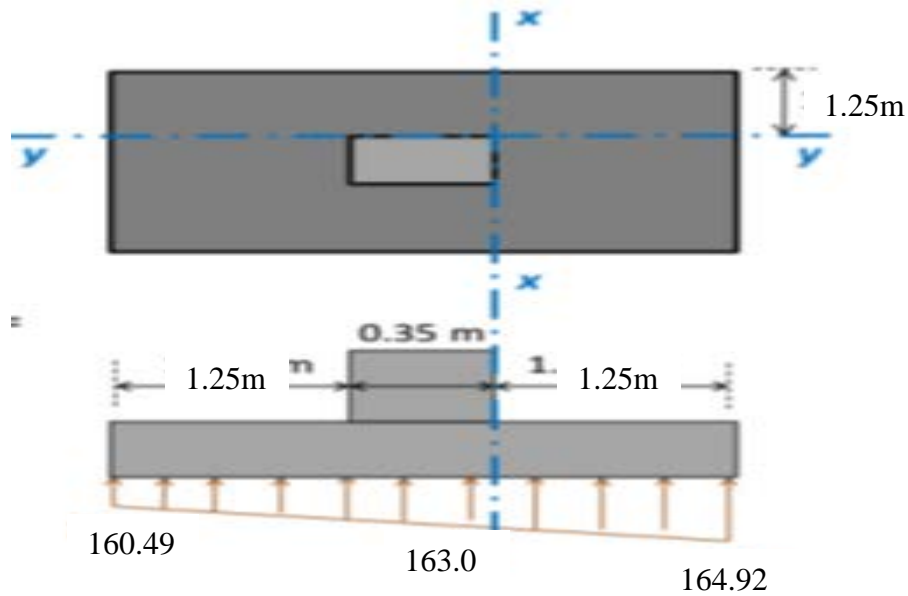
$$W = 9*0.65*25 = 146.25\text{Kn}$$

$$\sigma = \frac{P}{A} \left(1 \pm \frac{6ey}{B} \pm \frac{6ex}{B}\right)$$

$$ex = \frac{MY}{P} = \frac{4}{2050} = 0.00195 \quad ey = \frac{MX}{P} = \frac{10}{2050} = 0.00487$$

$$\sigma_{\max} = \frac{146.29}{9} \left(1 + \frac{6 \cdot 0.00487}{3} + \frac{6 \cdot 0.00195}{3} \right) = 164.92 \text{ kN/mm}^2$$

$$\sigma_{\min} = \frac{146.29}{9} \left(1 - \frac{6 \cdot 0.00487}{3} - \frac{6 \cdot 0.00195}{3} \right) = 160.49 \text{ kN/mm}^2$$



$$M_{xx} = \left(163.07 \cdot \frac{1.25^2}{2} \right) + (164.92 - 160.49) \cdot \left(\frac{1.25}{2} \right) \cdot \left(1.25 \cdot \frac{2}{3} \right)$$

$$= 195.96 \text{ kN.m}$$

$$M_{yy} = \frac{164.92 + 160.49}{2} \cdot \left(\frac{1.25^2}{2} \right) = 127.11 \text{ kN.m}$$

Effective depth

$$dx = h - c - 0.5 \phi_{\text{bar}} \quad \text{use C-40 and}$$

$$= 650 - 40 - 0.5 \cdot 32$$

$$= 594 \text{ mm}$$

$$dy = h - c - 1.5 \phi_{\text{bar}} = 650 - 40 - 1.5 \cdot 32$$

$$= 562 \text{ mm}$$

Main reinforcement -longitudinal bar

$$K = \frac{M_{xx}}{f_c d b d^2} = 0.0058 < K_{bal} = 0.167$$

$$K = \frac{195.96 \times 10^6}{32 \times 3000 \times 594^2} = 0.0058 < K_{bal} = 0.167 \dots \dots \dots \text{ok}$$

Compression reinforcement is not required

Obtain lever arm Z

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53K}] \leq 0.95d$$

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53 \times 0.0058}]$$

$$= 0.495d \leq 0.95d \dots \dots \dots \text{ok}$$

$$= 0.995 \times 0.594 \leq 0.95 \times 0.594$$

$$= 0.564$$

$$A_{s, req} = \frac{M_{xx}}{0.87 f_y d Z} = \frac{195.96 \times 10^6}{0.87 \times 400 \times 0.564} = 998.41 \text{ mm}^2$$

$$A_{s, min} = \frac{0.26 \times f_{ctm} \times b d}{f_{yk}} \text{ where } f_{ck} \geq 25$$

$$A_{s, min} = \frac{0.26 \times 3.2 \times 3000 \times 594}{400} = 3706.56 \text{ mm}^2$$

$$A_{s, max} = 0.04 A_c = 0.04 \times b h$$

$$A_{s, max} = 0.04 A_c = 0.04 \times 3000 \times 594 = 71280 \text{ mm}^2$$

Since $A_s < A_{s, min}$ use $A_{s, min} = 3706.56 \text{ mm}^2$

Number of bar = $3706.56 / (3.14 \times 24^2 / 4) = 9 \text{ } \Phi 24$

Main reinforcement- transverse bar-transverse bar

$$K = \frac{M_{yy}}{f_{cdb} * d^2} = < K_{bal} = 0.167$$

$$K = \frac{127.11 * 10^6}{32 * 3000 * 562^2} = 0.00419 < K_{bal} = 0.167$$

Compression reinforcement is not required

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53K}] \leq 0.95d$$

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53 * 0.00419}] = 0.996d \leq 0.95d = 0.5339d$$

$$A_{s, req} = \frac{M_{yy}}{0.87 f_{yk} Z}$$

$$A_{s, req} = \frac{127.11 * 10^6}{0.87 * 400 * 0.5339} = 684.13 \text{ mm}^2$$

$$A_{s, min} = \frac{0.26 * f_{ctm} * b d}{f_{yk}} \text{ where } f_{ck} \geq 25$$

$$A_{s, min} = \frac{0.26 * 3.2 * 3000 * 562}{400} = 3506.88 \text{ mm}^2$$

$$A_{s, max} = 0.04 A_c = 0.04 * b h$$

$$A_{s, max} = 0.04 A_c = 0.04 * 3000 * 562 = 67440 \text{ mm}^2$$

Since $A_s < A_{s, min}$ use $A_{s, min} = 3506.88 \text{ mm}^2$

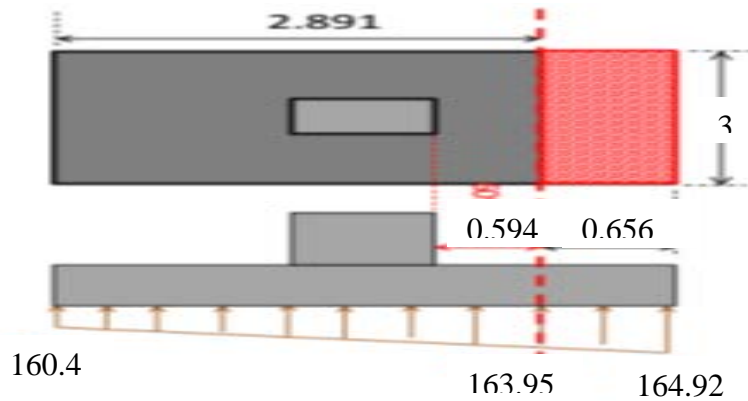
Number of bar = $3506.88 / (3.14 * 24^2 / 4) = 8 \text{ } \Phi 24$

i. Vertical shear (wide beam shear)

Critical shear at 1.0d from face of column

$$\text{Average pressure at critical section} = 160.49 + \frac{2.344}{3} * 4.43 = 163.95 \text{ kN/m}^2$$

2.344



Design shear force, $V_{Ed} = 163.95 \times 0.656 \times 3 = 322.65 \text{ kN}$

Note bar extend beyond critical section at $656 - 35 = 606 > (l_{bd} + d) = 36\phi + d$ use $\phi 32$

$$= 36 \times 32 + 594 = 1746 \text{ mm}, A_{sl} = 0 \text{ mm}^2$$

$$\rho_l = \frac{A_{sl}}{bd} = 0$$

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{594}} = 1.58 < 2$$

$$V_{rdc} = 0$$

$$V_{min} = [0.035 * K / 2 * \sqrt{f_{ck}}] bd$$

$$V_{min} = [0.035 * 1.58^{3/2} * \sqrt{32}] * 300 * 594 = 700.707 \text{ kN}$$

$$V_{Ed} (322.65 \text{ kN}) < V_{min} (700.707 \text{ kN}) \dots \dots \dots \text{ok}$$

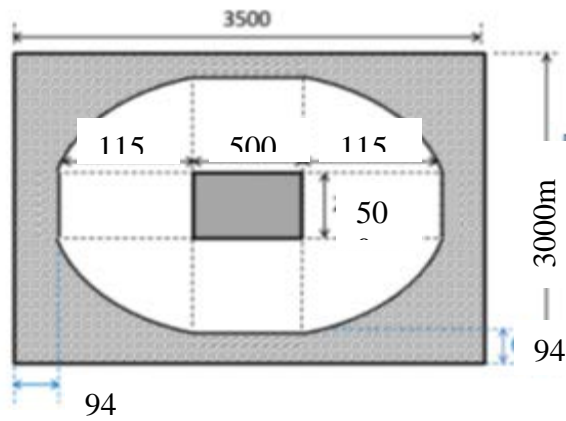
ii. Punching shear

Critical shear at $2d$ from face of column

$$\text{Average } d = \frac{D + d_y}{2} = \frac{594 + 562}{2} = 578 \text{ mm}$$

$$2d = 2 * 578 = 1156 \text{ mm}$$

$$3000 \text{ mm}$$



Control perimeter

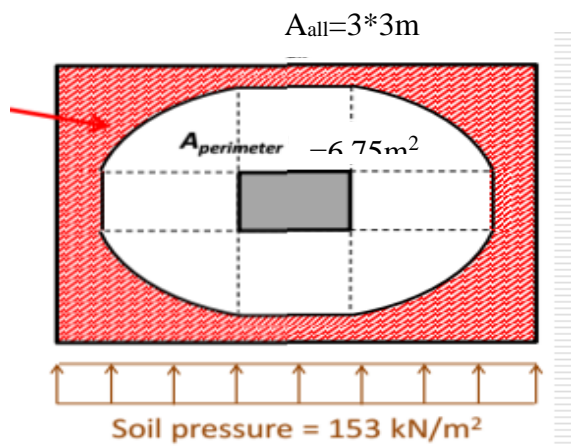
$$U_1 = 2(500 + 500) + (2\pi * 1156) = 92560 \text{ mm}$$

Area within perimeter

$$A = 0.5 * 0.5 + 2(0.5 * 1.156) + 2 * 0.5 * 1.156 + \pi * 1.156^2$$

$$6.756 \text{ m}^2$$

Average punching shear force at control perimeter



$$V_{Ed} = 153(3*3 - 6.75) = 344.25 \text{ kN}$$

Punching shear stress

$$v_{Ed} = \frac{\beta V_{Ed}}{ud}$$

Where

$$\beta = 1 + k \left(\frac{M_{Ed}}{V_{Ed}d} \right) \left(\frac{u_1}{w_1} \right)$$

$$K = 0.65 \quad \frac{c_1}{c_2} = \frac{500}{500} = 1$$

$$W_1 = 0.5C_1^2 + C_1C_2 + 4C_2d = 16d^2 + 2 \Pi d C_1$$

$$= 0.5 * 500^2 + 500 * 500 + 4 * 500d = 16d^2 + 2 \Pi d C_1$$

$$= 0.5 * 500^2 + 500 * 500 + 4 * 500d + 16578^2 + 2 \Pi * 578 * 500$$

$$= 8.69 * 10^6 \text{ mm}^2$$

$$\beta = 1 + 0.65 * \left(\frac{50 * 10^6}{344.25} \right) \left(\frac{92560}{8.69 * 10^6} \right) = 1.101$$

$$V_{Ed} = \frac{1.101 * 344.25 * 10^3}{9260 * 594} = 0.069 \text{ N/mm}^2$$

Punching shear resistance

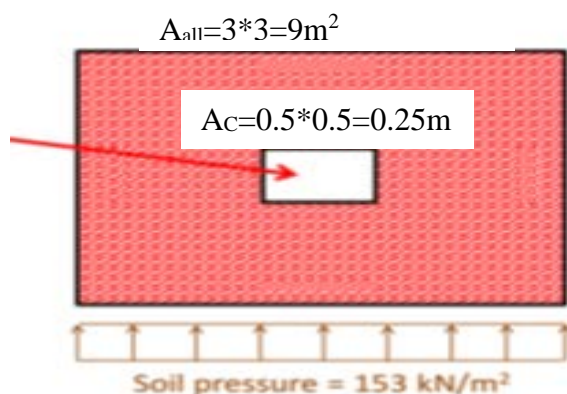
$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{594}} = 1.58 < 2 \dots \dots \dots \text{ok}$$

$$V_{Rd,c} = V_{min} = [0.035 * K^{3/2} * \sqrt{f_{ck}}]$$

$$V_{Rd,c} = [0.035 * 1.58^{3/2} * \sqrt{32}] = 0.221 \text{ N/mm}^2 > V_{Ed} = 0.069 \text{ N/mm}^2$$

iii. Maximum punching shear at column perimeter

$$V_{Ed,max} = 1464.29 \text{ kN}$$



Column column perimeter (V_0)

$$V_0=2(500+500)=2000\text{mm}$$

Punching shear stress

$$V_{Ed} = \frac{\beta V E d}{u d}$$

$$W_1=0.5C_1^2+C_1C_2=0.5*500^2+500*500=0.375*10^6\text{mm}^2$$

$$\beta = 1 + 0.65 * \left(\frac{50*10^6}{14674.26*10^3}\right)\left(\frac{2000}{0.375*10^6}\right)=1.085$$

$$V_{Ed} = \frac{1.085*1464.29*10^3}{2000*588}=1.89\text{N/mm}^2$$

Maximum shear resistance

$$V_{Ed,max}=0.5 \left[0.6 \left(1 - \frac{f_{ck}}{500}\right)\right] * \frac{f_{ck}}{1.5}$$

$$V_{Ed,max}=0.5 \left[0.6 \left(1 - \frac{32}{500}\right)\right] * \frac{32}{1.5}=5.99\text{N/mm}^2 > V_{Ed} \dots \dots \dots \text{ok}$$

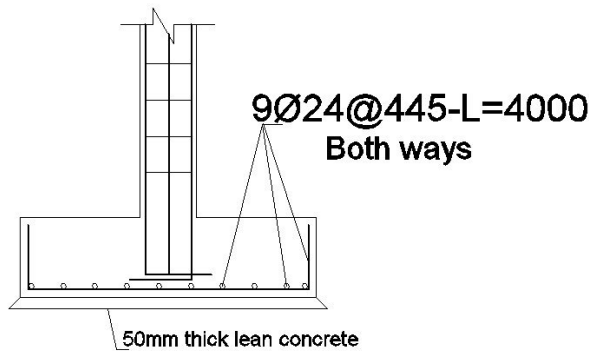


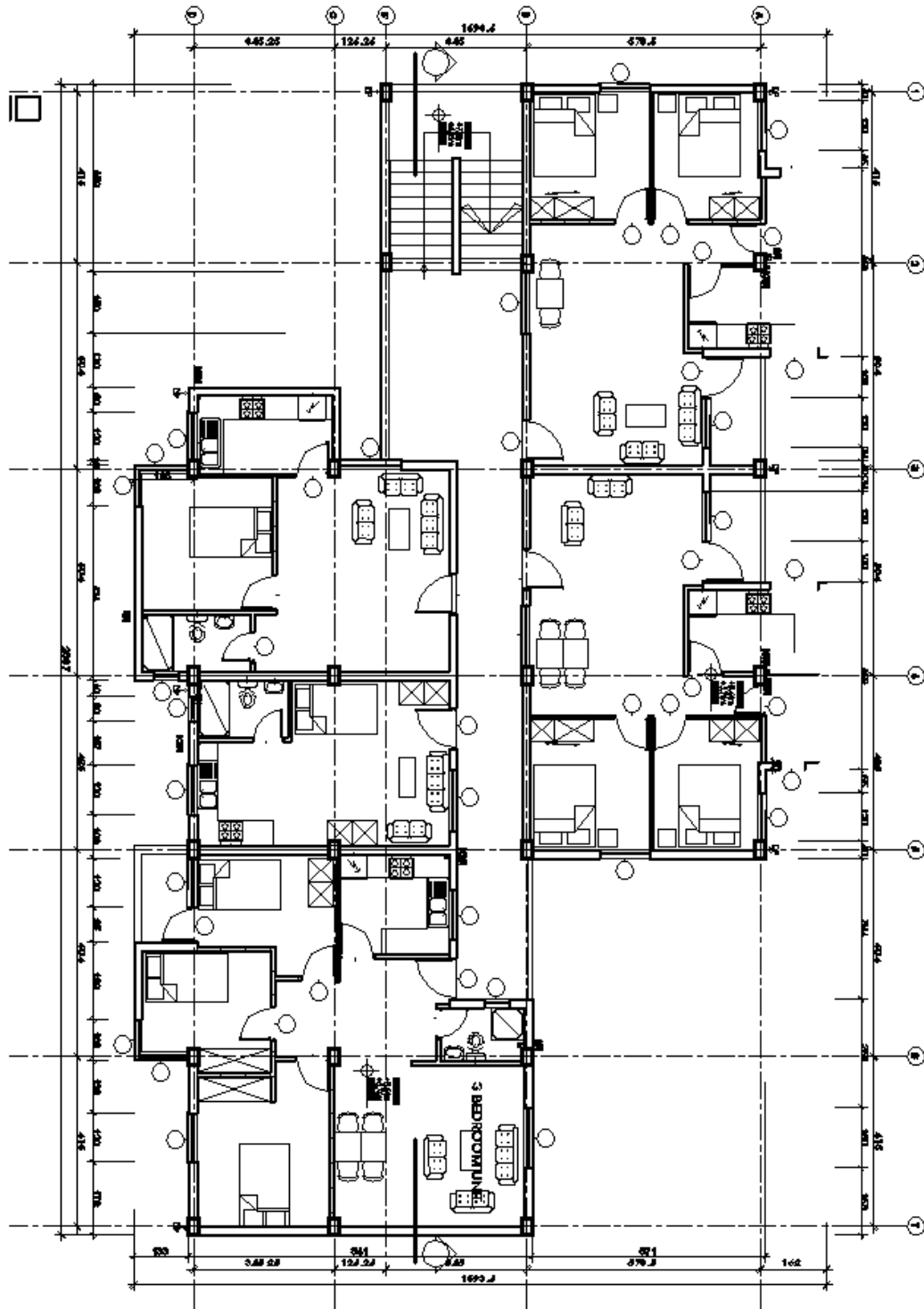
Figure 9-1 footing reinforcement layout

10.Reference

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- Kassimali, A. (2011). Structural analysis, fourth edition, SI. christopher M.shortt.

11. Appendix

Appendix A (architectural design review) Typical 1st- 4th floor



Appendix B (Acquired tables from ESEN)*Table 10-1 minimum cover $C_{min,b}$ requirements with regard to bond*

Bond Requirement	
Arrangement of bars	Minimum cover $C_{min,b}$ *
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n) (see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $C_{min,b}$, should be increased by 5 mm.	

Table 10-2 Exposure classes related to environmental conditions in accordance with ES-EN 206.1

Class Designation	Description of the Environment	Informative examples where exposure classes may occur
1. No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3. Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4. Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surface exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to ES-EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to ES-EN 206-1, Table 2	Natural soils and ground water
XA3	High aggressive chemical environment according to ES-EN 206-1, Table 2	Natural soils and ground water

Table 10-3 Recommended structural classification

Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design working life of 100 years	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2
Strength Class ¹⁾²⁾	≥C30/37 reduce class by 1	≥C30/37 reduce class by 1	≥C35/45 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C40/50 reduce class by 1	≥C45/55 reduce class by 1
Member with slab geometry (Position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

 Table 10-4 values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table 10-5 basic ratios of span/effective depth for reinforced concrete members without axial compression.

Structural System	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one – or two-way spanning simply supported slab	11.0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	11.3	18	26
Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.

Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.

Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory

Table 10-6 Bending moment coefficient for rectangular panel

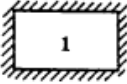
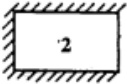
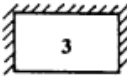
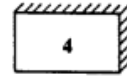
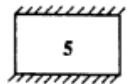
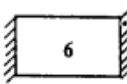
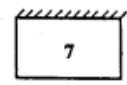
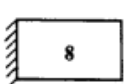
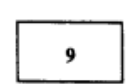
Support Condition	Coeff.	Values of L_y/L_x								Long span coefficients, α_m and α_p for all values of L_y/L_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
	α_m α_p	0.032 0.024	0.037 0.028	0.042 0.032	0.046 0.035	0.050 0.037	0.053 0.040	0.059 0.044	0.063 0.048	0.032 0.024
	α_m α_p	0.039 0.029	0.044 0.033	0.048 0.036	0.052 0.039	0.055 0.041	0.058 0.043	0.063 0.047	0.067 0.050	0.039 0.029
	α_m α_p	0.039 0.030	0.049 0.036	0.056 0.042	0.062 0.047	0.068 0.051	0.073 0.055	0.082 0.062	0.089 0.067	0.039 0.030
	α_m α_p	0.047 0.036	0.056 0.042	0.063 0.047	0.069 0.051	0.074 0.055	0.078 0.059	0.087 0.065	0.093 0.070	0.047 0.036
	α_m α_p	0.046 0.034	0.050 0.038	0.054 0.040	0.057 0.043	0.060 0.045	0.062 0.047	0.067 0.050	0.070 0.053	- 0.034
	α_m α_p	- 0.034	- 0.046	- 0.056	- 0.065	- 0.072	- 0.078	- 0.091	- 0.100	0.045 0.034
	α_m α_p	0.057 0.043	0.065 0.048	0.071 0.053	0.076 0.057	0.081 0.060	0.084 0.063	0.092 0.069	0.098 0.074	- 0.044
	α_m α_p	- 0.044	- 0.054	- 0.063	- 0.071	- 0.078	- 0.084	- 0.096	- 0.105	0.058 0.044
	α_p	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

Table 10-7 factors for adjusting span moments M_{xf} and M_{yf}

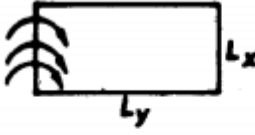
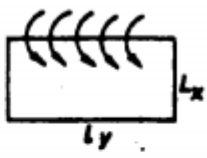
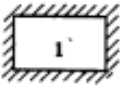
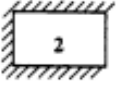
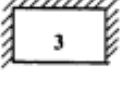
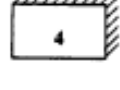
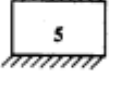
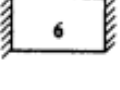
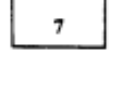
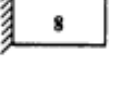
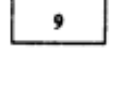






L_y/L_x				
	c_x	c_y	c_x	c_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481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

Table 10-8 shear force coefficients for uniformly loaded rectangular panels

Type of panel and location	Edge	β_x for values of L_y/L_x								β_y
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
	Continuous	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinuous	-	-	-	-	-	-	-	-	0.24
	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	-
	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
	Continuous	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
	Discontinuous	-	-	-	-	-	-	-	-	0.26
	Continuous	-	-	-	-	-	-	-	-	0.40
	Discontinuous	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
	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
	Continuous	-	-	-	-	-	-	-	-	0.45
	Discontinuous	0.30	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
	Discontinuous	-	-	-	-	-	-	-	-	0.33

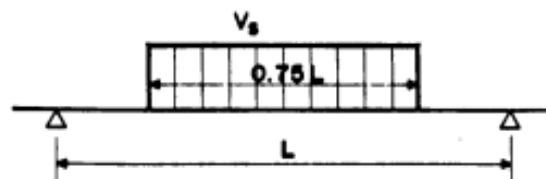


Table 10-9 α_1 α_2 α_3 α_4 α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1.0$	$\alpha_1 = 1.0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_1 = 0.7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1.0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1.0$
Concrete cover	Straight	$\alpha_2 = 1 - 0.15 (c_d - \phi) / \phi$ ≥ 0.7 ≤ 1.0	$\alpha_2 = 1.0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_2 = 1 - 0.15 (c_d - 3\phi) / \phi$ ≥ 0.7 ≤ 1.0 (see Figure 8.3 for values of c_d)	$\alpha_2 = 1.0$
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ ≥ 0.7 ≤ 1.0	$\alpha_3 = 1.0$
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e)	$\alpha_4 = 0.7$	$\alpha_4 = 0.7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0.04p$ ≥ 0.7 ≤ 1.0	-

Appendix C (Analysis and design of slab)**C-1 ,Design load calculation for Typical 1st- 4th floor****P-1 & P-4**

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.108	1.52
		(2.88*0.1*8.9)					
23.655		2.5632					
LIVE LOAD Q _k					2	KN/m ²	
DEAD LOAD G _k					8.15	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					13.998	KN/m ²	

P-2 & P-3

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
LIVE LOAD Q _k					4	KN/m ²	
DEAD LOAD G _k					6.63	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					14.9505	KN/m ²	

P-5

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed & terrazo				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.114	1.60
		(2.88*0.2*5.04)					
25.38		2.90304					
LIVE LOAD Q _k					4	KN/m ²	
DEAD LOAD G _k					8.23	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					17.112	KN/m ²	

P-6

Structural design of G+4 Apartment + shop building B.Sc Thesis project

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed & terrazo				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.167	2.34
		2.88*0.2*4.25					
	14.66	2.448					
LIVE LOAD Q _k					4	KN/m ²	
DEAD LOAD G _k					8.97	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					18.107	KN/m ²	

P-7

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed & terrazo				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.168	2.35
		2.88*0.2*6.89					
	23.688	3.96864					
LIVE LOAD Q _k					4	KN/m ²	
DEAD LOAD G _k					8.98	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					18.117	KN/m ²	

P-8

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
f.f.cement screed				concrete	23	0.030	0.69
f.f.ceiling plaster				concrete	23	0.030	0.69
LIVE LOAD Q _k					2	KN/m ²	
DEAD LOAD G _k					6.63	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					11.9505	KN/m ²	

P-10

Structural design of G+4 Apartment + shop building B.Sc Thesis project

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.117	1.63
		17.39					
LIVE LOAD Q _k					2	KN/m ²	
DEAD LOAD G _k					8.26	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					14.154	KN/m ²	

P-11

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
cement screed				concrete	23	0.030	0.69
ceiling plaster				concrete	23	0.030	0.69
partition load	A _{slab} (m ²)	V _{partition}	A/V	HCB	14	0.385	5.39
		3.44					
LIVE LOAD Q _k					2	KN/m ²	
DEAD LOAD G _k					12.02	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					19.229	KN/m ²	

P-12, P-9 and P-13

				Material	Unit weight	Dimension	Load KN/m ²
Slab				Reinforced concrete	25	0.210	5.25
f.f.cement screed				concrete	23	0.030	0.69
f.f.ceiling plaster				concrete	23	0.030	0.69
LIVE LOAD Q _k					2	KN/m ²	
DEAD LOAD G _k					6.63	KN/m ²	
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					11.9505	KN/m ²	

C-1, C2 and C3

Structural design of G+4 Apartment + shop building B.Sc Thesis project

	Material	Unit weight	Dimension	Load KN/m²
Slab	Reinforced concrete	25	0.210	5.25
f.f.cement screed	concrete	23	0.030	0.69
f.f.celling plaster	concrete	23	0.030	0.69
LIVE LOAD Q_k		2	KN/m ²	
DEAD LOAD G_k		6.63	KN/m ²	
DESIGN LOAD $P_d = 1.35 G_k + 1.5 Q_k$		11.9505	KN/m ²	

C-2 analysis of individual panel moment for typical 1st-4th floor

panel	support condition	ly/lx	lx(m)	lx ²	pd	moment coefficient		moment location	moment $M_i = \alpha_i P_d L^2$
						location	value		
P-1	Type 4	1.37	4.15	17	14	α_{xs}	0.073	$M_{xs} =$	17.60
						α_{xf}	0.054	$M_{xf} =$	13.02
						α_{ys}	0.047	$M_{ys} =$	11.33
						α_{yf}	0.036	$M_{yf} =$	8.68
P-2	Type 2	1.13	5.04	25	14.95	α_{xs}	0.045	$M_{xs} =$	17.09
						α_{xf}	0.034	$M_{xf} =$	12.91
						α_{ys}	0.039	$M_{ys} =$	14.81
						α_{yf}	0.029	$M_{yf} =$	11.01
P-3	Type 2	1.13	5.04	25	14.95	α_{xs}	0.045	$M_{xs} =$	17.09
						α_{xf}	0.034	$M_{xf} =$	12.91
						α_{ys}	0.039	$M_{ys} =$	14.81
						α_{yf}	0.029	$M_{yf} =$	11.01
P-4	Type 4	1.34	4.25	18	14	α_{xs}	0.072	$M_{xs} =$	18.21
						α_{xf}	0.053	$M_{xf} =$	13.40
						α_{ys}	0.047	$M_{ys} =$	11.89
						α_{yf}	0.036	$M_{yf} =$	9.10
P-5	Type 4	1.07	4.70	22	17.11	α_{xs}	0.056	$M_{xs} =$	21.17
						α_{xf}	0.042	$M_{xf} =$	15.87
						α_{ys}	0.047	$M_{ys} =$	17.76
						α_{yf}	0.036	$M_{yf} =$	13.61
P-6	Type 1	1.11	4.25	18	18.1	α_{xs}	0.037	$M_{xs} =$	12.10
						α_{xf}	0.028	$M_{xf} =$	9.15
						α_{ys}	0.032	$M_{ys} =$	10.46
						α_{yf}	0.024	$M_{yf} =$	7.85
P-7	Type 1	1.07	4.70	22	18.1	α_{xs}	0.035	$M_{xs} =$	13.99
						α_{xf}	0.027	$M_{xf} =$	10.80
						α_{ys}	0.032	$M_{ys} =$	12.79
						α_{yf}	0.024	$M_{yf} =$	9.60
P-8	Type 3	1.13	4.15	17	11.95	α_{xs}	0.051	$M_{xs} =$	10.50
						α_{xf}	0.038	$M_{xf} =$	7.82
						α_{ys}	0.039	$M_{ys} =$	8.03
						α_{yf}	0.03	$M_{yf} =$	6.17
P-9	Type 4	1.46	3.45	12	11.95	α_{xs}	0.077	$M_{xs} =$	10.95
						α_{xf}	0.057	$M_{xf} =$	8.11
						α_{ys}	0.047	$M_{ys} =$	6.69
						α_{yf}	0.036	$M_{yf} =$	5.12
P-10	Type 1	1.23	3.45	12	14.15	α_{xs}	0.044	$M_{xs} =$	7.41
						α_{xf}	0.033	$M_{xf} =$	5.56
						α_{ys}	0.032	$M_{ys} =$	5.39
						α_{yf}	0.024	$M_{yf} =$	4.04
P-11	Type 3	1.46	3.45	12	19.23	α_{xs}	0.071	$M_{xs} =$	16.25
						α_{xf}	0.054	$M_{xf} =$	12.36
						α_{ys}	0.039	$M_{ys} =$	8.93
						α_{yf}	0.03	$M_{yf} =$	6.87
P-12	Type 1	1.20	3.45	12	11.95	α_{xs}	0.042	$M_{xs} =$	5.97
						α_{xf}	0.032	$M_{xf} =$	4.55
						α_{ys}	0.032	$M_{ys} =$	4.55
						α_{yf}	0.024	$M_{yf} =$	3.41
P-13	Type 4	1.46	3.45	12	11.95	α_{xs}	0.077	$M_{xs} =$	10.95
						α_{xf}	0.058	$M_{xf} =$	8.25
						α_{ys}	0.047	$M_{ys} =$	6.69
						α_{yf}	0.036	$M_{yf} =$	5.12

C-2 support moment adjustment for typical 1-4 floor

Panel P-1 and P-2 –on axis 2 adjustment is required.

$$\begin{aligned} \Delta M_s &= 17.6 - 17.09 = 0.51 \text{ KNm/m} \\ 0.2M_{s,\max} &= 0.2 \times 17.6 = 3.52 \text{ KNm/m} \\ \Delta M_s &= 0.51 < 0.2M_{s,\max} = \text{TRUE} \end{aligned}$$

Therefore, average of the moments can be taken.

$$M_{s,\text{avg}} = \frac{1}{2} (17.6 + 17.09) = 17.35 \text{ KNm/m}$$

Panel P-3 and P-4 –on axis 4 adjustment is required.

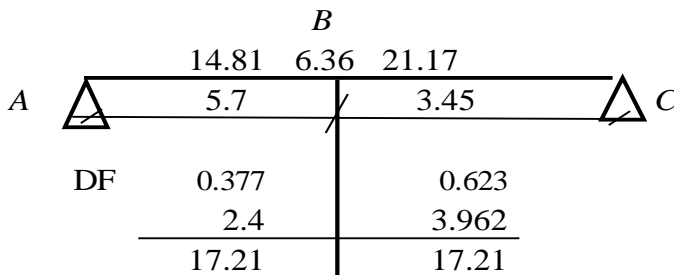
$$\begin{aligned} \Delta M_s &= 17.09 - 18.21 = -1.12 \text{ KNm/m} \\ 0.2M_{s,\max} &= 0.2 \times 17.09 = 3.418 \text{ KNm/m} \\ \Delta M_s &= -1.12 < 0.2M_{s,\max} = \text{TRUE} \end{aligned}$$

Therefore, average of the moments can be taken.

$$M_{s,\text{avg}} = \frac{1}{2} (17.09 + 18.21) = 17.65 \text{ KNm/m}$$

Panel P-2 and P-5 –on axis B adjustment is required.

$$\begin{aligned} \Delta M_s &= 21.17 - 14.81 = 6.36 \\ 0.2M_{s,\max} &= 0.2 \times 21.17 = 4.234 \\ \Delta M_s &= 6.36 > 0.2M_{s,\max} \text{ FALSE} \\ &\text{moment distribution method required} \end{aligned}$$



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 5.7} = 0.132 \text{ I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 3.47} = 0.22 \text{ I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.377$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.623$$

Panel P-3 and P-6 –on axis B adjustment is require

$$\Delta M_s = 14.81 - 12.1 = 2.71$$

$$0.2M_{s,max} = 0.2 \times 14.81 = 2.962$$

$$\Delta M_s = 2.71 < 0.2M_{s,max} \quad \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (14.81 + 12.1) = 13.46$$

Panel P-4 and P-7 –on axis B adjustment is require

$$\Delta M_s = 12.79 - 11.89 = 0.9$$

$$0.2M_{s,max} = 0.2 \times 12.79 = 2.558$$

$$\Delta M_s = 0.9 < 0.2M_{s,max} \quad \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (12.79 + 11.89) = 12.34$$

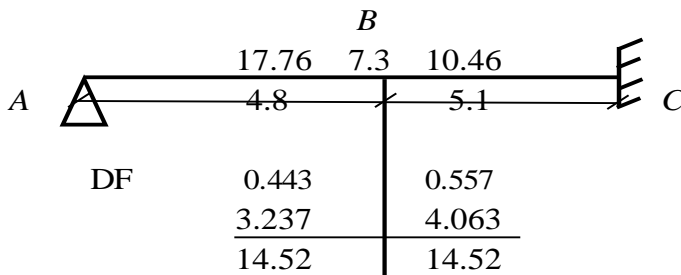
Panel P-5 and P-6 –on axis 3 adjustment is required.

$$\Delta M_s = 17.76 - 10.46 = 7.3$$

$$0.2M_{s,max} = 0.2 \times 17.76 = 3.552$$

$$\Delta M_s = 7.3 < 0.2M_{s,max} \quad \text{FALSE}$$

Moment distribution method required



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 6} = \underline{0.156 I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = \underline{0.20 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.443$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.557$$

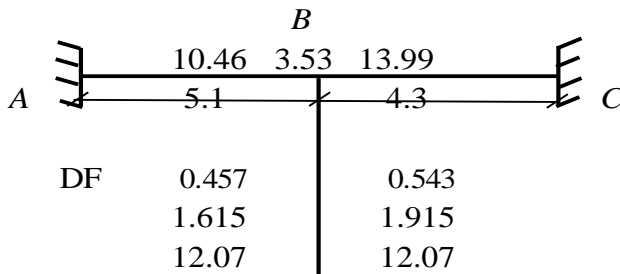
Panel P-6 and P-7 –on axis 4 adjustment is required.

$$\Delta M_s = 13.99 - 10.46 = 3.53$$

$$0.2M_{s,max} = 0.2 \times 13.99 = 2.798$$

$$\Delta M_s = 3.53 < 0.2M_{s,max} \quad \text{FALSE}$$

Moment distribution method required



$$K_{AB} = \frac{I}{L} = \frac{I}{6} = 0.196 I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = 0.23 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.457$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.543$$

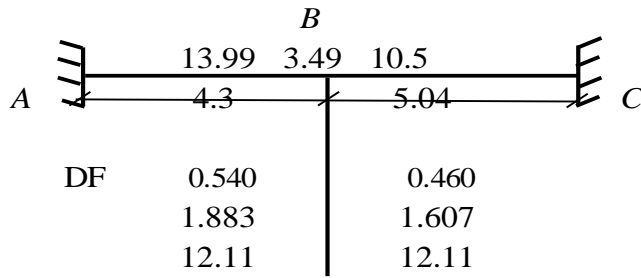
Panel P-7 and P-8 –on axis 5 adjustment is required.

$$\Delta M_s = 13.99 - 10.5 = 3.49$$

$$0.2M_{s,max} = 0.2 \times 13.99 = 2.798$$

$$\Delta M_s = 3.49 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{6} = 0.233 I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = 0.20 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.540$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.460$$

Panel P-8 and P-9 –on axis 6 adjustment is required.

$$\Delta M_s = 10.95 - 10.5 = 0.45$$

$$0.2M_{s,max} = 0.2 \times 10.95 = 2.19$$

$$\Delta M_s = 0.45 < 0.2M_{s,max} \quad \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (10.95 + 10.5) = 10.73$$

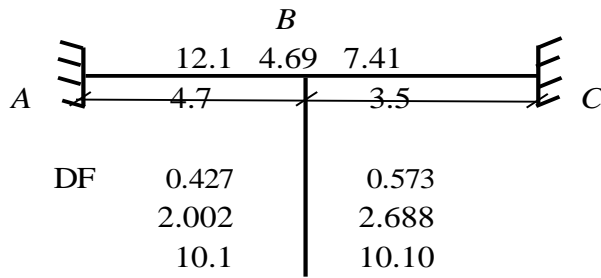
Panel P-6 and P-10 –on axis C adjustment is required.

$$\Delta M_s = 12.1 - 7.41 = 4.69$$

$$0.2M_{s,max} = 0.2 \times 12.1 = 2.42$$

$$\Delta M_s = 4.69 > 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{6} = 0.213 I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = 0.29 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.427$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.573$$

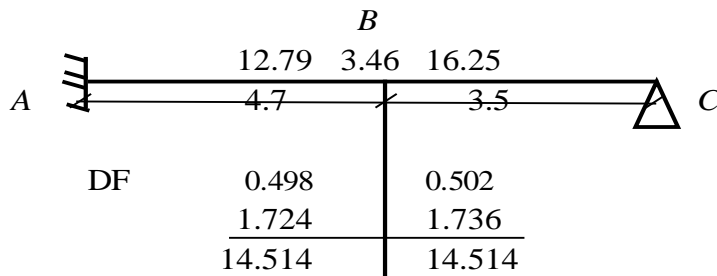
Panel P-7 and P-11 –on axis C adjustment is required.

$$\Delta M_s = 16.25 - 12.79 = 3.46$$

$$0.2M_{s,max} = 0.2 \times 16.25 = 3.25$$

$$\Delta M_s = 3.46 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.213 I$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = 0.21 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.498$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.502$$

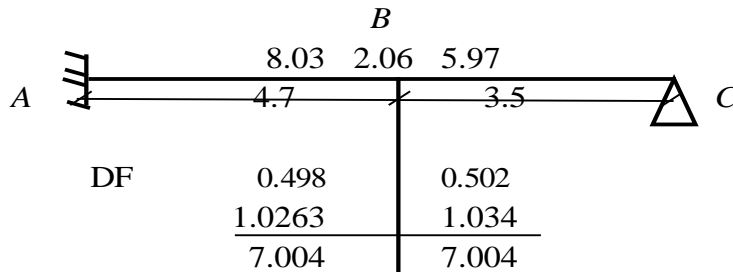
Panel P-8 and P-12 –on axis C adjustment is required.

$$\Delta M_s = 8.03 - 5.97 = 2.06$$

$$0.2M_{s,max} = 0.2 \times 8.03 = 1.606$$

$$\Delta M_s = 2.06 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = \underline{0.213 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.21 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.498$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.502$$

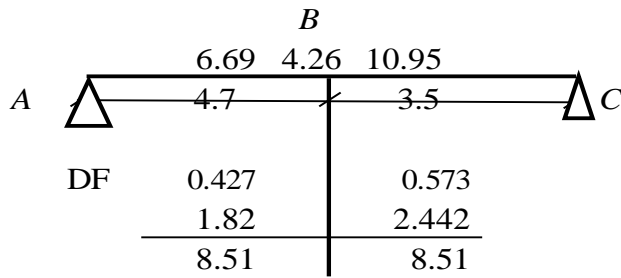
Panel P-9 and P-13 –on axis C adjustment is required.

$$\Delta M_s = 10.95 - 6.69 = 4.26$$

$$0.2M_{s,max} = 0.2 \times 10.95 = 2.19$$

$$\Delta M_s = 4.26 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.16 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.21 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.427$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.573$$

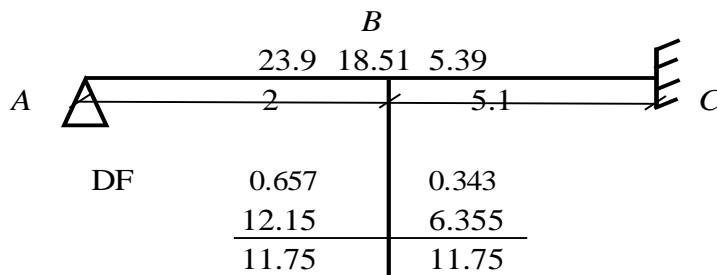
Panel C-1 and P-10 –on axis 3 adjustment is required.

$$\Delta M_s = 23.9 - 5.39 = 18.51$$

$$0.2M_{s,max} = 0.2 \times 23.9 = 4.78$$

$$\Delta M_s = 18.51 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 6} = \underline{0.375 I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = \underline{0.20 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.657$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.343$$

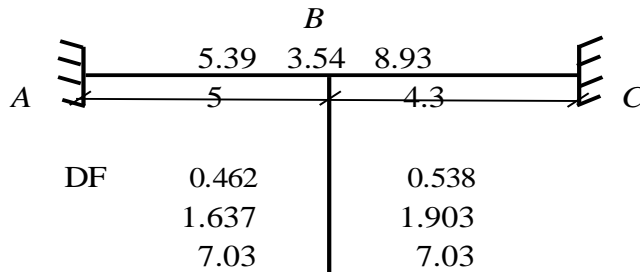
Panel P-10 and P-11 –on axis 4 adjustment is required.

$$\Delta M_s = 8.93 - 5.39 = 3.54$$

$$0.2M_{s,max} = 0.2 \times 8.93 = 1.786$$

$$\Delta M_s = 3.54 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{6} = 0.2 I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = 0.23 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.462$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.538$$

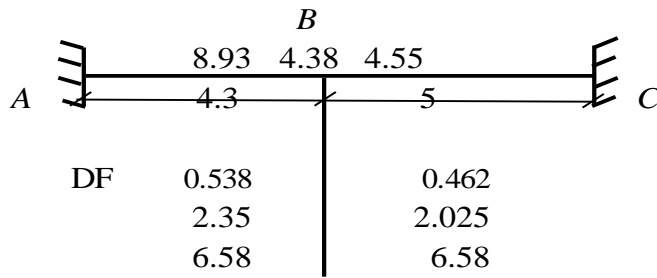
Panel P-11 and P-12 –on axis 5 adjustment is required.

$$\Delta M_s = 8.93 - 4.55 = 4.38$$

$$0.2M_{s,max} = 0.2 \times 8.93 = 1.786$$

$$\Delta M_s = 4.38 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{6} = 0.233 I$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = 0.20 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.538$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.462$$

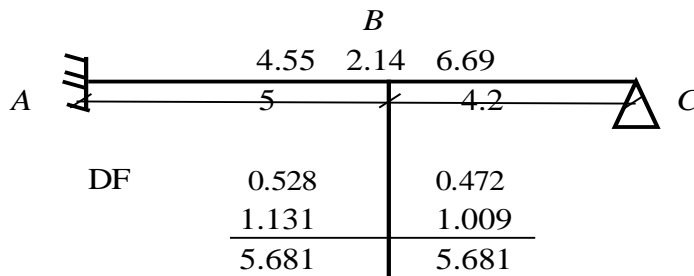
Panel P-12 and P-13 –on axis 6 adjustment is required.

$$\Delta M_s = 6.69 - 4.55 = 2.14$$

$$0.2M_{s,max} = 0.2 \times 6.69 = 1.338$$

$$\Delta M_s = 2.14 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used

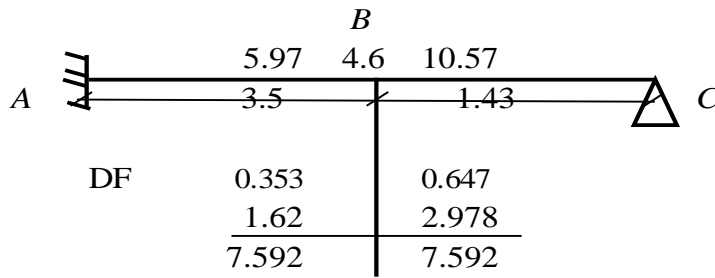


$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.2 I$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = 0.18 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.528$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.472$$



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = 0.286 I$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4*5} = 0.52 I$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.353$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.647$$

C-3 field moment adjustment for typical 1-4 floor

P-1

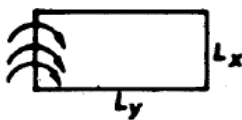
$$M_{xf} = 13.02 \quad M_{xs} = 17.6 \quad M_{xs,adj} = 17.35$$

$$M_{yf} = 8.68 \quad M_{ys} = 11.33 \quad M_{ys,adj} = 11.33$$

$$L_y/L_x = 1.37$$

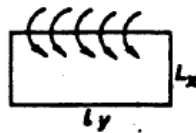
$$\Delta M_{xs} = M_{xs} - M_{xs,adj} = 0.25$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 0.00 \text{ neglect}$$



$$C_x = 0.318$$

$$C_y = 0.118$$



$$C_x = 0.391$$

$$C_y = 0.337$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_x \Delta M_{ys} = 0.098$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{13.118}$$

$$\Delta M_{yf} = C_y \Delta M_{xs} + C_y \Delta M_{ys} = 0.084$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{8.764}$$

P-2

$$\begin{aligned}
 M_{xf} &= 12.91 & M_{Xs} &= 17.09 & M_{Xs,adj} &= 17.09 \\
 M_{yf} &= 11.01 & M_{ys} &= 14.81 & M_{ys,adj} &= 17.21 \\
 L_y/L_x &= 1.13
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect}$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -2.40 \quad \text{neglect}$$

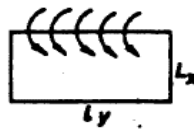
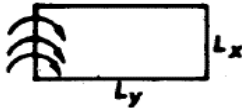
No adjustment required

P-3

$$\begin{aligned}
 M_{xf} &= 12.91 & M_{Xs} &= 17.09 & M_{Xs,adj} &= 17.65 \\
 M_{yf} &= 11.01 & M_{ys} &= 14.81 & M_{ys,adj} &= 13.46 \\
 L_y/L_x &= 1.13
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = -0.56 \quad \text{neglect}$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 1.35$$



$$C_x = 0.351$$

$$C_x = 0.323$$

$$C_y = 0.206$$

$$C_y = 0.371$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_x \Delta M_{ys} = 0.474$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{13.384}$$

$$\Delta M_{yf} = C_y \Delta M_{xs} + C_y \Delta M_{ys} = 0.278$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{11.288}$$

P-4

$$\begin{aligned}
 M_{xf} &= 13.4 & M_{Xs} &= 18.21 & M_{Xs,adj} &= 18.21 \\
 M_{yf} &= 9.1 & M_{ys} &= 11.89 & M_{ys,adj} &= 12.34 \\
 L_y/L_x &= 1.34
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect}$$

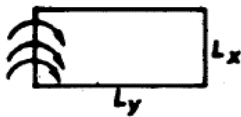
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -0.45 \quad \text{neglect}$$

No field moment adjustment required

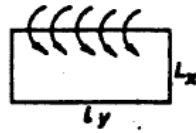
P-5

$$\begin{aligned}
 M_{xf} &= 15.87 & M_{Xs} &= 21.17 & M_{Xs,adj} &= 21.17 \\
 M_{yf} &= 13.61 & M_{ys} &= 17.76 & M_{ys,adj} &= 14.52 \\
 L_y/L_x &= 1.07
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 3.24
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.371 \\
 C_y &= 0.24
 \end{aligned}$$



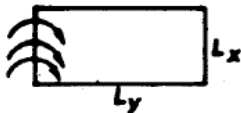
$$\begin{aligned}
 C_x &= 0.296 \\
 C_y &= 0.376
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 1.202 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{17.072} \\
 \Delta M_{yf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.778 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{14.388}
 \end{aligned}$$

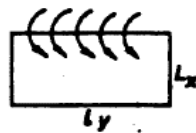
P-6

$$\begin{aligned}
 M_{xf} &= 9.15 & M_{Xs} &= 12.1 & M_{Xs,adj} &= 10.1 \\
 M_{yf} &= 7.85 & M_{ys} &= 10.46 & M_{ys,adj} &= 12.07 \\
 L_y/L_x &= 1.11
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 2.00 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -1.61 \quad \text{neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.359 \\
 C_y &= 0.213
 \end{aligned}$$



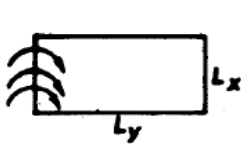
$$\begin{aligned}
 C_x &= 0.319 \\
 C_y &= 0.373
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.638 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{9.788} \\
 \Delta M_{yf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.746 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{8.596}
 \end{aligned}$$

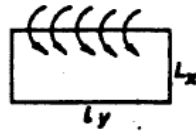
P-7

$$\begin{aligned}
 M_{xf} &= 10.8 & M_{Xs} &= 13.99 & M_{Xs,adj} &= 12.11 \\
 M_{yf} &= 9.6 & M_{ys} &= 12.79 & M_{ys,adj} &= 14.51 \\
 L_y/L_x &= 1.07
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 1.88 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -1.72 \text{ neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.371 \\
 C_y &= 0.24
 \end{aligned}$$



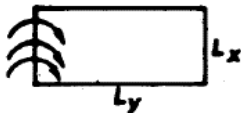
$$\begin{aligned}
 C_x &= 0.296 \\
 C_y &= 0.376
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.556 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{11.356} \\
 \Delta M_{yf} &= C_y \Delta M_{xs} + C_x \Delta M_{ys} = 0.707 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{10.307}
 \end{aligned}$$

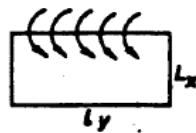
P-8

$$\begin{aligned}
 M_{xf} &= 7.82 & M_{Xs} &= 8.03 & M_{Xs,adj} &= 7 \\
 M_{yf} &= 6.17 & M_{ys} &= 10.5 & M_{ys,adj} &= 10.73 \\
 L_y/L_x &= 1.13
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 1.03 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -0.23 \text{ neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.351 \\
 C_y &= 0.206
 \end{aligned}$$



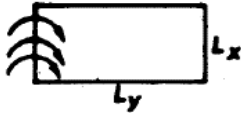
$$\begin{aligned}
 C_x &= 0.323 \\
 C_y &= 0.371
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.333 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{8.153} \\
 \Delta M_{yf} &= C_y \Delta M_{xs} + C_x \Delta M_{ys} = 0.382 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{6.552}
 \end{aligned}$$

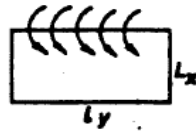
P-9

$$\begin{aligned}
 M_{xf} &= 8.11 & M_{Xs} &= 10.95 & M_{Xs,adj} &= 8.51 \\
 M_{yf} &= 5.12 & M_{ys} &= 6.69 & M_{ys,adj} &= 10.95 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 2.44 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -4.26 \text{ neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.311 \\
 C_y &= 0.102
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.41 \\
 C_y &= 0.321
 \end{aligned}$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 1.000$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{9.110}$$

$$\Delta M_{yf} = C_x \Delta M_{ys} + C_y \Delta M_{xs} = 0.783$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{5.903}$$

P-10

$$\begin{aligned}
 M_{xf} &= 5.56 & M_{Xs} &= 7.41 & M_{Xs,adj} &= 8.52 \\
 M_{yf} &= 4.04 & M_{ys} &= 5.39 & M_{ys,adj} &= 7.03 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = -1.11 \text{ neglect}$$

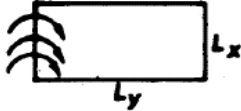
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -1.64 \text{ neglect}$$

No adjustment of field moment required,

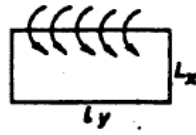
P-11

$$\begin{aligned}
 M_{xf} &= 12.36 & M_{Xs} &= 16.25 & M_{Xs,adj} &= 16.25 \\
 M_{yf} &= 6.87 & M_{ys} &= 8.93 & M_{ys,adj} &= 6.58 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 2.35
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.311 \\
 C_y &= 0.102
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.41 \\
 C_y &= 0.321
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.731 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{13.091} \\
 \Delta M_{yf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.240 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{7.110}
 \end{aligned}$$

P-12

$$\begin{aligned}
 M_{xf} &= 4.55 & M_{Xs} &= 5.97 & M_{Xs,adj} &= 7.59 \\
 M_{yf} &= 3.41 & M_{ys} &= 4.55 & M_{ys,adj} &= 5.68 \\
 L_y/L_x &= 1.2
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = -1.62 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -1.13 \quad \text{neglect}
 \end{aligned}$$

No adjustment of field moment required,

P-13

$$\begin{aligned}
 M_{xf} &= 8.25 & M_{Xs} &= 10.95 & M_{Xs,adj} &= 10.95 \\
 M_{yf} &= 5.12 & M_{ys} &= 6.69 & M_{ys,adj} &= 6.69 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 0.00 \quad \text{neglect}
 \end{aligned}$$

No adjustment of field moment required,

C-4 shear force calculation for typical 1st-4th floor

Structural design of G+4 Apartment + shop building B.Sc Thesis project

Panel	support condition	ly/lx	lx	pd	shear force coefficient		shear location	Shear force $V_i = \beta_{vi} P_d L_x$
					Location	value		
P-1	Type 4	1.37	4.15	7.383		$\beta_{vxc} = 0.51$	$V_{xc} = 15.626$	
						$\beta_{vxd} = 0.34$	$V_{xd} = 10.417$	
						$\beta_{vyc} = 0.4$	$V_{yc} = 12.256$	
						$\beta_{vyd} = 0.26$	$V_{yd} = 7.966$	
P-2	Type 2	1.13	5.04	12.588		$\beta_{vxc} = 0.4$	$V_{xc} = 25.377$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.36$	$V_{yc} = 22.840$	
						$\beta_{vyd} = 0.24$	$V_{yd} = 15.226$	
P-3	Type 2	1.13	5.04	12.588		$\beta_{vxc} = 0.4$	$V_{xc} = 25.377$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.36$	$V_{yc} = 22.840$	
						$\beta_{vyd} = 0.24$	$V_{yd} = 15.226$	
P-4	Type 4	1.34	4.25	7.383		$\beta_{vxc} = 0.51$	$V_{xc} = 16.003$	
						$\beta_{vxd} = 0.34$	$V_{xd} = 10.668$	
						$\beta_{vyc} = 0.4$	$V_{yc} = 12.551$	
						$\beta_{vyd} = 0.26$	$V_{yd} = 8.158$	
P-5	Type 4	1.07	4.70	10.497		$\beta_{vxc} = 0.43$	$V_{xc} = 21.214$	
						$\beta_{vxd} = 0.28$	$V_{xd} = 13.814$	
						$\beta_{vyc} = 0.4$	$V_{yc} = 19.734$	
						$\beta_{vyd} = 0.26$	$V_{yd} = 12.827$	
P-6	Type 1	1.11	4.25	11.492		$\beta_{vxc} = 0.36$	$V_{xc} = 17.583$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.33$	$V_{yc} = 16.118$	
						$\beta_{vyd} = 0$	$V_{yd} = 0.000$	
P-7	Type 1	1.07	4.70	11.502		$\beta_{vxc} = 0.35$	$V_{xc} = 18.921$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.33$	$V_{yc} = 17.840$	
						$\beta_{vyd} = 0$	$V_{yd} = 0$	
P-8	Type 3	1.13	4.15	9.588		$\beta_{vxc} = 0.41$	$V_{xc} = 16.314$	
						$\beta_{vxd} = 0.27$	$V_{xd} = 10.743$	
						$\beta_{vyc} = 0.36$	$V_{yc} = 14.324$	
						$\beta_{vyd} = 0$	$V_{yd} = 0.000$	
P-9	Type 4	1.46	3.45	9.588		$\beta_{vxc} = 0.53$	$V_{xc} = 17.532$	
						$\beta_{vxd} = 0.35$	$V_{xd} = 11.578$	
						$\beta_{vyc} = 0.4$	$V_{yc} = 13.231$	
						$\beta_{vyd} = 0.26$	$V_{yd} = 8.600$	
P-10	Type 1	1.23	3.45	7.539		$\beta_{vxc} = 0.39$	$V_{xc} = 10.144$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.33$	$V_{yc} = 8.583$	
						$\beta_{vyd} = 0$	$V_{yd} = 0$	
P-11	Type 3	1.46	3.45	12.614		$\beta_{vxc} = 0.5$	$V_{xc} = 21.759$	
						$\beta_{vxd} = 0.33$	$V_{xd} = 14.361$	
						$\beta_{vyc} = 0.36$	$V_{yc} = 15.667$	
						$\beta_{vyd} = 0$	$V_{yd} = 0$	
P-12	Type 1	1.20	3.45	9.588		$\beta_{vxc} = 0.39$	$V_{xc} = 12.901$	
						$\beta_{vxd} = 0$	$V_{xd} = 0$	
						$\beta_{vyc} = 0.33$	$V_{yc} = 10.916$	
						$\beta_{vyd} = 0$	$V_{yd} = 0$	
P-13	Type 4	1.46	3.45	9.588		$\beta_{vxc} = 0.53$	$V_{xc} = 17.532$	
						$\beta_{vxd} = 0.35$	$V_{xd} = 11.578$	
						$\beta_{vyc} = 0.4$	$V_{yc} = 13.231$	
						$\beta_{vyd} = 0.26$	$V_{yd} = 8.600$	

C-4 Reinforcement calculation for typical 1st-4th floor

panel	moment	Ast=Msd/Z*f _{yd}	ø	spacing	spacing used	Reinforcement
P-1	M _{xs} = 17.6	310.62	12	363.92	360	ø 12 c/c 360
	M _{xf} = 13.12	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 11.33	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 8.76	259.00	12	436.45	430	ø 12 c/c 430
P-2	M _{xs} = 17.35	306.20	12	369.17	360	ø 12 c/c 360
	M _{xf} = 12.91	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 14.81	261.38	12	432.48	430	ø 12 c/c 430
	M _{yf} = 11.01	259.00	12	436.45	430	ø 12 c/c 430
P-3	M _{xs} = 17.09	301.62	12	374.78	370	ø 12 c/c 370
	M _{xf} = 13.38	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 14.81	261.38	12	432.48	430	ø 12 c/c 430
	M _{yf} = 11.29	259.00	12	436.45	430	ø 12 c/c 430
P-4	M _{xs} = 17.65	311.50	12	362.89	360	ø 12 c/c 360
	M _{xf} = 13.4	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 11.89	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 9.1	259.00	12	436.45	430	ø 12 c/c 430
P-5	M _{xs} = 17.21	303.73	12	372.17	360	ø 12 c/c 360
	M _{xf} = 17.07	301.26	12	375.22	360	ø 12 c/c 360
	M _{ys} = 17.76	313.44	12	360.64	360	ø 12 c/c 360
	M _{yf} = 14.39	259.00	12	436.45	430	ø 12 c/c 430
P-6	M _{xs} = 13.46	259.00	12	436.45	430	ø 12 c/c 430
	M _{xf} = 9.79	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 14.52	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 8.6	259.00	12	436.45	430	ø 12 c/c 430
P-7	M _{xs} = 12.34	259.00	12	436.45	430	ø 12 c/c 430
	M _{xf} = 11.36	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 12.07	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 10.31	259.00	12	436.45	430	ø 12 c/c 430
P-8	M _{xs} = 8.03	259.00	12	436.45	430	ø 12 c/c 430
	M _{xf} = 8.15	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 12.11	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 6.55	259.00	12	436.45	430	ø 12 c/c 430
P-9	M _{xs} = 10.73	259.00	12	436.45	430	ø 12 c/c 430
	M _{xf} = 9.11	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 6.69	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 5.9	259.00	12	436.45	430	ø 12 c/c 430
P-10	M _{xs} = 10.1	259.00	12	436.45	430	ø 12 c/c 430
	M _{xf} = 5.56	259.00	12	436.45	430	ø 12 c/c 430
	M _{ys} = 11.75	259.00	12	436.45	430	ø 12 c/c 430
	M _{yf} = 4.04	259.00	12	436.45	430	ø 12 c/c 430

panel	moment		Ast=Msd/Z*f _{yd}	∅	spacing	spacing used	Reinforcement		
P-11	M _{xs} =	14.51	259.00	12	436.45	430	∅ 12	c/c	430
	M _{xf} =	13.09	259.00	12	436.45	430	∅ 12	c/c	430
	M _{ys} =	7.03	259.00	12	436.45	430	∅ 12	c/c	430
	M _{yf} =	7.11	259.00	12	436.45	430	∅ 12	c/c	430
P-12	M _{xs} =	7	259.00	12	436.45	430	∅ 12	c/c	430
	M _{xf} =	4.55	259.00	12	436.45	430	∅ 12	c/c	430
	M _{ys} =	6.58	259.00	12	436.45	430	∅ 12	c/c	430
	M _{yf} =	3.41	259.00	12	436.45	430	∅ 12	c/c	430
P-13	M _{xs} =	8.51	259.00	12	436.45	430	∅ 12	c/c	430
	M _{xf} =	8.25	259.00	12	436.45	430	∅ 12	c/c	430
	M _{ys} =	5.68	259.00	12	436.45	430	∅ 12	c/c	430
	M _{yf} =	5.12	259.00	12	436.45	430	∅ 12	c/c	430

For support moment reinforcement calculation in the above table all left and top side moments are used, the remaining right and bottom side supports are calculated in the table below

panel	moment		Ast=Msd/Z*f _{yd}	∅	spacing	spacing used	Reinforcement		
right side edge supports	P4=	18.21	321.38	12	351.73	350	∅ 12	c/c	350
	P9=	10.95	259.00	12	436.45	430	∅ 12	c/c	430
	P13=	6.69	259.00	12	436.45	430	∅ 12	c/c	430
Bottom edge supports	P5=	21.17	373.62	12	302.55	300	∅ 12	c/c	300
	P11=	16.25	286.79	12	394.15	390	∅ 12	c/c	390
	P13=	10.95	259.00	12	436.45	430	∅ 12	c/c	430

C-4 individual moment analysis result for roof slab

panel	support conditio	ly/lx	lx(m)	lx ²	pd	moment coefficient		moment location	Bending moment
						location	value		
P-1	Type 4	1.37	4.15	17.223	9.742	α_{xs}	0.073	$M_{xs} =$	12.25
						α_{xf}	0.054	$M_{xf} =$	9.06
						α_{ys}	0.047	$M_{ys} =$	7.89
						α_{yf}	0.036	$M_{yf} =$	6.04
P-2	Type 2	1.13	5.04	25.402	9.742	α_{xs}	0.045	$M_{xs} =$	11.14
						α_{xf}	0.034	$M_{xf} =$	8.41
						α_{ys}	0.039	$M_{ys} =$	9.65
						α_{yf}	0.029	$M_{yf} =$	7.18
P-3	Type 2	1.13	5.04	25.402	9.742	α_{xs}	0.045	$M_{xs} =$	11.14
						α_{xf}	0.034	$M_{xf} =$	8.41
						α_{ys}	0.039	$M_{ys} =$	9.65
						α_{yf}	0.029	$M_{yf} =$	7.18
P-4	Type 4	1.34	4.25	18.063	9.742	α_{xs}	0.072	$M_{xs} =$	12.67
						α_{xf}	0.053	$M_{xf} =$	9.33
						α_{ys}	0.047	$M_{ys} =$	8.27
						α_{yf}	0.036	$M_{yf} =$	6.33
P-5	Type 4	1.20	3.15	9.923	9.742	α_{xs}	0.072	$M_{xs} =$	6.96
						α_{xf}	0.053	$M_{xf} =$	5.12
						α_{ys}	0.047	$M_{ys} =$	4.54
						α_{yf}	0.036	$M_{yf} =$	3.48
P-6	Type 4	1.07	4.70	22.090	9.742	α_{xs}	0.056	$M_{xs} =$	12.05
						α_{xf}	0.042	$M_{xf} =$	9.04
						α_{ys}	0.047	$M_{ys} =$	10.11
						α_{yf}	0.036	$M_{yf} =$	7.75
P-7	Type 1	1.11	4.25	18.063	9.742	α_{xs}	0.037	$M_{xs} =$	6.51
						α_{xf}	0.028	$M_{xf} =$	4.93
						α_{ys}	0.032	$M_{ys} =$	5.63
						α_{yf}	0.024	$M_{yf} =$	4.22
P-8	Type 1	1.07	4.70	22.090	9.742	α_{xs}	0.035	$M_{xs} =$	7.53
						α_{xf}	0.027	$M_{xf} =$	5.81
						α_{ys}	0.032	$M_{ys} =$	6.89
						α_{yf}	0.024	$M_{yf} =$	5.16
P-9	Type 3	1.13	4.15	17.223	9.742	α_{xs}	0.051	$M_{xs} =$	8.56
						α_{xf}	0.038	$M_{xf} =$	6.38
						α_{ys}	0.039	$M_{ys} =$	6.54
						α_{yf}	0.03	$M_{yf} =$	5.03

panel	support conditio	ly/lx	lx(m)	lx ²	pd	moment coefficient		moment location	Bending moment
						location	value		
P-10	Type 4	1.46	3.45	11.903	9.742	α_{xs}	0.077	$M_{xs} =$	8.93
						α_{xf}	0.057	$M_{xf} =$	6.61
						α_{ys}	0.047	$M_{ys} =$	5.45
						α_{yf}	0.036	$M_{yf} =$	4.17
P-11	Type 1	1.23	3.45	11.903	9.742	α_{xs}	0.044	$M_{xs} =$	5.10
						α_{xf}	0.033	$M_{xf} =$	3.83
						α_{ys}	0.032	$M_{ys} =$	3.71
						α_{yf}	0.024	$M_{yf} =$	2.78
P-12	Type 3	1.46	3.45	11.903	9.742	α_{xs}	0.071	$M_{xs} =$	8.23
						α_{xf}	0.054	$M_{xf} =$	6.26
						α_{ys}	0.039	$M_{ys} =$	4.52
						α_{yf}	0.03	$M_{yf} =$	3.48
P-13	Type 1	1.20	3.45	11.903	9.742	α_{xs}	0.042	$M_{xs} =$	4.87
						α_{xf}	0.032	$M_{xf} =$	3.71
						α_{ys}	0.032	$M_{ys} =$	3.71
						α_{yf}	0.024	$M_{yf} =$	2.78
P-14	Type 4	1.46	3.45	11.903	9.742	α_{xs}	0.077	$M_{xs} =$	8.93
						α_{xf}	0.058	$M_{xf} =$	6.73
						α_{ys}	0.047	$M_{ys} =$	5.45
						α_{yf}	0.036	$M_{yf} =$	4.17

C-4 support moment adjustment for roof slab

Panel P-1 and P-2 –on axis 2 adjustment is required.

$$\Delta M_s = 12.25 - 11.14 = 1.11$$

$$0.2M_{s,max} = 0.2 \times 12.25 = 2.45$$

$$\Delta M_s = 1.11 < 0.2M_{s,max} \quad \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (12.25 + 11.14) = 11.70$$

Panel P-3 and P-4 –on axis 4 adjustment is required.

$$\Delta M_s = 12.67 - 11.14 = 1.53$$

$$0.2M_{s,max} = 0.2 \times 12.67 = 2.534$$

$$\Delta M_s = 1.53 < 0.2M_{s,max} \quad \text{TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (12.67 + 11.14) = 11.91$$

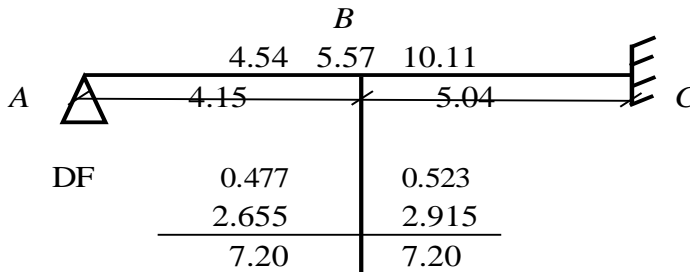
Panel P-5 and P-6 –on axis 2 adjustment is required.

$$\Delta M_s = 10.1 - 4.54 = 5.57$$

$$0.2M_{s,max} = 0.2 \times 10.11 = 2.022$$

$$\Delta M_s = 5.57 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 6} = \underline{0.181 I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = \underline{0.20 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.477$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.523$$

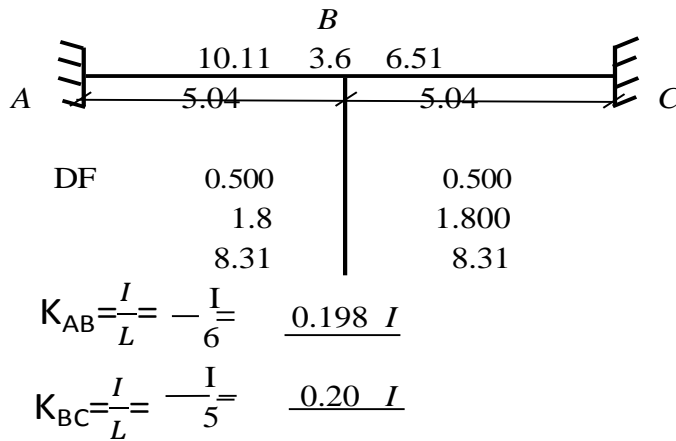
Panel P-6 and P-7 –on axis 3 adjustment is required.

$$\Delta M_s = 10.1 - 6.51 = 3.6$$

$$0.2M_{s,max} = 0.2 \times 10.11 = 2.022$$

$$\Delta M_s = 3.6 < 0.2M_{s,max} \quad \text{FALSE}$$

Therefore, Moment distribution method used



$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.500$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.500$$

Panel P-7 and P-8 –on axis 4 adjustment is required.

$$\Delta M_s = 7.53 - 6.51 = 1.02$$

$$0.2M_{s,max} = 0.2 \times 7.53 = 1.506$$

$$\Delta M_s = 1.02 < 0.2M_{s,max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (7.53 + 6.51) = 7.02$$

Panel P-8 and P-9 –on axis 5 adjustment is required.

$$\Delta M_s = 7.53 - 6.54 = 0.99$$

$$0.2M_{s,max} = 0.2 \times 7.53 = 1.506$$

$$\Delta M_s = 0.99 < 0.2M_{s,max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (7.53 + 6.54) = 7.04$$

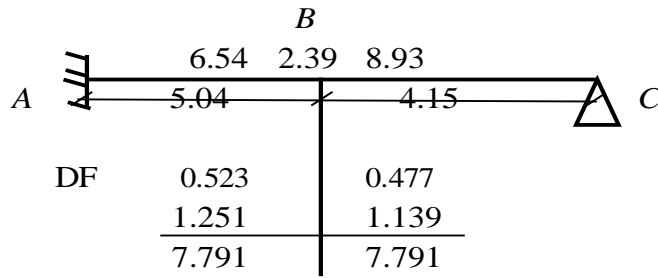
Panel P- 9and P-10 –on axis 6 adjustment is required.

$$\Delta M_s = 8.93 - 6.54 = 2.39$$

$$0.2M_{s,max} = 0.2 \times 8.93 = 1.786$$

$$\Delta M_s = 2.39 < 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = \underline{0.198 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4*5} = \underline{0.18 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.523$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.477$$

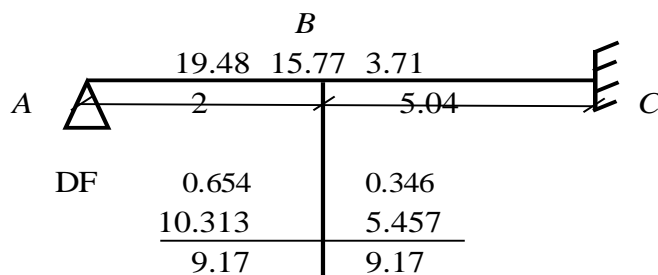
Panel C1- 9 and P-11 –on axis 3 adjustment is required.

$$\Delta M_s = 19.48 - 3.71 = 15.77$$

$$0.2M_{s,max} = 0.2 \times 19.48 = 3.896$$

$$\Delta M_s = 15.77 < 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4*6} = \underline{0.375 I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = \underline{0.20 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.654$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.346$$

Panel P- 11and P-12 –on axis 4 adjustment is required.

$$\Delta M_s = 4.52 - 3.71 = 0.81$$

$$0.2M_{s,max} = 0.2 \times 4.52 = 0.904$$

$$\Delta M_s = 0.81 < 0.2M_{s,max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (4.52 + 3.71) = 4.12$$

Panel P- 12and P-13 –on axis 5 adjustment is required.

$$\Delta M_s = 4.52 - 3.71 = 0.81$$

$$0.2M_{s,max} = 0.2 \times 4.52 = 0.904$$

$$\Delta M_s = 0.81 < 0.2M_{s,max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,avg} = \frac{1}{2} (4.52 + 3.71) = 4.12$$

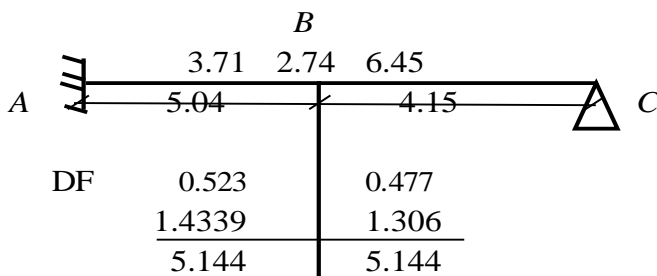
Panel P- 13 and P-14 –on axis 5 adjustment is required.

$$\Delta M_s = 5.45 - 3.71 = 1.74$$

$$0.2M_{s,max} = 0.2 \times 5.45 = 1.09$$

$$\Delta M_s = 1.74 > 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = \underline{0.198 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.18 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.523$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.477$$

Panel P- 4 and P-8 –on axis B adjustment is required.

$$\Delta M_s = 8.27 - 6.89 = 1.38$$

$$0.2M_{s,\max} = 0.2 \times 8.27 = 1.654$$

$$\Delta M_s = 1.38 < 0.2M_{s,\max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,\text{avg}} = \frac{1}{2} (8.27 + 6.89) = 7.58$$

Panel P- 7 and P-11 –on axis C adjustment is required.

$$\Delta M_s = 5.63 - 5.1 = 0.53$$

$$0.2M_{s,\max} = 0.2 \times 5.63 = 1.126$$

$$\Delta M_s = 0.53 < 0.2M_{s,\max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,\text{avg}} = \frac{1}{2} (5.63 + 5.1) = 5.37$$

Panel P- 8 and P-12 –on axis C adjustment is required.

$$\Delta M_s = 6.89 - 8.23 = -1.34$$

$$0.2M_{s,\max} = 0.2 \times 6.89 = 1.378$$

$$\Delta M_s = -1.34 < 0.2M_{s,\max} \text{ TRUE}$$

Therefore, average of the moments can be taken.

$$M_{s,\text{avg}} = \frac{1}{2} (6.89 + 8.23) = 7.56$$

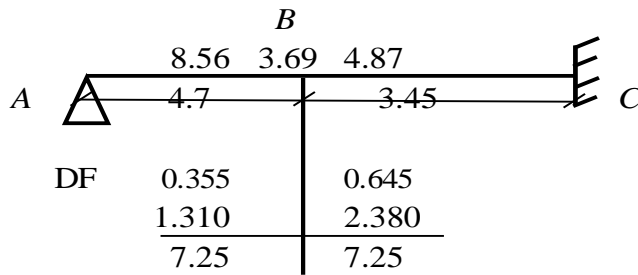
Panel P- 9 and P-13 –on axis C adjustment is required.

$$\Delta M_s = 8.56 - 4.87 = 3.69$$

$$0.2M_{s,\max} = 0.2 \times 8.56 = 1.712$$

$$\Delta M_s = 3.69 > 0.2M_{s,\max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 6} = \underline{0.16 I}$$

$$K_{BC} = \frac{I}{L} = \frac{I}{5} = \underline{0.29 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.355$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.645$$

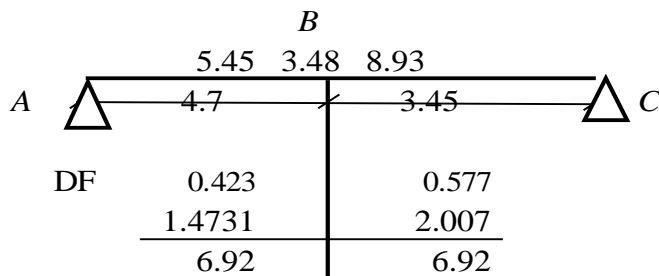
Panel P- 10 and P-14 –on axis C adjustment is required.

$$\Delta M_s = 8.93 - 5.45 = 3.48$$

$$0.2M_{s,max} = 0.2 \times 8.93 = 1.786$$

$$\Delta M_s = 3.48 < 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.16 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.22 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.423$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.577$$

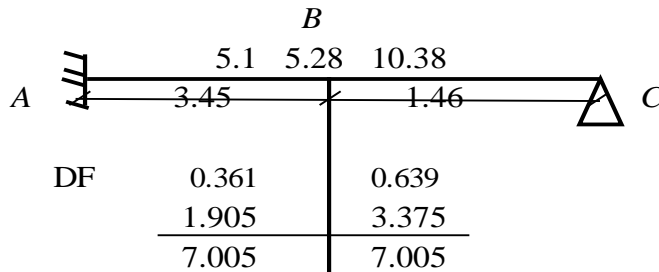
Panel P- 11 and C-2 –on axis D adjustment is required.

$$\Delta M_s = 10.38 - 5.1 = 5.28$$

$$0.2M_{s,max} = 0.2 \times 10.38 = 2.076$$

$$\Delta M_s = 5.28 < 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = \underline{0.29 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4 \times 5} = \underline{0.51 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.361$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.639$$

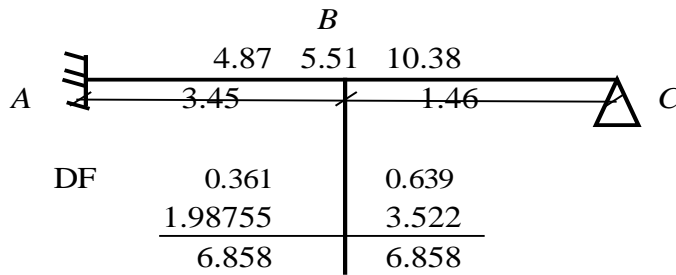
Panel P- 13 and C-3 –on axis D adjustment is required.

$$\Delta M_s = 10.38 - 4.87 = 5.51$$

$$0.2M_{s,max} = 0.2 \times 10.38 = 2.076$$

$$\Delta M_s = 5.51 < 0.2M_{s,max} \text{ FALSE}$$

Therefore, Moment distribution method used



$$K_{AB} = \frac{I}{L} = \frac{I}{5} = \underline{0.29 I}$$

$$K_{BC} = \frac{3I}{4L} = \frac{3I}{4*5} = \underline{0.51 I}$$

$$DF_{AB} = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.361$$

$$DF_{BC} = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.639$$

C-5 field moment adjustment for roof slab

P-2

$$M_{xf} = 8.41 \quad M_{Xs} = 11.4 \quad M_{Xs,adj} = 11.7$$

$$M_{yf} = 7.18 \quad M_{ys} = 9.65 \quad M_{ys,adj} = 9.65$$

$$Ly/Lx = 1.46$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = -0.30 \quad \text{neglect}$$

$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 0.00 \quad \text{neglect}$$

No adjustment of field moment required,

P-3

$$M_{xf} = 8.41 \quad M_{Xs} = 11.14 \quad M_{Xs,adj} = 11.14$$

$$M_{yf} = 7.18 \quad M_{ys} = 9.65 \quad M_{ys,adj} = 9.65$$

$$Ly/Lx = 1.46$$

$$\Delta M_{xs} = M_{xs} - M_{Xs,adj} = 0.00 \quad \text{neglect}$$

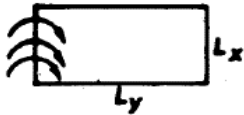
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = 0.00 \quad \text{neglect}$$

No adjustment of field moment required,

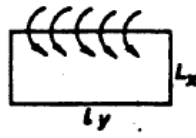
P-4

$$\begin{aligned}
 M_{xf} &= 9.33 & M_{xs} &= 12.67 & M_{xs,adj} &= 8.27 \\
 M_{yf} &= 6.33 & M_{ys} &= 8.27 & M_{ys,adj} &= 11.91 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 4.40 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -3.64 \quad \text{neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.318 \\
 C_y &= 0.126
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.383 \\
 C_y &= 0.341
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_x \Delta M_{ys} = 1.685 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{11.015} \\
 \Delta M_{yf} &= C_y \Delta M_{xs} + C_y \Delta M_{ys} = 1.500 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{7.830}
 \end{aligned}$$

P-5

$$\begin{aligned}
 M_{xf} &= 5.12 & M_{xs} &= 4.54 & M_{xs,adj} &= 4.54 \\
 M_{yf} &= 3.48 & M_{ys} &= 6.96 & M_{ys,adj} &= 7.43 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

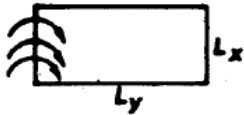
$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 0.00 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -0.47 \quad \text{neglect}
 \end{aligned}$$

No adjustment of field moment required,

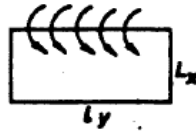
P-6

$$\begin{aligned}
 M_{xf} &= 9.04 & M_{xs} &= 10.11 & M_{xs,adj} &= 7.2 \\
 M_{yf} &= 7.75 & M_{ys} &= 12.05 & M_{ys,adj} &= 10.85 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 2.91 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 1.20
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.311 \\
 C_y &= 0.102
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.41 \\
 C_y &= 0.321
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 1.566 \\
 M_{xf,adj} &= \Delta M_{xf} + M_{xf} = \mathbf{10.606} \\
 \Delta M_{yf} &= C_x \Delta M_{xs} + C_y \Delta M_{ys} = 1.057 \\
 M_{yf,adj} &= \Delta M_{yf} + M_{yf} = \mathbf{8.807}
 \end{aligned}$$

P-7

$$\begin{aligned}
 M_{xf} &= 5.81 & M_{xs} &= 5.63 & M_{xs,adj} &= 8.31 \\
 M_{yf} &= 5.63 & M_{ys} &= 6.51 & M_{ys,adj} &= 8.12 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

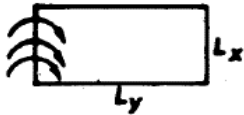
$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = -2.68 \quad \text{neglect} \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -1.61 \quad \text{neglect}
 \end{aligned}$$

No adjustment of field moment required,

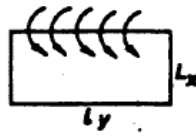
P-8

$$\begin{aligned}
 M_{xf} &= 5.81 & M_{xs} &= 7.53 & M_{xs,adj} &= 7.02 \\
 M_{yf} &= 5.16 & M_{ys} &= 6.89 & M_{ys,adj} &= 7.58 \\
 L_y/L_x &= 1.11
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 0.51 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = -0.69 \quad \text{neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.354 \\
 C_y &= 0.225
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.317 \\
 C_y &= 0.372
 \end{aligned}$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.162$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{5.972}$$

$$\Delta M_{yf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.190$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{5.350}$$

P-9

$$\begin{aligned}
 M_{xf} &= 5.03 & M_{xs} &= 8.56 & M_{xs,adj} &= 8.56 \\
 M_{yf} &= 6.38 & M_{ys} &= 6.54 & M_{ys,adj} &= 7.04 \\
 L_y/L_x &= 1.46
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{xs,adj} = 0.00 \quad \text{neglect}$$

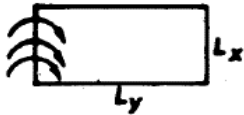
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -0.50 \quad \text{neglect}$$

No adjustment of field moment required,

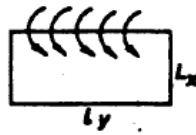
P-10

$$\begin{aligned}
 M_{xf} &= 4.17 & M_{xs} &= 8.93 & M_{xs,adj} &= 7.79 \\
 M_{yf} &= 6.61 & M_{ys} &= 5.45 & M_{ys,adj} &= 5.45 \\
 L_y/L_x &= 1.13
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 1.14 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 0.00 \quad \text{neglect}
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.35 \\
 C_y &= 0.204
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.324 \\
 C_y &= 0.371
 \end{aligned}$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_x \Delta M_{ys} = 0.369$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{4.539}$$

$$\Delta M_{yf} = C_y \Delta M_{xs} + C_y \Delta M_{ys} = 0.423$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{7.033}$$

P-11

$$\begin{aligned}
 M_{xf} &= 3.83 & M_{xs} &= 5.1 & M_{xs,adj} &= 5.37 \\
 M_{yf} &= 2.78 & M_{ys} &= 3.71 & M_{ys,adj} &= 9.17 \\
 L_y/L_x &= 1.13
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{xs,adj} = -0.27 \quad \text{neglect}$$

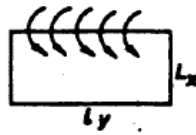
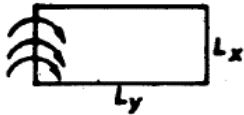
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -5.46 \quad \text{neglect}$$

No adjustment of field moment required,

P-12

$$\begin{aligned}
 M_{xf} &= 6.26 & M_{xs} &= 8.23 & M_{xs,adj} &= 7.56 \\
 M_{yf} &= 3.48 & M_{ys} &= 4.52 & M_{ys,adj} &= 4.12 \\
 L_y/L_x &= 1.23
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 0.67 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 0.40
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.331 \\
 C_y &= 0.162
 \end{aligned}$$

$$\begin{aligned}
 C_x &= 0.354 \\
 C_y &= 0.358
 \end{aligned}$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.370$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{6.630}$$

$$\Delta M_{yf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.305$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{3.785}$$

P-13

$$\begin{aligned}
 M_{xf} &= 3.71 & M_{xs} &= 4.87 & M_{xs,adj} &= 7.25 \\
 M_{yf} &= 2.78 & M_{ys} &= 3.71 & M_{ys,adj} &= 4.12 \\
 L_y/L_x &= 1.23
 \end{aligned}$$

$$\Delta M_{xs} = M_{xs} - M_{xs,adj} = -2.38 \quad \text{neglect}$$

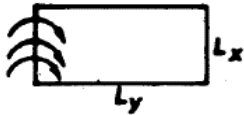
$$\Delta M_{ys} = M_{ys} - M_{ys,adj} = -0.41 \quad \text{neglect}$$

No adjustment of field moment required,

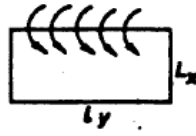
P-14

$$\begin{aligned}
 M_{xf} &= 6.73 & M_{xs} &= 8.93 & M_{xs,adj} &= 6.92 \\
 M_{yf} &= 4.17 & M_{ys} &= 5.45 & M_{ys,adj} &= 5.14 \\
 L_y/L_x &= 1.2
 \end{aligned}$$

$$\begin{aligned}
 \Delta M_{xs} &= M_{xs} - M_{xs,adj} = 2.01 \\
 \Delta M_{ys} &= M_{ys} - M_{ys,adj} = 0.31
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.338 \\
 C_y &= 0.172
 \end{aligned}$$



$$\begin{aligned}
 C_x &= 0.344 \\
 C_y &= 0.364
 \end{aligned}$$

$$\Delta M_{xf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.796$$

$$M_{xf,adj} = \Delta M_{xf} + M_{xf} = \mathbf{7.526}$$

$$\Delta M_{yf} = C_x \Delta M_{xs} + C_y \Delta M_{ys} = 0.785$$

$$M_{yf,adj} = \Delta M_{yf} + M_{yf} = \mathbf{4.955}$$

C-5 shear force calculation for roof slab

panel	support condition	ly/lx	lx	pd	shear force coefficient Location value	shear force location	Shear force $V_i = \beta_{vi} P_d L_x$
P-1	Type 4	1.37	4.15	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	20.619
					$\beta_{vxd} = 0.34$	$V_{xd} =$	13.746
					$\beta_{vyc} = 0.4$	$V_{yc} =$	16.172
					$\beta_{vyd} = 0.26$	$V_{yd} =$	10.512
P-2	Type 2	1.13	5.04	9.742	$\beta_{vxc} = 0.4$	$V_{xc} =$	19.640
					$\beta_{vxd} = 0$	$V_{xd} =$	0
					$\beta_{vyc} = 0.36$	$V_{yc} =$	17.676
					$\beta_{vyd} = 0.24$	$V_{yd} =$	11.784
P-3	Type 2	1.13	5.04	9.742	$\beta_{vxc} = 0.4$	$V_{xc} =$	19.640
					$\beta_{vxd} = 0$	$V_{xd} =$	0
					$\beta_{vyc} = 0.36$	$V_{yc} =$	17.676
					$\beta_{vyd} = 0.24$	$V_{yd} =$	11.784
P-4	Type 4	1.34	4.25	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	21.116
					$\beta_{vxd} = 0.34$	$V_{xd} =$	14.077
					$\beta_{vyc} = 0.4$	$V_{yc} =$	16.561
					$\beta_{vyd} = 0.26$	$V_{yd} =$	10.765
P-5	Type 4	1.20	3.15	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	15.651
					$\beta_{vxd} = 0.34$	$V_{xd} =$	10.434
					$\beta_{vyc} = 0.4$	$V_{yc} =$	12.275
					$\beta_{vyd} = 0.26$	$V_{yd} =$	7.979
P-6	Type 4	1.07	4.70	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	23.352
					$\beta_{vxd} = 0.34$	$V_{xd} =$	16
					$\beta_{vyc} = 0.4$	$V_{yc} =$	18.315
					$\beta_{vyd} = 0.26$	$V_{yd} =$	11.905
P-7	Type 1	1.11	4.25	9.742	$\beta_{vxc} = 0.36$	$V_{xc} =$	14.905
					$\beta_{vxd} = 0$	$V_{xd} =$	0
					$\beta_{vyc} = 0.33$	$V_{yc} =$	13.663
					$\beta_{vyd} = 0$	$V_{yd} =$	0
P-8	Type 1	1.07	4.70	9.742	$\beta_{vxc} = 0.36$	$V_{xc} =$	16.483
					$\beta_{vxd} = 0$	$V_{xd} =$	0.000
					$\beta_{vyc} = 0.33$	$V_{yc} =$	15.110
					$\beta_{vyd} = 0$	$V_{yd} =$	0.000

Structural design of G+4 Apartment + shop building B.Sc Thesis project

panel	support condition	ly/lx	lx	pd	shear force coefficient Location value	shear force location	Shear force $V_i = \beta_{vi} P_d L_x$
P-9	Type 3	1.13	4.15	9.742	$\beta_{vxc} = 0.5$	$V_{xc} =$	20.215
					$\beta_{vxd} = 0.33$	$V_{xd} =$	13.342
					$\beta_{vyc} = 0.36$	$V_{yc} =$	14.555
					$\beta_{vyd} = 0$	$V_{yd} =$	0.000
P-10	Type 4	1.46	3.45	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	17.141
					$\beta_{vxd} = 0.34$	$V_{xd} =$	11
					$\beta_{vyc} = 0.4$	$V_{yc} =$	13.444
					$\beta_{vyd} = 0.26$	$V_{yd} =$	9
P-11	Type 1	1.23	3.45	9.742	$\beta_{vxc} = 0.36$	$V_{xc} =$	12.100
					$\beta_{vxd} = 0$	$V_{xd} =$	0.000
					$\beta_{vyc} = 0.33$	$V_{yc} =$	11.091
					$\beta_{vyd} = 0$	$V_{yd} =$	0
P-12	Type 3	1.46	3.45	9.742	$\beta_{vxc} = 0.5$	$V_{xc} =$	16.805
					$\beta_{vxd} = 0.33$	$V_{xd} =$	11
					$\beta_{vyc} = 0.36$	$V_{yc} =$	12.100
					$\beta_{vyd} = 0$	$V_{yd} =$	0
P-13	Type 1	1.20	3.45	9.742	$\beta_{vxc} = 0.36$	$V_{xc} =$	12.100
					$\beta_{vxd} = 0$	$V_{xd} =$	0.000
					$\beta_{vyc} = 0.33$	$V_{yc} =$	11.091
					$\beta_{vyd} = 0$	$V_{yd} =$	0.000
P-14	Type 4	1.46	3.45	9.742	$\beta_{vxc} = 0.51$	$V_{xc} =$	17.141
					$\beta_{vxd} = 0.34$	$V_{xd} =$	11.427
					$\beta_{vyc} = 0.4$	$V_{yc} =$	13.444
					$\beta_{vyd} = 0.26$	$V_{yd} =$	8.739
C-1	Type 7	2.05	1.80	9.742	$\beta_{vxc} = 0.64$	$V_{xc} =$	11.223
					$\beta_{vxd} = 0.42$	$V_{xd} =$	7.365
					$\beta_{vyc} = 0$	$V_{yc} =$	0.000
					$\beta_{vyd} = 0.3$	$V_{yd} =$	5.261
C-2	Type 7	3.76	1.34	9.742	$\beta_{vxc} = 0.94$	$V_{xc} =$	12.271
					$\beta_{vxd} = 0.6$	$V_{xd} =$	7.833
					$\beta_{vyc} = 0$	$V_{yc} =$	0.000
					$\beta_{vyd} = 0.3$	$V_{yd} =$	3.916
C-3	Type 7	3.76	1.34	9.742	$\beta_{vxc} = 0.94$	$V_{xc} =$	12.271
					$\beta_{vxd} = 0.6$	$V_{xd} =$	7.833
					$\beta_{vyc} = 0$	$V_{yc} =$	0.000
					$\beta_{vyd} = 0.3$	$V_{yd} =$	3.916

C-5 Reinforcement calculation for roof slab

panel	moment		$A_{st} = M_{sd} / Z * f_{yd}$	ϕ	spacing	spacing used	Reinforcement		
P-1	$M_{xs} =$	12.25	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	9.06	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	7.89	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	6.04	238.24	10	329.50	320	ϕ 10	c/c	320
P-2	$M_{xs} =$	11.7	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	8.41	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	9.65	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	7.18	238.24	10	329.50	320	ϕ 10	c/c	320
P-3	$M_{xs} =$	11.4	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	8.41	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	9.65	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	7.18	238.24	10	329.50	320	ϕ 10	c/c	320
P-4	$M_{xs} =$	11.91	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	11.02	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	8.27	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	7.83	238.24	10	329.50	320	ϕ 10	c/c	320
P-5	$M_{xs} =$	7.43	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	5.12	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	4.54	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	3.48	238.24	10	329.50	320	ϕ 10	c/c	320
P-6	$M_{xs} =$	10.85	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{xf} =$	10.61	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{ys} =$	7.2	238.24	10	329.50	320	ϕ 10	c/c	320
	$M_{yf} =$	8.81	238.24	10	329.50	320	ϕ 10	c/c	320

Structural design of G+4 Apartment + shop building B.Sc Thesis project

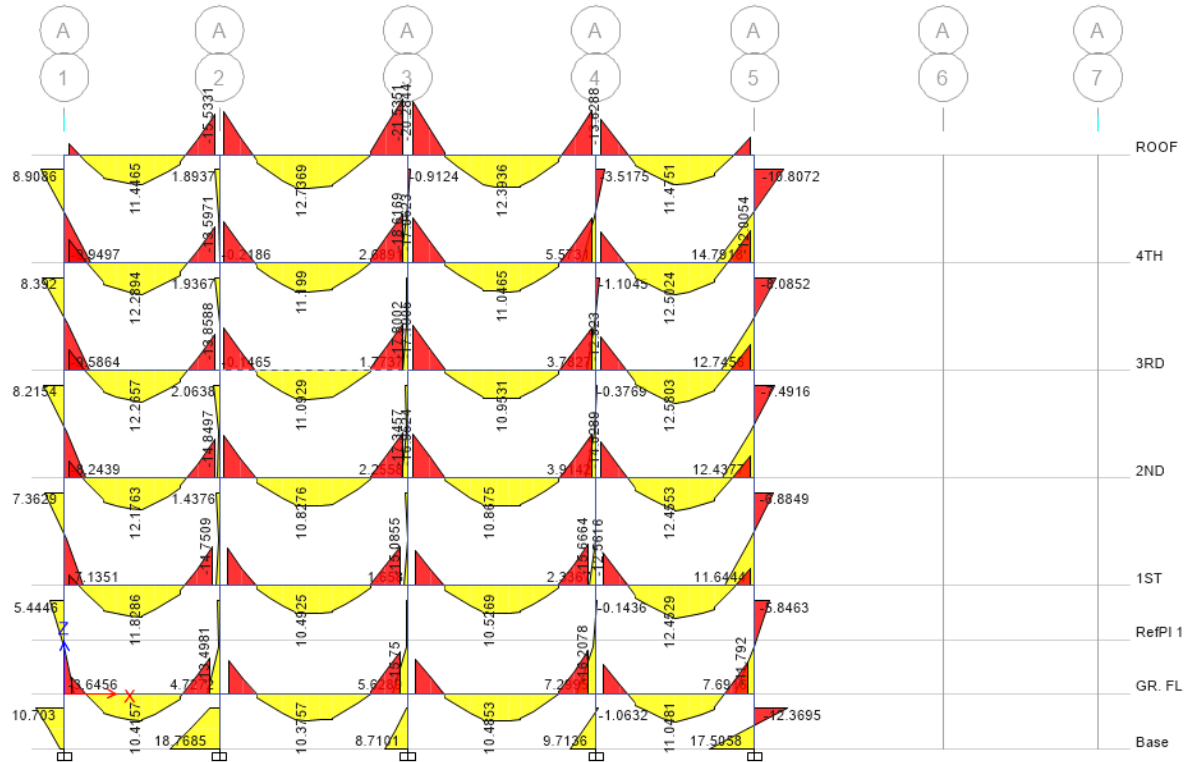
panel	moment		Ast=Msd/Z*fyd	ø	spacing	spacing used	Reinforcement		
P-7	$M_{xs} =$	8.12	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	5.81	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	8.31	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	5.63	238.24	10	329.50	320	ø 10	c/c	320
P-8	$M_{xs} =$	7.02	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	5.97	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	7.58	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	5.35	238.24	10	329.50	320	ø 10	c/c	320
P-9	$M_{xs} =$	8.56	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	6.38	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	7.04	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	5.03	238.24	10	329.50	320	ø 10	c/c	320
P-10	$M_{xs} =$	7.79	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	4.54	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	5.45	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	7.03	238.24	10	329.50	320	ø 10	c/c	320
P-11	$M_{xs} =$	5.37	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	3.83	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	9.17	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	2.78	238.24	10	329.50	320	ø 10	c/c	320
P-12	$M_{xs} =$	7.56	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	6.63	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	4.12	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	3.76	238.24	10	329.50	320	ø 10	c/c	320
P-13	$M_{xs} =$	7.25	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	3.71	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	4.12	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	2.78	238.24	10	329.50	320	ø 10	c/c	320
P-14	$M_{xs} =$	6.92	238.24	10	329.50	320	ø 10	c/c	320
	$M_{xf} =$	7.53	238.24	10	329.50	320	ø 10	c/c	320
	$M_{ys} =$	5.14	238.24	10	329.50	320	ø 10	c/c	320
	$M_{yf} =$	4.96	238.24	10	329.50	320	ø 10	c/c	320

For support moment reinforcement calculation in the above table all left and top side moments are used, the remaining right and bottom side supports are calculated in the table below

panel	moment		$A_{st} = M_{sd} / Z * f_{yd}$	ϕ	spacing	spacing used	Reinforcement		
right side edge supports	P4=	12.27	238.24	10	329.50	320	ϕ 10	c/c	320
	P10=	8.93	238.24	10	329.50	320	ϕ 10	c/c	320
	P14=	5.45	238.24	10	329.50	320	ϕ 10	c/c	320
Bottom edge supports	P5=	6.96	238.24	10	329.50	320	ϕ 10	c/c	320
	P6=	12.05	238.24	10	329.50	320	ϕ 10	c/c	320
	P12=	8.23	238.24	10	329.50	320	ϕ 10	c/c	320
	P14=	8.93	238.24	10	329.50	320	ϕ 10	c/c	320

Appendix D (Beam Reinforcement calculation)

D-1 Longitudinal Reinforcement calculation



Bending moment diagram of beam in axis A

axis A Roof slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	4.21	250	350	0.01	347.86	single	125.125	3500	34.79	125.125	0.8	2ø14
span	1-2	11.45	250	350	0.02	344.13	single	125.125	3500	95.66	125.125	0.8	2ø14
support	2	15.53	250	350	0.03	341.98	single	125.125	3500	130.56	130.5577	0.8	2ø14
span	2-3	12.73	250	350	0.02	343.46	single	125.125	3500	106.56	125.125	0.8	2ø14
support	3	21.53	250	350	0.04	338.78	single	125.125	3500	182.71	182.709	1.2	2ø14
span	3-4	12.39	250	350	0.02	343.64	single	125.125	3500	103.66	125.125	0.8	2ø14
support	4	13.62	250	350	0.02	342.99	single	125.125	3500	114.16	125.125	0.8	2ø14
span	4-5	11.47	250	350	0.02	344.12	single	125.125	3500	95.83	125.125	0.8	2ø14
support	5	6.58	250	350	0.01	346.65	single	125.125	3500	54.57	125.125	0.8	2ø14

axis A 4TH FLOOR													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	8.65	250	350	0.01	345.58	single	125.125	3500	71.96	125.125	0.8	2ø14
span	1-2	12.29	250	350	0.02	343.69	single	125.125	3500	102.81	125.125	0.8	2ø14
support	2	13.6	250	350	0.02	343.00	single	125.125	3500	113.99	125.125	0.8	2ø14
span	2-3	11.2	250	350	0.02	344.26	single	125.125	3500	93.53	125.125	0.8	2ø14
support	3	18.62	250	350	0.03	340.34	single	125.125	3500	157.29	157.2895	1.0	2ø14
span	3-4	11.05	250	350	0.02	344.34	single	125.125	3500	92.26	125.125	0.8	2ø14
support	4	5.57	250	350	0.01	347.17	single	125.125	3500	46.13	125.125	0.8	2ø14
span	4-5	12.5	250	350	0.02	343.58	single	125.125	3500	104.60	125.125	0.8	2ø14
support	5	12.01	250	350	0.02	343.83	single	125.125	3500	100.42	125.125	0.8	2ø14

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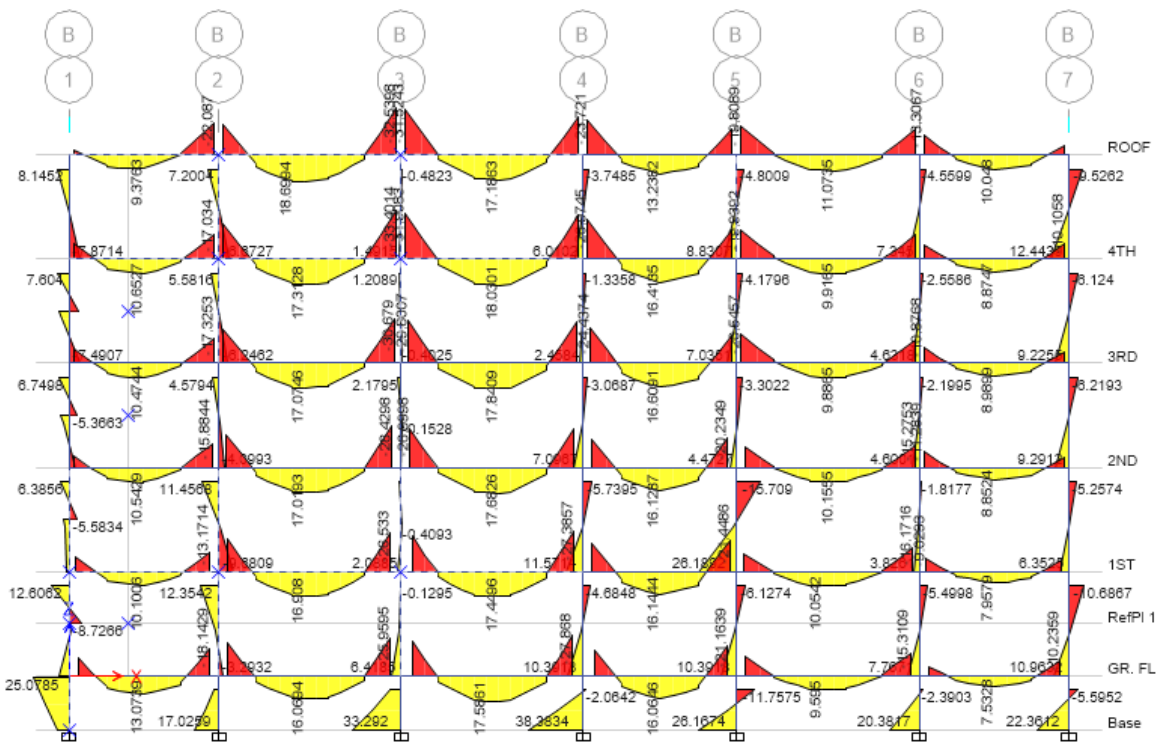
axis A 3RD FLOOR													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	7.64	250	350	0.01	346.10	single	125.125	3500	63.46	125.125	0.8	2ø14
span	1-2	12.27	250	350	0.02	343.70	single	125.125	3500	102.64	125.125	0.8	2ø14
support	2	13.86	250	350	0.02	342.87	single	125.125	3500	116.22	125.125	0.8	2ø14
span	2-3	11.09	250	350	0.02	344.32	single	125.125	3500	92.60	125.125	0.8	2ø14
support	3	17.8	250	350	0.03	340.78	single	125.125	3500	150.17	150.1698	1.0	2ø14
span	3-4	10.95	250	350	0.02	344.39	single	125.125	3500	91.41	125.125	0.8	2ø14
support	4	12.93	250	350	0.02	343.35	single	125.125	3500	108.27	125.125	0.8	2ø14
span	4-5	12.58	250	350	0.02	343.54	single	125.125	3500	105.28	125.125	0.8	2ø14
support	5	9.82	250	350	0.02	344.98	single	125.125	3500	81.84	125.125	0.8	2ø14

axis A 2ND FLOOR													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	6.3	250	350	0.01	346.79	single	125.125	3500	52.23	125.125	0.8	2ø14
span	1-2	12.18	250	350	0.02	343.75	single	125.125	3500	101.87	125.125	0.8	2ø14
support	2	14.85	250	350	0.02	342.34	single	125.125	3500	124.71	125.125	0.8	2ø14
span	2-3	10.83	250	350	0.02	344.45	single	125.125	3500	90.39	125.125	0.8	2ø14
support	3	17.35	250	350	0.03	341.02	single	125.125	3500	146.27	146.2706	1.0	2ø14
span	3-4	10.87	250	350	0.02	344.43	single	125.125	3500	90.73	125.125	0.8	2ø14
support	4	14.02	250	350	0.02	342.78	single	125.125	3500	117.59	125.125	0.8	2ø14
span	4-5	12.46	250	350	0.02	343.60	single	125.125	3500	104.26	125.125	0.8	2ø14
support	5	8.39	250	350	0.01	345.72	single	125.125	3500	69.77	125.125	0.8	2ø14

axis A 1ST FLOOR													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	4.15	250	350	0.01	347.89	single	125.125	3500	34.30	125.125	0.8	2ø14
span	1-2	11.83	250	350	0.02	343.93	single	125.125	3500	98.89	125.125	0.8	2ø14
support	2	14.75	250	350	0.02	342.40	single	125.125	3500	123.85	125.125	0.8	2ø14
span	2-3	10.49	250	350	0.02	344.63	single	125.125	3500	87.51	125.125	0.8	2ø14
support	3	15.09	250	350	0.02	342.22	single	125.125	3500	126.77	126.7725	0.8	2ø14
span	3-4	10.53	250	350	0.02	344.61	single	125.125	3500	87.85	125.125	0.8	2ø14
support	4	15.67	250	350	0.03	341.91	single	125.125	3500	131.76	131.7631	0.9	2ø14
span	4-5	12.45	250	350	0.02	343.60	single	125.125	3500	104.17	125.125	0.8	2ø14
support	5	6.75	250	350	0.01	346.56	single	125.125	3500	56.00	125.125	0.8	2ø14

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axis A GR FLOOR													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	6.28	250	350	0.01	346.80	single	125.125	3500	52.06	125.125	0.8	2ø14
span	1-2	10.42	250	350	0.02	344.66	single	125.125	3500	86.92	125.125	0.8	2ø14
support	2	13.5	250	350	0.02	343.05	single	125.125	3500	113.14	125.125	0.8	2ø14
span	2-3	10.38	250	350	0.02	344.68	single	125.125	3500	86.58	125.125	0.8	2ø14
support	3	15.75	250	350	0.03	341.87	single	125.125	3500	132.45	132.4522	0.9	2ø14
span	3-4	10.47	250	350	0.02	344.64	single	125.125	3500	87.34	125.125	0.8	2ø14
support	4	16.21	250	350	0.03	341.63	single	125.125	3500	136.42	136.4178	0.9	2ø14
span	4-5	11.05	250	350	0.02	344.34	single	125.125	3500	92.26	125.125	0.8	2ø14
support	5	11.79	250	350	0.02	343.95	single	125.125	3500	98.55	125.125	0.8	2ø14



Bending moment diagram of beam in axis B

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axis B Roof slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	3.35	250	350	0.01	348.30	single	125.125	3500	27.65	125.125	0.8	2ø14
span	1-2	9.38	250	350	0.02	345.20	single	125.125	3500	78.12	125.125	0.8	2ø14
support	2	22.09	250	350	0.04	338.48	single	125.125	3500	187.63	187.6286	1.2	2ø14
span	2-3	18.7	250	350	0.03	340.30	single	125.125	3500	157.99	157.9851	1.0	2ø14
support	3	32.54	250	350	0.05	332.74	single	125.125	3500	281.16	281.1585	1.8	2ø14
span	3-4	17.19	250	350	0.03	341.11	single	125.125	3500	144.89	144.8856	0.9	2ø14
support	4	16.21	250	350	0.03	341.63	single	125.125	3500	136.42	136.4178	0.9	2ø14
span	4-5	23.72	250	350	0.04	337.60	single	125.125	3500	202.00	202	1.3	2ø14
support	5	19.81	250	350	0.03	339.71	single	125.125	3500	167.66	167.6553	1.1	2ø14
span	5-6	11.07	250	350	0.02	344.33	single	125.125	3500	92.43	125.125	0.8	2ø14
support	6	13.31	250	350	0.02	343.15	single	125.125	3500	111.51	125.125	0.8	2ø14
span	6-7	10.05	250	350	0.02	344.86	single	125.125	3500	83.78	125.125	0.8	2ø14
support	7	6.18	250	350	0.01	346.86	single	125.125	3500	51.22	125.125	0.8	2ø14

axis B 4TH floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	10.34	250	350	0.02	344.71	single	125.125	3500	86.24	125.125	0.8	2ø14
span	1-2	10.66	250	350	0.02	344.54	single	125.125	3500	88.95	125.125	0.8	2ø14
support	2	17.03	250	350	0.03	341.19	single	125.125	3500	143.50	143.5013	0.9	2ø14
span	2-3	17.31	250	350	0.03	341.04	single	125.125	3500	145.92	145.9243	0.9	2ø14
support	3	33.4	250	350	0.05	332.26	single	125.125	3500	289.01	289.0078	1.9	2ø14
span	3-4	18.03	250	350	0.03	340.66	single	125.125	3500	152.16	152.1649	1.0	2ø14
support	4	26.97	250	350	0.04	335.83	single	125.125	3500	230.89	230.8901	1.5	2ø14
span	4-5	16.41	250	350	0.03	341.52	single	125.125	3500	138.14	138.1438	0.9	2ø14
support	5	20.49	250	350	0.03	339.34	single	125.125	3500	173.60	173.5966	1.1	2ø14

axis B 3rd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	9.72	250	350	0.02	345.03	single	125.125	3500	80.99	125.125	0.8	2ø14
span	1-2	10.47	250	350	0.02	344.64	single	125.125	3500	87.34	125.125	0.8	2ø14
support	2	17.33	250	350	0.03	341.03	single	125.125	3500	146.10	146.0975	0.9	2ø14
span	2-3	17.07	250	350	0.03	341.17	single	125.125	3500	143.85	143.8473	0.9	2ø14
support	3	30.68	250	350	0.05	333.78	single	125.125	3500	264.26	264.2637	1.7	2ø14
span	3-4	17.84	250	350	0.03	340.76	single	125.125	3500	150.52	150.5167	1.0	2ø14
support	4	24.44	250	350	0.04	337.21	single	125.125	3500	208.37	208.3731	1.4	2ø14
span	4-5	16.61	250	350	0.03	341.41	single	125.125	3500	139.87	139.8709	0.9	2ø14
support	5	20.55	250	350	0.03	339.31	single	125.125	3500	174.12	174.1215	1.1	2ø14
span	5-6	9.89	250	350	0.02	344.94	single	125.125	3500	82.43	125.125	0.8	2ø14
support	6	10.88	250	350	0.02	344.42	single	125.125	3500	90.82	125.125	0.8	2ø14
span	6-7	8.99	250	350	0.01	345.41	single	125.125	3500	74.83	125.125	0.8	2ø14
support	7	9.22	250	350	0.02	345.29	single	125.125	3500	76.77	125.125	0.8	2ø14

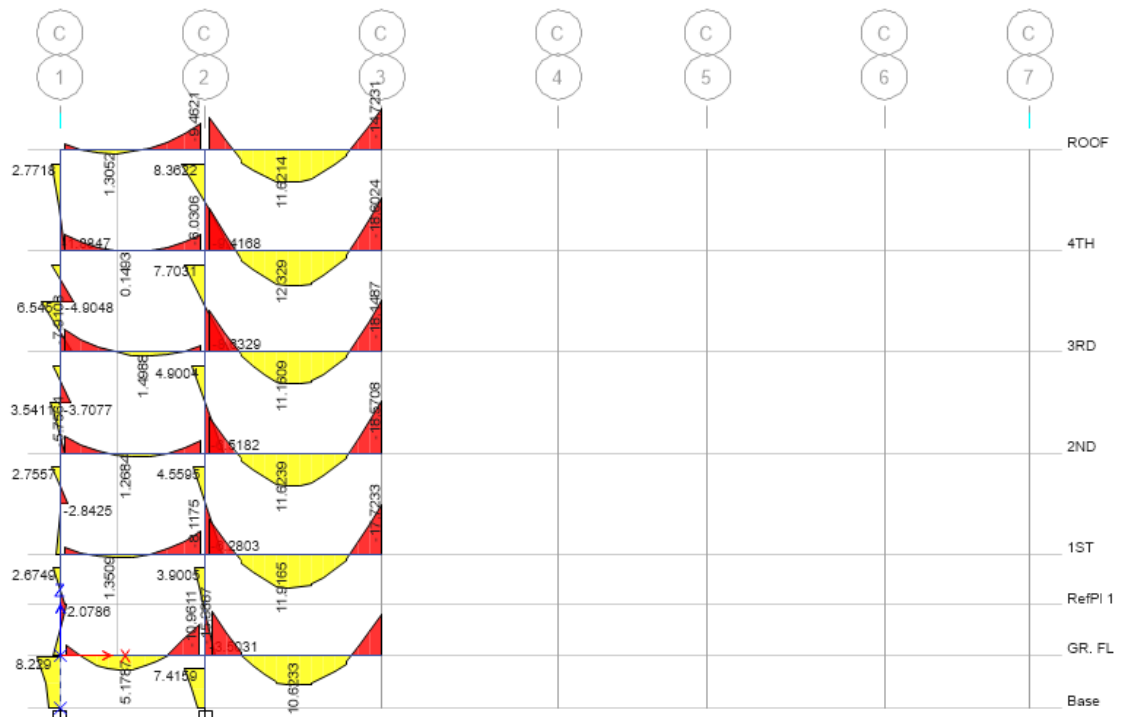
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axis B 2nd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	7.52	250	350	0.01	346.17	single	125.125	3500	62.46	125.125	0.8	2ø14
span	1-2	10.54	250	350	0.02	344.60	single	125.125	3500	87.93	125.125	0.8	2ø14
support	2	15.88	250	350	0.03	341.80	single	125.125	3500	133.57	133.5723	0.9	2ø14
span	2-3	17.02	250	350	0.03	341.20	single	125.125	3500	143.41	143.4148	0.9	2ø14
support	3	28.43	250	350	0.05	335.02	single	125.125	3500	243.97	243.9726	1.6	2ø14
span	3-4	17.86	250	350	0.03	340.75	single	125.125	3500	150.69	150.6901	1.0	2ø14
support	4	26.87	250	350	0.04	335.88	single	125.125	3500	230.00	229.9964	1.5	2ø14
span	4-5	16.13	250	350	0.03	341.67	single	125.125	3500	135.73	135.7277	0.9	2ø14
support	5	20.24	250	350	0.03	339.48	single	125.125	3500	171.41	171.4108	1.1	2ø14
span	5-6	10.16	250	350	0.02	344.80	single	125.125	3500	84.72	125.125	0.8	2ø14
support	6	15.28	250	350	0.02	342.12	single	125.125	3500	128.41	128.4064	0.8	2ø14
span	6-7	8.85	250	350	0.01	345.48	single	125.125	3500	73.65	125.125	0.8	2ø14
support	7	9.29	250	350	0.02	345.25	single	125.125	3500	77.36	125.125	0.8	2ø14

axis B 1st floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	9.72	250	350	0.02	345.03	single	125.125	3500	80.99	125.125	0.8	2ø14
span	1-2	10.1	250	350	0.02	344.83	single	125.125	3500	84.21	125.125	0.8	2ø14
support	2	13.17	250	350	0.02	343.23	single	125.125	3500	110.32	125.125	0.8	2ø14
span	2-3	16.91	250	350	0.03	341.25	single	125.125	3500	142.46	142.4635	0.9	2ø14
support	3	26.53	250	350	0.04	336.07	single	125.125	3500	226.96	226.9602	1.5	2ø14
span	3-4	17.45	250	350	0.03	340.97	single	125.125	3500	147.14	147.1366	1.0	2ø14
support	4	17.39	250	350	0.03	341.00	single	125.125	3500	146.62	146.617	1.0	2ø14
span	4-5	16.14	250	350	0.03	341.66	single	125.125	3500	135.81	135.814	0.9	2ø14
support	5	21.45	250	350	0.04	338.83	single	125.125	3500	182.01	182.0069	1.2	2ø14
span	5-6	10.05	250	350	0.02	344.86	single	125.125	3500	83.78	125.125	0.8	2ø14
support	6	16.17	250	350	0.03	341.65	single	125.125	3500	136.07	136.0727	0.9	2ø14
span	6-7	7.96	250	350	0.01	345.94	single	125.125	3500	66.15	125.125	0.8	2ø14
support	7	8.24	250	350	0.01	345.79	single	125.125	3500	68.51	125.125	0.8	2ø14

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axis B GR floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	12.05	250	350	0.02	343.81	single	125.125	3500	100.76	125.125	0.8	2ø14
span	1-2	13.07	250	350	0.02	343.28	single	125.125	3500	109.46	125.125	0.8	2ø14
support	2	18.14	250	350	0.03	340.60	single	125.125	3500	153.12	153.1196	1.0	2ø14
span	2-3	16.01	250	350	0.03	341.73	single	125.125	3500	134.69	134.6929	0.9	2ø14
support	3	25.96	250	350	0.04	336.38	single	125.125	3500	221.88	221.878	1.4	2ø14
span	3-4	17.59	250	350	0.03	340.89	single	125.125	3500	148.35	148.3495	1.0	2ø14
support	4	27.89	250	350	0.05	335.32	single	125.125	3500	239.13	239.1263	1.6	2ø14
span	4-5	16.06	250	350	0.03	341.70	single	125.125	3500	135.12	135.124	0.9	2ø14
support	5	21.16	250	350	0.03	338.98	single	125.125	3500	179.46	179.4635	1.2	2ø14
span	5-6	9.6	250	350	0.02	345.09	single	125.125	3500	79.98	125.125	0.8	2ø14
support	6	15.31	250	350	0.02	342.10	single	125.125	3500	128.66	128.6644	0.8	2ø14
span	6-7	7.53	250	350	0.01	346.16	single	125.125	3500	62.54	125.125	0.8	2ø14
support	7	10.24	250	350	0.02	344.76	single	125.125	3500	85.39	125.125	0.8	2ø14



Bending moment diagram of beam in axis C

axis C ROOF Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	1.86	250	350	0.00	349.06	single	125.125	3500	15.32	125.125	0.8	2ø14
span	1-2	1.31	250	350	0.00	349.34	single	125.125	3500	10.78	125.125	0.8	2ø14
support	2	9.46	250	350	0.02	345.16	single	125.125	3500	78.80	125.125	0.8	2ø14
span	2-3	11.62	250	350	0.02	344.04	single	125.125	3500	97.10	125.125	0.8	2ø14
support	3	14.72	250	350	0.02	342.41	single	125.125	3500	123.59	125.125	0.8	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis C 4TH Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	5.84	250	350	0.01	347.03	single	125.125	3500	48.38	125.125	0.8	2ø14
span	1-2	0.15	250	350	0.00	349.92	single	125.125	3500	1.23	125.125	0.8	2ø14
support	2	6.03	250	350	0.01	346.93	single	125.125	3500	49.97	125.125	0.8	2ø14
span	2-3	12.33	250	350	0.02	343.67	single	125.125	3500	103.15	125.125	0.8	2ø14
support	3	18.6	250	350	0.03	340.35	single	125.125	3500	157.12	157.1156	1.0	2ø14

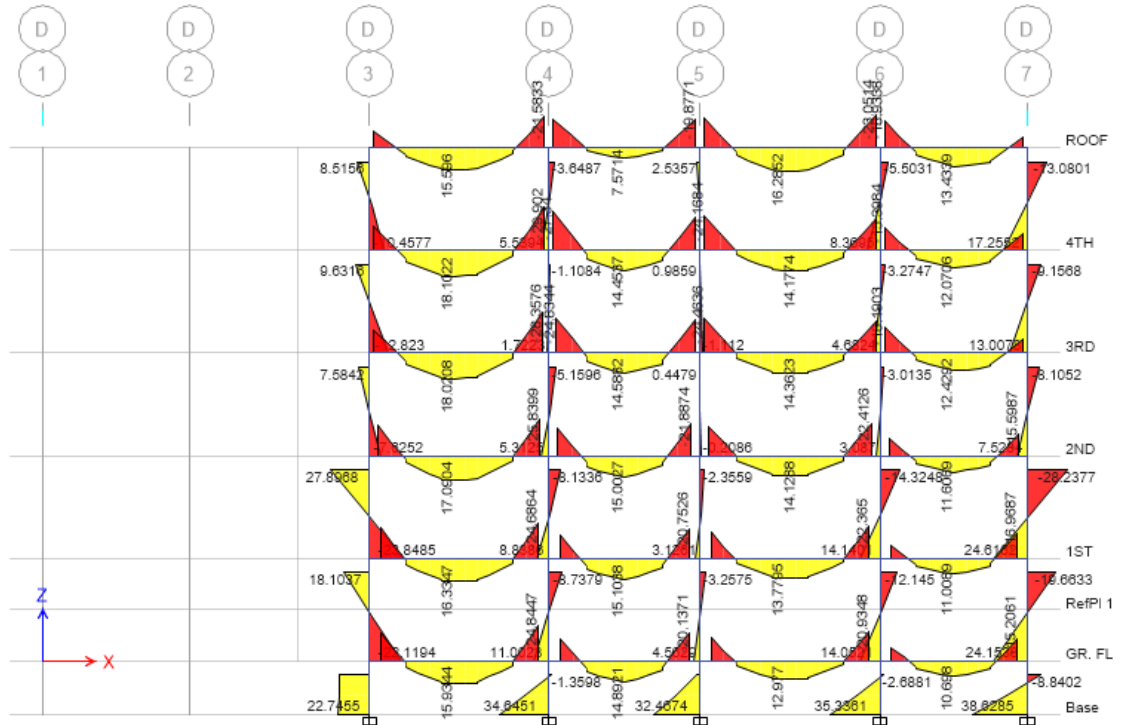
axis C 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	7.91	250	350	0.01	345.96	single	125.125	3500	65.73	125.125	0.8	2ø14
span	1-2	1.5	250	350	0.00	349.24	single	125.125	3500	12.35	125.125	0.8	2ø14
support	2	8.63	250	350	0.01	345.59	single	125.125	3500	71.79	125.125	0.8	2ø14
span	2-3	11.16	250	350	0.02	344.28	single	125.125	3500	93.19	125.125	0.8	2ø14
support	3	18.15	250	350	0.03	340.59	single	125.125	3500	153.21	153.2064	1.0	2ø14

axis C 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	5.76	250	350	0.01	347.07	single	125.125	3500	47.71	125.125	0.8	2ø14
span	1-2	1.27	250	350	0.00	349.36	single	125.125	3500	10.45	125.125	0.8	2ø14
support	2	5.18	250	350	0.01	347.37	single	125.125	3500	42.87	125.125	0.8	2ø14
span	2-3	11.62	250	350	0.02	344.04	single	125.125	3500	97.10	125.125	0.8	2ø14
support	3	18.57	250	350	0.03	340.37	single	125.125	3500	156.85	156.8548	1.0	2ø14

axis C 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	2.59	250	350	0.00	348.69	single	125.125	3500	21.35	125.125	0.8	2ø14
span	1-2	1.35	250	350	0.00	349.32	single	125.125	3500	11.11	125.125	0.8	2ø14
support	2	8.12	250	350	0.01	345.86	single	125.125	3500	67.50	125.125	0.8	2ø14
span	2-3	11.92	250	350	0.02	343.88	single	125.125	3500	99.66	125.125	0.8	2ø14
support	3	17.72	250	350	0.03	340.82	single	125.125	3500	149.48	149.4762	1.0	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis C GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	1	3.33	250	350	0.01	348.31	single	125.125	3500	27.49	125.125	0.8	2ø14
span	1-2	5.18	250	350	0.01	347.37	single	125.125	3500	42.87	125.125	0.8	2ø14
support	2	15.39	250	350	0.03	342.06	single	125.125	3500	129.35	129.3527	0.8	2ø14
span	2-3	10.62	250	350	0.02	344.56	single	125.125	3500	88.61	125.125	0.8	2ø14
support	3	14.43	250	350	0.02	342.57	single	125.125	3500	121.10	125.125	0.8	2ø14



Bending moment diagram of beam in axis D

axis D Roof Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	11.73	250	350	0.02	343.98	single	125.125	3500	98.04	125.125	0.8	2ø14
span	3-4	15.6	250	350	0.03	341.95	single	125.125	3500	131.16	131.1603	0.9	2ø14
support	4	21.58	250	350	0.04	338.76	single	125.125	3500	183.15	183.1479	1.2	2ø14
span	4-5	7.57	250	350	0.01	346.14	single	125.125	3500	62.88	125.125	0.8	2ø14
support	5	19.88	250	350	0.03	339.67	single	125.125	3500	168.27	168.2663	1.1	2ø14
span	5-6	16.29	250	350	0.03	341.58	single	125.125	3500	137.11	137.1081	0.9	2ø14
support	6	23.05	250	350	0.04	337.96	single	125.125	3500	196.08	196.0833	1.3	2ø14
span	6-7	13.43	250	350	0.02	343.09	single	125.125	3500	112.54	125.125	0.8	2ø14
support	7	7.1	250	350	0.01	346.38	single	125.125	3500	58.93	125.125	0.8	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis D 4th Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	16.84	250	350	0.03	341.29	single	125.125	3500	141.86	141.8584	0.9	2ø14
span	3-4	18.1	250	350	0.03	340.62	single	125.125	3500	152.77	152.7724	1.0	2ø14
support	4	29.9	250	350	0.05	334.21	single	125.125	3500	257.21	257.2115	1.7	2ø14
span	4-5	14.45	250	350	0.02	342.55	single	125.125	3500	121.28	125.125	0.8	2ø14
support	5	24.17	250	350	0.04	337.35	single	125.125	3500	205.98	205.9814	1.3	2ø14
span	5-6	14.18	250	350	0.02	342.70	single	125.125	3500	118.96	125.125	0.8	2ø14
support	6	15.39	250	350	0.03	342.06	single	125.125	3500	129.35	129.3527	0.8	2ø14
span	6-7	12.07	250	350	0.02	343.80	single	125.125	3500	100.93	125.125	0.8	2ø14
support	7	11.97	250	350	0.02	343.86	single	125.125	3500	100.08	125.125	0.8	2ø14

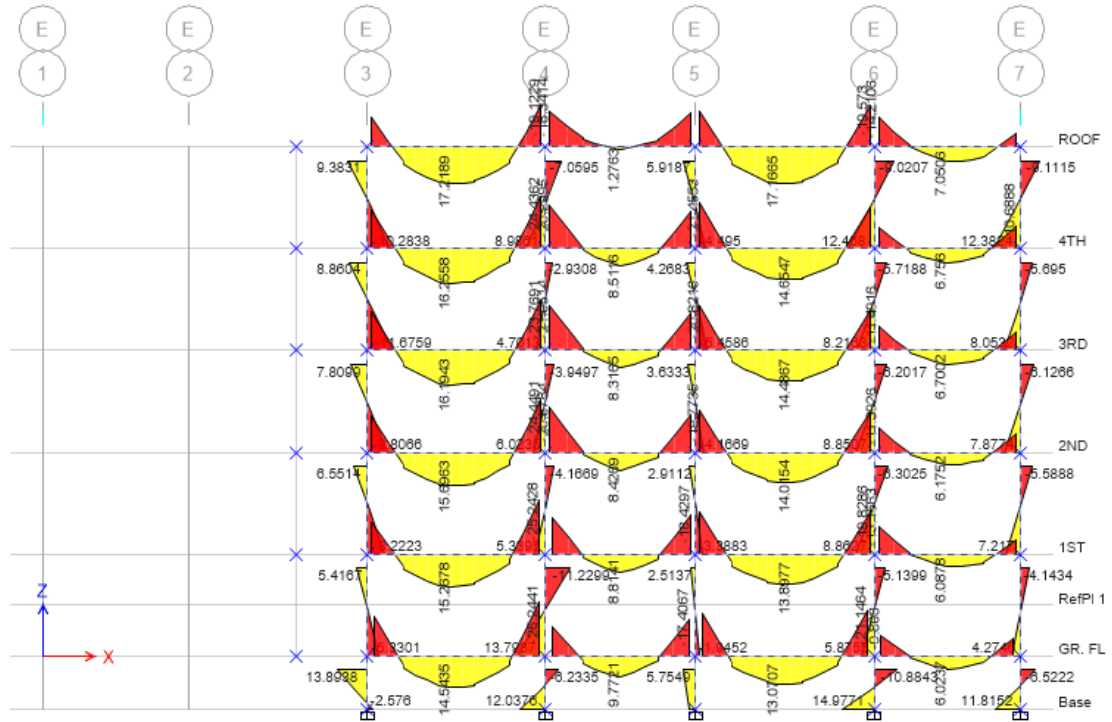
axis D 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	16.44	250	350	0.03	341.50	single	125.125	3500	138.40	138.4028	0.9	2ø14
span	3-4	18.02	250	350	0.03	340.66	single	125.125	3500	152.08	152.0781	1.0	2ø14
support	4	28.36	250	350	0.05	335.06	single	125.125	3500	243.34	243.3439	1.6	2ø14
span	4-5	14.59	250	350	0.02	342.48	single	125.125	3500	122.48	125.125	0.8	2ø14
support	5	24.46	250	350	0.04	337.20	single	125.125	3500	208.55	208.5503	1.4	2ø14
span	5-6	14.36	250	350	0.02	342.60	single	125.125	3500	120.50	125.125	0.8	2ø14
support	6	18.19	250	350	0.03	340.57	single	125.125	3500	153.55	153.5537	1.0	2ø14
span	6-7	12.43	250	350	0.02	343.62	single	125.125	3500	104.00	125.125	0.8	2ø14
support	7	9.33	250	350	0.02	345.23	single	125.125	3500	77.70	125.125	0.8	2ø14

axis D 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	21.03	250	350	0.03	339.05	single	125.125	3500	178.32	178.3242	1.2	2ø14
span	3-4	17.09	250	350	0.03	341.16	single	125.125	3500	144.02	144.0203	0.9	2ø14
support	4	25.84	250	350	0.04	336.44	single	125.125	3500	220.81	220.8093	1.4	2ø14
span	4-5	15	250	350	0.02	342.26	single	125.125	3500	126.00	125.9989	0.8	2ø14
support	5	21.89	250	350	0.04	338.59	single	125.125	3500	185.87	185.8705	1.2	2ø14
span	5-6	14.13	250	350	0.02	342.72	single	125.125	3500	118.53	125.125	0.8	2ø14
support	6	22.41	250	350	0.04	338.31	single	125.125	3500	190.44	190.4438	1.2	2ø14
span	6-7	11.61	250	350	0.02	344.04	single	125.125	3500	97.02	125.125	0.8	2ø14
support	7	15.6	250	350	0.03	341.95	single	125.125	3500	131.16	131.1603	0.9	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis D 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	21.53	250	350	0.04	338.78	single	125.125	3500	182.71	182.709	1.2	2ø14
span	3-4	16.33	250	350	0.03	341.56	single	125.125	3500	137.45	137.4533	0.9	2ø14
support	4	24.69	250	350	0.04	337.07	single	125.125	3500	210.59	210.5895	1.4	2ø14
span	4-5	15.1	250	350	0.02	342.21	single	125.125	3500	126.86	126.8585	0.8	2ø14
support	5	20.75	250	350	0.03	339.20	single	125.125	3500	175.87	175.8718	1.1	2ø14
span	5-6	13.78	250	350	0.02	342.91	single	125.125	3500	115.53	125.125	0.8	2ø14
support	6	22.37	250	350	0.04	338.33	single	125.125	3500	190.09	190.0918	1.2	2ø14
span	6-7	11.01	250	350	0.02	344.36	single	125.125	3500	91.92	125.125	0.8	2ø14
support	7	16.97	250	350	0.03	341.22	single	125.125	3500	142.98	142.9824	0.9	2ø14

axis D GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	20.14	250	350	0.03	339.53	single	125.125	3500	170.54	170.537	1.1	2ø14
span	3-4	15.93	250	350	0.03	341.77	single	125.125	3500	134.00	134.0033	0.9	2ø14
support	4	24.84	250	350	0.04	336.99	single	125.125	3500	211.92	211.9203	1.4	2ø14
span	4-5	14.89	250	350	0.02	342.32	single	125.125	3500	125.05	125.125	0.8	2ø14
support	5	20.14	250	350	0.03	339.53	single	125.125	3500	170.54	170.537	1.1	2ø14
span	5-6	12.98	250	350	0.02	343.33	single	125.125	3500	108.69	125.125	0.8	2ø14
support	6	20.93	250	350	0.03	339.11	single	125.125	3500	177.45	177.4481	1.2	2ø14
span	6-7	10.7	250	350	0.02	344.52	single	125.125	3500	89.29	125.125	0.8	2ø14
support	7	15.21	250	350	0.02	342.15	single	125.125	3500	127.80	127.8043	0.8	2ø14



Bending moment diagram of beam in axis E

axis E Roof Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	13.97	250	350	0.02	342.81	single	125.125	3500	117.16	125.125	0.8	2ø14
span	3-4	17.22	250	350	0.03	341.09	single	125.125	3500	145.15	145.1453	0.9	2ø14
support	4	19.12	250	350	0.03	340.08	single	125.125	3500	161.64	161.64	1.1	2ø14
span	4-5	1.28	250	350	0.00	349.35	single	125.125	3500	10.53	125.125	0.8	2ø14
support	5	15.91	250	350	0.03	341.78	single	125.125	3500	133.83	133.8309	0.9	2ø14
span	5-6	17.17	250	350	0.03	341.12	single	125.125	3500	144.71	144.7125	0.9	2ø14
support	6	19.57	250	350	0.03	339.84	single	125.125	3500	165.56	165.5615	1.1	2ø14
span	6-7	7.05	250	350	0.01	346.41	single	125.125	3500	58.51	125.125	0.8	2ø14
support	7	6.32	250	350	0.01	346.78	single	125.125	3500	52.40	125.125	0.8	2ø14

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axis E 4th Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	18.3	250	350	0.03	340.51	single	125.125	3500	154.51	154.5088	1.0	2ø14
span	3-4	16.26	250	350	0.03	341.60	single	125.125	3500	136.85	136.8492	0.9	2ø14
support	4	24.44	250	350	0.04	337.21	single	125.125	3500	208.37	208.3731	1.4	2ø14
span	4-5	8.51	250	350	0.01	345.65	single	125.125	3500	70.78	125.125	0.8	2ø14
support	5	21.46	250	350	0.04	338.82	single	125.125	3500	182.09	182.0947	1.2	2ø14
span	5-6	14.65	250	350	0.02	342.45	single	125.125	3500	122.99	125.125	0.8	2ø14
support	6	19.41	250	350	0.03	339.92	single	125.125	3500	164.17	164.1665	1.1	2ø14
span	6-7	6.76	250	350	0.01	346.56	single	125.125	3500	56.08	125.125	0.8	2ø14
support	7	10.69	250	350	0.02	344.52	single	125.125	3500	89.21	125.125	0.8	2ø14

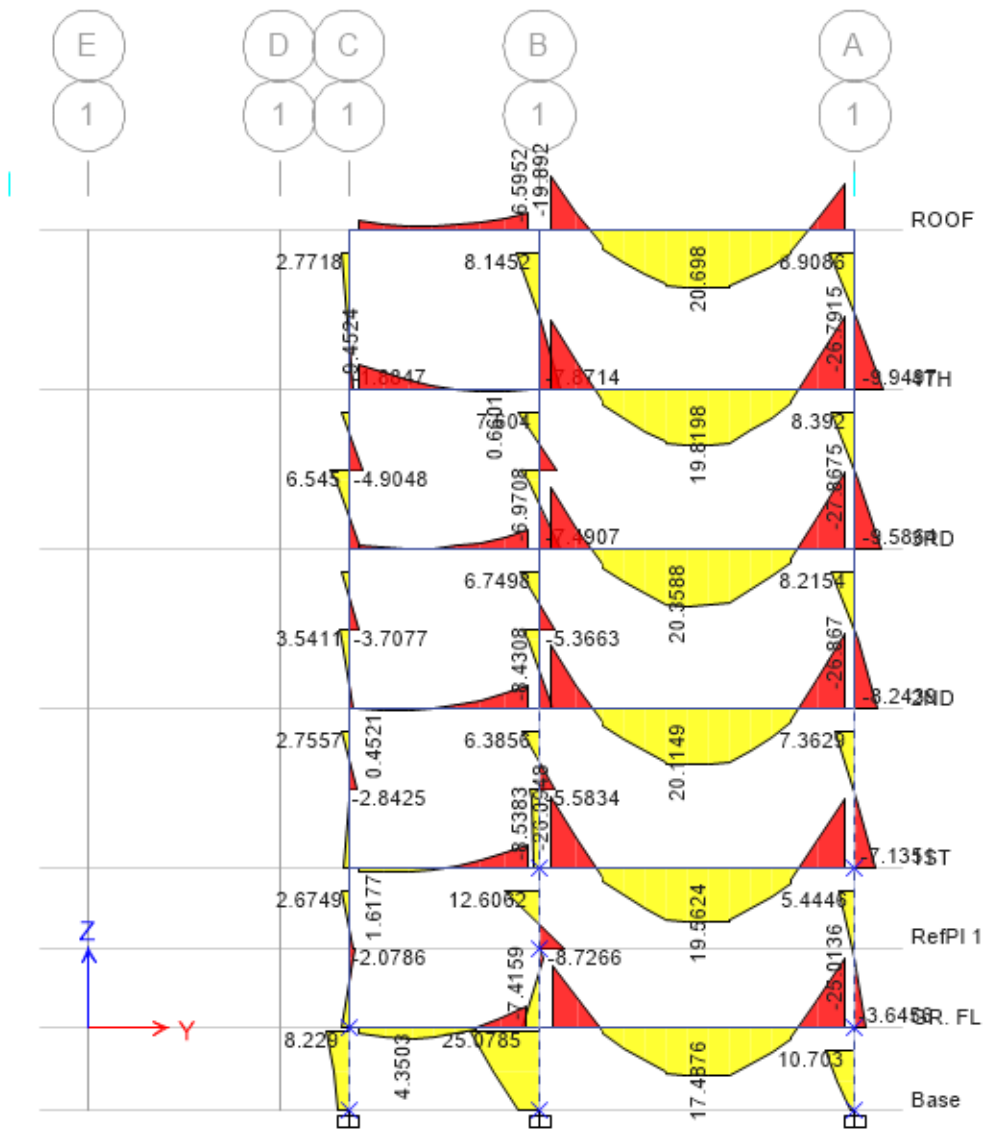
axis E 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	18.11	250	350	0.03	340.62	single	125.125	3500	152.86	152.8592	1.0	2ø14
span	3-4	16.19	250	350	0.03	341.64	single	125.125	3500	136.25	136.2453	0.9	2ø14
support	4	23.77	250	350	0.04	337.57	single	125.125	3500	202.44	202.4421	1.3	2ø14
span	4-5	8.32	250	350	0.01	345.75	single	125.125	3500	69.18	125.125	0.8	2ø14
support	5	20.82	250	350	0.03	339.17	single	125.125	3500	176.48	176.4847	1.1	2ø14
span	5-6	14.49	250	350	0.02	342.53	single	125.125	3500	121.62	125.125	0.8	2ø14
support	6	11.49	250	350	0.02	344.11	single	125.125	3500	96.00	125.125	0.8	2ø14
span	6-7	6.7	250	350	0.01	346.59	single	125.125	3500	55.58	125.125	0.8	2ø14
support	7	8.56	250	350	0.01	345.63	single	125.125	3500	71.20	125.125	0.8	2ø14

axis E 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	16.84	250	350	0.03	341.29	single	125.125	3500	141.86	141.8584	0.9	2ø14
span	3-4	15.7	250	350	0.03	341.90	single	125.125	3500	132.02	132.0215	0.9	2ø14
support	4	24.45	250	350	0.04	337.20	single	125.125	3500	208.46	208.4617	1.4	2ø14
span	4-5	8.43	250	350	0.01	345.70	single	125.125	3500	70.11	125.125	0.8	2ø14
support	5	19.77	250	350	0.03	339.73	single	125.125	3500	167.31	167.3062	1.1	2ø14
span	5-6	14.02	250	350	0.02	342.78	single	125.125	3500	117.59	125.125	0.8	2ø14
support	6	11.39	250	350	0.02	344.16	single	125.125	3500	95.15	125.125	0.8	2ø14
span	6-7	6.18	250	350	0.01	346.86	single	125.125	3500	51.22	125.125	0.8	2ø14
support	7	8.35	250	350	0.01	345.74	single	125.125	3500	69.43	125.125	0.8	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis E 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	15.33	250	350	0.03	342.09	single	125.125	3500	128.84	128.8365	0.8	2ø14
span	3-4	15.27	250	350	0.02	342.12	single	125.125	3500	128.32	128.3204	0.8	2ø14
support	4	25.24	250	350	0.04	336.77	single	125.125	3500	215.47	215.4723	1.4	2ø14
span	4-5	8.81	250	350	0.01	345.50	single	125.125	3500	73.31	125.125	0.8	2ø14
support	5	18.43	250	350	0.03	340.45	single	125.125	3500	155.64	155.6381	1.0	2ø14
span	5-6	13.9	250	350	0.02	342.84	single	125.125	3500	116.56	125.125	0.8	2ø14
support	6	19.83	250	350	0.03	339.70	single	125.125	3500	167.83	167.8299	1.1	2ø14
span	6-7	6.09	250	350	0.01	346.90	single	125.125	3500	50.47	125.125	0.8	2ø14
support	7	6.75	250	350	0.01	346.56	single	125.125	3500	56.00	125.125	0.8	2ø14

axis E GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	3	18.53	250	350	0.03	340.39	single	125.125	3500	156.51	156.5071	1.0	2ø14
span	3-4	14.54	250	350	0.02	342.51	single	125.125	3500	122.05	125.125	0.8	2ø14
support	4	25.24	250	350	0.04	336.77	single	125.125	3500	215.47	215.4723	1.4	2ø14
span	4-5	9.77	250	350	0.02	345.00	single	125.125	3500	81.42	125.125	0.8	2ø14
support	5	17.41	250	350	0.03	340.99	single	125.125	3500	146.79	146.7902	1.0	2ø14
span	5-6	13.07	250	350	0.02	343.28	single	125.125	3500	109.46	125.125	0.8	2ø14
support	6	21.15	250	350	0.03	338.99	single	125.125	3500	179.38	179.3759	1.2	2ø14
span	6-7	6.02	250	350	0.01	346.94	single	125.125	3500	49.89	125.125	0.8	2ø14
support	7	8.28	250	350	0.01	345.77	single	125.125	3500	68.85	125.125	0.8	2ø14



Bending moment diagram of beam in axis 1

axis 1 Roof Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	mnK	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	3.73	250	350	0.01	348.11	single	125.125	3500	30.81	125.125	1.6	2ø14
span	C-B	1.68	250	350	0.00	349.15	single	125.125	3500	13.83	125.125	1.6	2ø14
support	B	19.89	250	350	0.03	339.66	single	125.125	3500	168.35	168.3536	2.1	2ø14
span	B-A	20.7	250	350	0.03	339.23	single	125.125	3500	175.43	175.4341	2.2	3ø14
support	A	16.51	250	350	0.03	341.47	single	125.125	3500	139.01	139.0072	1.8	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 1 4TH Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	9.45	250	350	0.02	345.17	single	125.125	3500	78.71	125.125	1.6	2ø14
span	C-B	0.62	250	350	0.00	349.69	single	125.125	3500	5.10	125.125	1.6	2ø14
support	B	26.79	250	350	0.04	335.92	single	125.125	3500	229.28	229.2817	2.9	3ø14
span	B-A	19.82	250	350	0.03	339.70	single	125.125	3500	167.74	167.7426	2.1	3ø14
support	A	26.79	250	350	0.04	335.92	single	125.125	3500	229.28	229.2817	2.9	3ø14

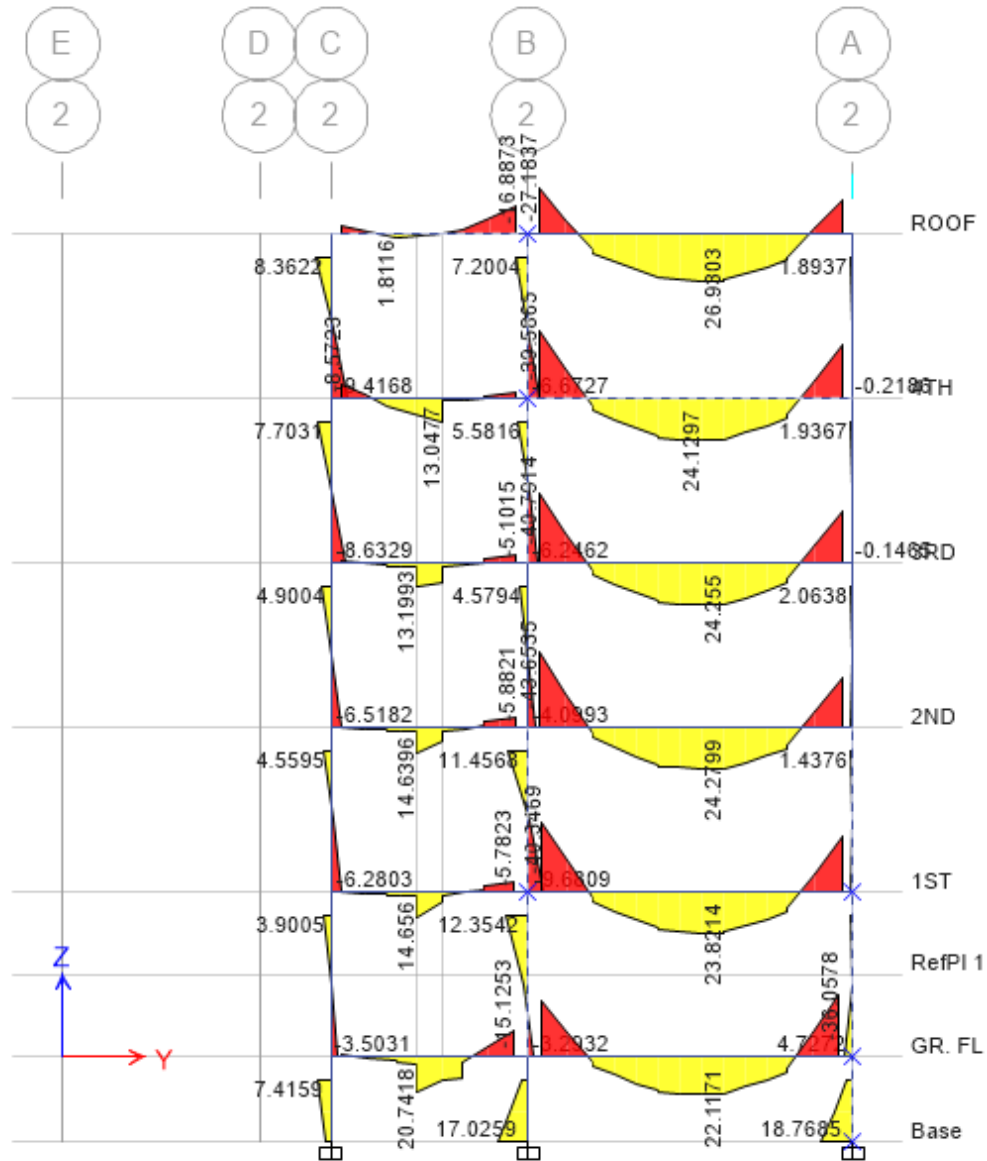
axis 1 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	1.61	250	350	0.00	349.19	single	125.125	3500	13.26	125.125	1.6	2ø14
span	C-B	0.21	250	350	0.00	349.89	single	125.125	3500	1.73	125.125	1.6	2ø14
support	B	7.49	250	350	0.01	346.18	single	125.125	3500	62.20	125.125	1.6	2ø14
span	B-A	20.36	250	350	0.03	339.41	single	125.125	3500	172.46	172.4598	2.2	3ø14
support	A	27.87	250	350	0.05	335.33	single	125.125	3500	238.95	238.9469	3.0	3ø14

axis 1 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	0.18	250	350	0.00	349.91	single	125.125	3500	1.48	125.125	1.6	2ø14
span	C-B	0.45	250	350	0.00	349.77	single	125.125	3500	3.70	125.125	1.6	2ø14
support	B	8.43	250	350	0.01	345.70	single	125.125	3500	70.11	125.125	1.6	2ø14
span	B-A	20.11	250	350	0.03	339.55	single	125.125	3500	170.27	170.2749	2.2	3ø14
support	A	26.87	250	350	0.04	335.88	single	125.125	3500	230.00	229.9964	2.9	3ø14

axis 1 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	1.18	250	350	0.00	349.40	single	125.125	3500	9.71	125.125	1.6	2ø14
span	C-B	1.58	250	350	0.00	349.20	single	125.125	3500	13.01	125.125	1.6	2ø14
support	B	8.54	250	350	0.01	345.64	single	125.125	3500	71.04	125.125	1.6	2ø14
span	B-A	19.56	250	350	0.03	339.84	single	125.125	3500	165.47	165.4743	2.1	3ø14
support	A	25.43	250	350	0.04	336.67	single	125.125	3500	217.16	217.1611	2.8	3ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 1 GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	1.83	250	350	0.00	349.07	single	125.125	3500	15.07	125.125	1.6	2ø14
span	C-B	4.35	250	350	0.01	347.79	single	125.125	3500	35.96	125.125	1.6	2ø14
support	B	7.42	250	350	0.01	346.22	single	125.125	3500	61.62	125.125	1.6	2ø14
span	B-A	17.49	250	350	0.03	340.95	single	125.125	3500	147.48	147.4831	1.9	2ø14
support	A	25.01	250	350	0.04	336.90	single	125.125	3500	213.43	213.4293	2.7	3ø14



Bending moment diagram of beam in axis 2

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 2 Roof slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	5.34	250	350	0.01	347.29	single	125.125	3500	44.21	125.125	0.8	2ø14
span	C-B	1.81	250	350	0.00	349.08	single	125.125	3500	14.91	125.125	0.8	2ø14
support	B	27.18	250	350	0.04	335.71	single	125.125	3500	232.77	232.7678	1.5	2ø14
span	B-A	26.93	250	350	0.04	335.85	single	125.125	3500	230.53	230.5326	1.5	2ø14
support	A	20.38	250	350	0.03	339.40	single	125.125	3500	172.63	172.6347	1.1	2ø14

axis 2 4TH Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	8.57	250	350	0.01	345.62	single	125.125	3500	71.29	125.125	0.8	2ø14
span	C-B	13.05	250	350	0.02	343.29	single	125.125	3500	109.29	125.125	0.8	2ø14
support	B	39.59	250	350	0.06	328.74	single	125.125	3500	346.23	346.2301	2.3	3ø14
span	B-A	24.14	250	350	0.04	337.37	single	125.125	3500	205.72	205.7158	1.3	2ø14
support	A	31.43	250	350	0.05	333.36	single	125.125	3500	271.06	271.0628	1.8	2ø14

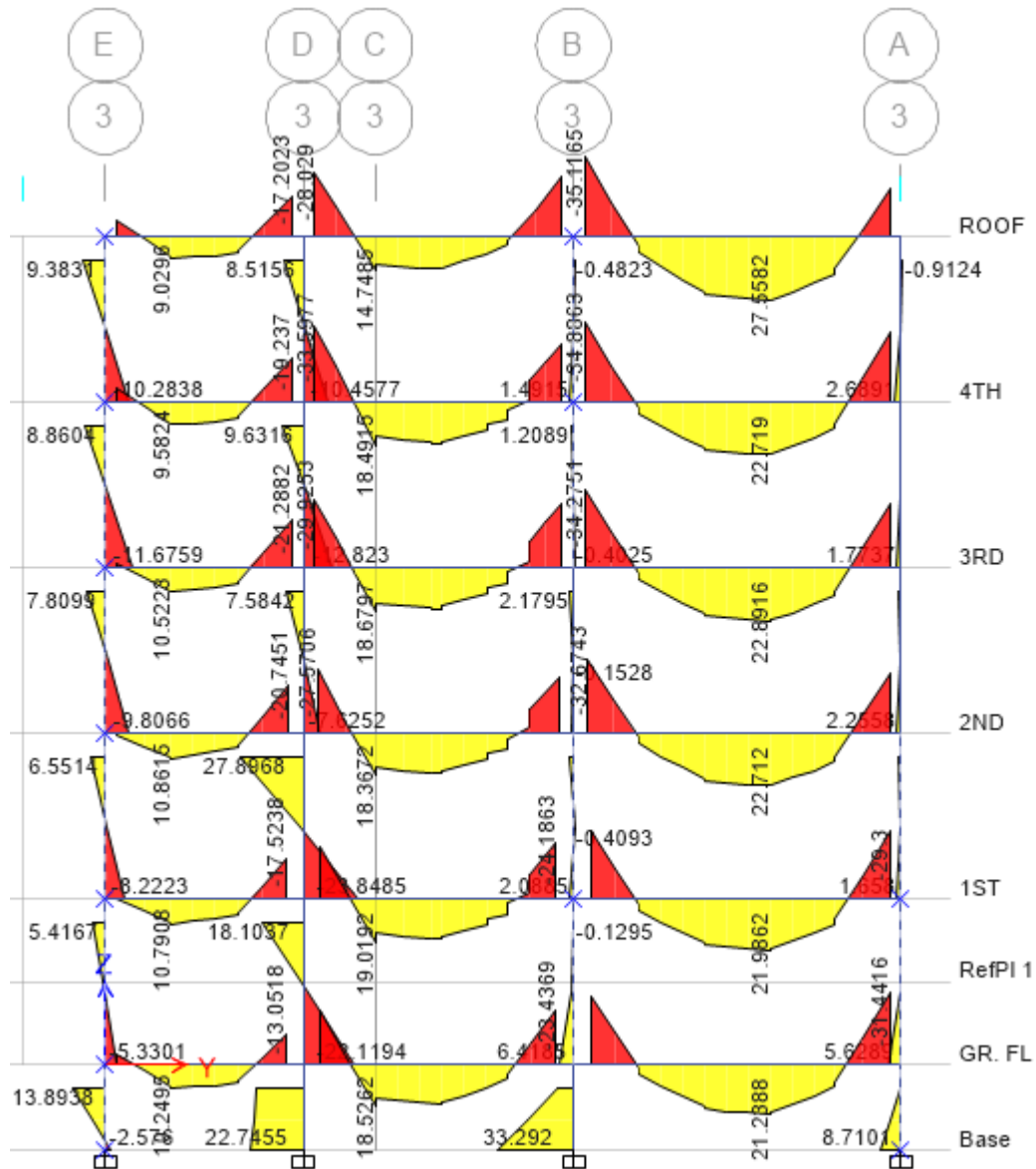
axis 2 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	0.94	250	350	0.00	349.53	single	125.125	3500	7.73	125.125	0.8	2ø14
span	C-B	13.2	250	350	0.02	343.21	single	125.125	3500	110.57	125.125	0.8	2ø14
support	B	40.79	250	350	0.07	328.05	single	125.125	3500	357.48	357.4753	2.3	3ø14
span	B-A	24.26	250	350	0.04	337.31	single	125.125	3500	206.78	206.7784	1.3	2ø14
support	A	30.62	250	350	0.05	333.81	single	125.125	3500	263.72	263.7205	1.7	2ø14

axis 2 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	0.47	250	350	0.00	349.76	single	125.125	3500	3.86	125.125	0.8	2ø14
span	C-B	14.64	250	350	0.02	342.45	single	125.125	3500	122.91	125.125	0.8	2ø14
support	B	43.65	250	350	0.07	326.40	single	125.125	3500	384.48	384.483	2.5	3ø14
span	B-A	24.28	250	350	0.04	337.29	single	125.125	3500	206.96	206.9555	1.3	2ø14
support	A	29.36	250	350	0.05	334.51	single	125.125	3500	252.34	252.3404	1.6	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 2 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	0.16	250	350	0.00	349.92	single	125.125	3500	1.31	125.125	0.8	2ø14
span	C-B	14.66	250	350	0.02	342.44	single	125.125	3500	123.08	125.125	0.8	2ø14
support	B	40.35	250	350	0.07	328.31	single	125.125	3500	353.35	353.3462	2.3	3ø14
span	B-A	23.82	250	350	0.04	337.54	single	125.125	3500	202.88	202.8843	1.3	2ø14
support	A	32.07	250	350	0.05	333.00	single	125.125	3500	276.88	276.8789	1.8	2ø14

axis 2 GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	C	1.44	250	350	0.00	349.27	single	125.125	3500	11.85	125.125	0.8	2ø14
span	C-B	20.74	250	350	0.03	339.21	single	125.125	3500	175.78	175.7843	1.1	2ø14
support	B	15.13	250	350	0.02	342.20	single	125.125	3500	127.12	127.1164	0.8	2ø14
span	B-A	22.12	250	350	0.04	338.47	single	125.125	3500	187.89	187.8924	1.2	2ø14
support	A	36.06	250	350	0.06	330.76	single	125.125	3500	313.44	313.4396	2.0	2ø14



Bending moment diagram of beam in axis 3

axis 3 Roof slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	7.41	250	350	0.01	346.22	single	125.125	3500	61.53	125.125	0.8	2ø14
span	E-D	9.1	250	350	0.01	345.35	single	125.125	3500	75.76	125.125	0.8	2ø14
support	D	28.1	250	350	0.05	335.20	single	125.125	3500	241.01	241.0099	1.6	2ø14
span	D-B	35.12	250	350	0.06	331.29	single	125.125	3500	304.78	304.779	2.0	2ø14
support	B	27.56	250	350	0.04	335.50	single	125.125	3500	236.17	236.169	1.5	2ø14
span	B-A	27.56	250	350	0.04	335.50	single	125.125	3500	236.17	236.169	1.5	2ø14
support	A	20.71	250	350	0.03	339.22	single	125.125	3500	175.52	175.5217	1.1	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 3 4th floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	5.96	250	350	0.01	346.97	single	125.125	3500	49.38	125.125	0.8	2ø14
span	E-D	9.58	250	350	0.02	345.10	single	125.125	3500	79.81	125.125	0.8	2ø14
support	D	33.6	250	350	0.05	332.15	single	125.125	3500	290.84	290.8367	1.9	2ø14
span	D-B	18.49	250	350	0.03	340.41	single	125.125	3500	156.16	156.1595	1.0	2ø14
support	B	34.89	250	350	0.06	331.42	single	125.125	3500	302.66	302.6644	2.0	2ø14
span	B-A	22.72	250	350	0.04	338.14	single	125.125	3500	193.17	193.174	1.3	2ø14
support	A	30.17	250	350	0.05	334.06	single	125.125	3500	259.65	259.6504	1.7	2ø14

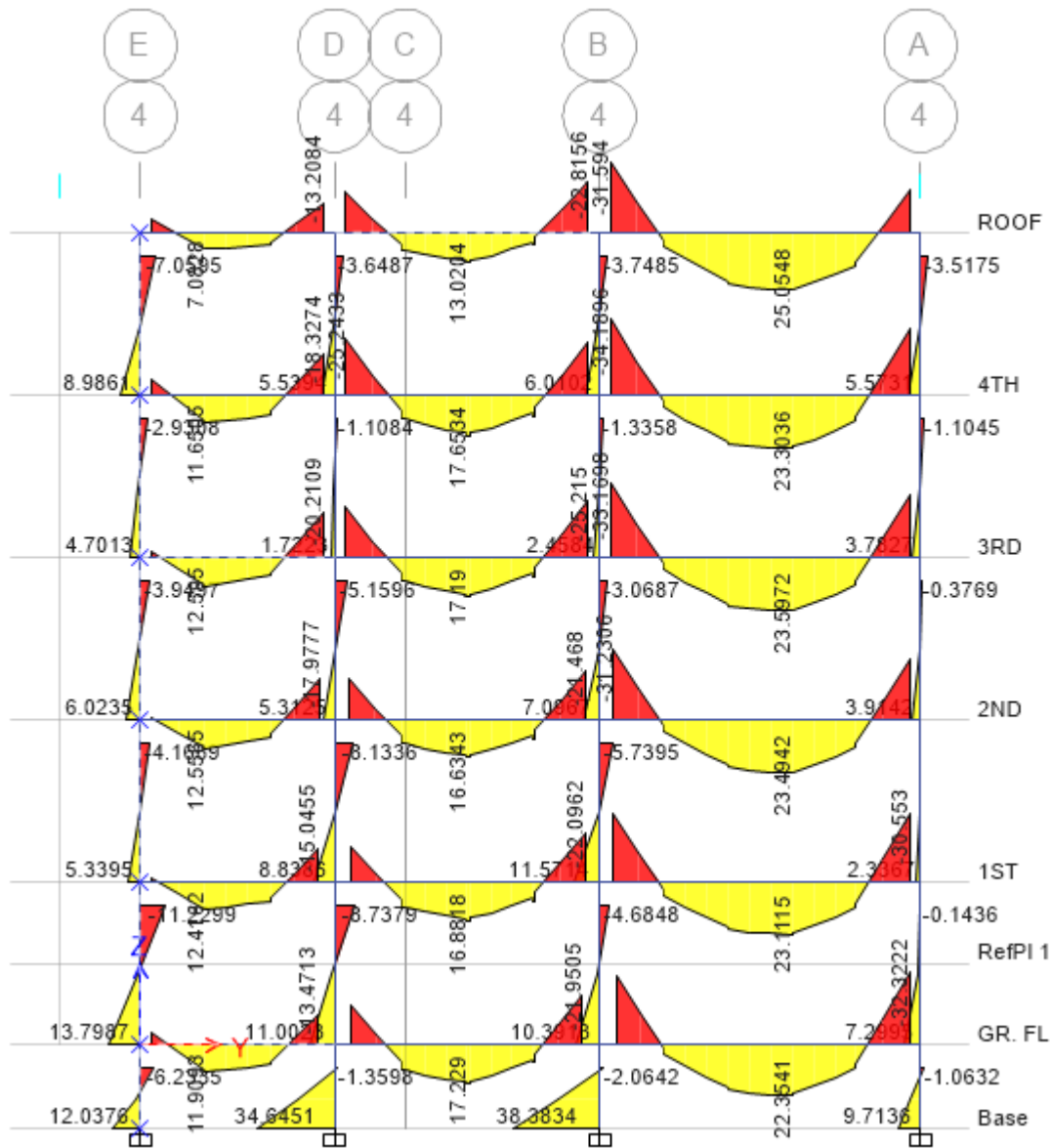
axis 3 3rd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.92	250	350	0.00	349.03	single	125.125	3500	15.82	125.125	0.8	2ø14
span	E-D	10.52	250	350	0.02	344.61	single	125.125	3500	87.77	125.125	0.8	2ø14
support	D	29.93	250	350	0.05	334.19	single	125.125	3500	257.48	257.4823	1.7	2ø14
span	D-B	18.78	250	350	0.03	340.26	single	125.125	3500	158.68	158.6809	1.0	2ø14
support	B	34.28	250	350	0.06	331.76	single	125.125	3500	297.06	297.0646	1.9	2ø14
span	B-A	22.89	250	350	0.04	338.05	single	125.125	3500	194.67	194.6723	1.3	2ø14
support	A	27.68	250	350	0.05	335.44	single	125.125	3500	237.24	237.244	1.5	2ø14

axis 3 2nd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	0.49	250	350	0.00	349.75	single	125.125	3500	4.03	125.125	0.8	2ø14
span	E-D	10.86	250	350	0.02	344.43	single	125.125	3500	90.65	125.125	0.8	2ø14
support	D	27.57	250	350	0.05	335.50	single	125.125	3500	236.26	236.2586	1.5	2ø14
span	D-B	18.38	250	350	0.03	340.47	single	125.125	3500	155.20	155.2037	1.0	2ø14
support	B	32.67	250	350	0.05	332.67	single	125.125	3500	282.34	282.3435	1.8	2ø14
span	B-A	22.71	250	350	0.04	338.15	single	125.125	3500	193.09	193.0859	1.3	2ø14
support	A	26.78	250	350	0.04	335.93	single	125.125	3500	229.19	229.1924	1.5	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 3 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	0.35	250	350	0.00	349.82	single	125.125	3500	2.88	125.125	0.8	2ø14
span	E-D	10.79	250	350	0.02	344.47	single	125.125	3500	90.05	125.125	0.8	2ø14
support	D	23.85	250	350	0.04	337.53	single	125.125	3500	203.15	203.1496	1.3	2ø14
span	D-B	19.02	250	350	0.03	340.13	single	125.125	3500	160.77	160.7693	1.0	2ø14
support	B	24.19	250	350	0.04	337.34	single	125.125	3500	206.16	206.1585	1.3	2ø14
span	B-A	21.99	250	350	0.04	338.54	single	125.125	3500	186.75	186.7494	1.2	2ø14
support	A	29.3	250	350	0.05	334.54	single	125.125	3500	251.80	251.7997	1.6	2ø14

axis 3 GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.85	250	350	0.01	347.54	single	125.125	3500	40.12	125.125	0.8	2ø14
span	E-D	10.25	250	350	0.02	344.75	single	125.125	3500	85.48	125.125	0.8	2ø14
support	D	22.12	250	350	0.04	338.47	single	125.125	3500	187.89	187.8924	1.2	2ø14
span	D-B	18.53	250	350	0.03	340.39	single	125.125	3500	156.51	156.5071	1.0	2ø14
support	B	23.44	250	350	0.04	337.75	single	125.125	3500	199.53	199.5258	1.3	2ø14
span	B-A	21.22	250	350	0.03	338.95	single	125.125	3500	179.99	179.9896	1.2	2ø14
support	A	31.44	250	350	0.05	333.35	single	125.125	3500	271.15	271.1536	1.8	2ø14



Bending moment diagram of beam in axis 4

axis 4 Roof Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	6.65	250	350	0.01	346.61	single	125.125	3500	55.16	125.125	1.6	2ø14
span	E-D	7.08	250	350	0.01	346.39	single	125.125	3500	58.76	125.125	1.6	2ø14
support	D	13.21	250	350	0.02	343.21	single	125.125	3500	110.66	125.125	1.6	2ø14
span	D-B	13.02	250	350	0.02	343.31	single	125.125	3500	109.04	125.125	1.6	2ø14
support	B	31.59	250	350	0.05	333.27	single	125.125	3500	272.52	272.5156	3.5	4ø14
span	B-A	25.05	250	350	0.04	336.88	single	125.125	3500	213.78	213.7845	2.7	3ø14
support	A	19.32	250	350	0.03	339.97	single	125.125	3500	163.38	163.3822	2.1	3ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 4 4th floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	7.07	250	350	0.01	346.40	single	125.125	3500	58.68	125.125	1.6	2ø14
span	E-D	11.65	250	350	0.02	344.02	single	125.125	3500	97.36	125.125	1.6	2ø14
support	D	25.24	250	350	0.04	336.77	single	125.125	3500	215.47	215.4723	2.7	3ø14
span	D-B	17.65	250	350	0.03	340.86	single	125.125	3500	148.87	148.8695	1.9	2ø14
support	B	34.19	250	350	0.06	331.81	single	125.125	3500	296.24	296.2395	3.8	4ø14
span	B-A	23.3	250	350	0.04	337.83	single	125.125	3500	198.29	198.2895	2.5	3ø14
support	A	29.89	250	350	0.05	334.21	single	125.125	3500	257.12	257.1212	3.3	4ø14

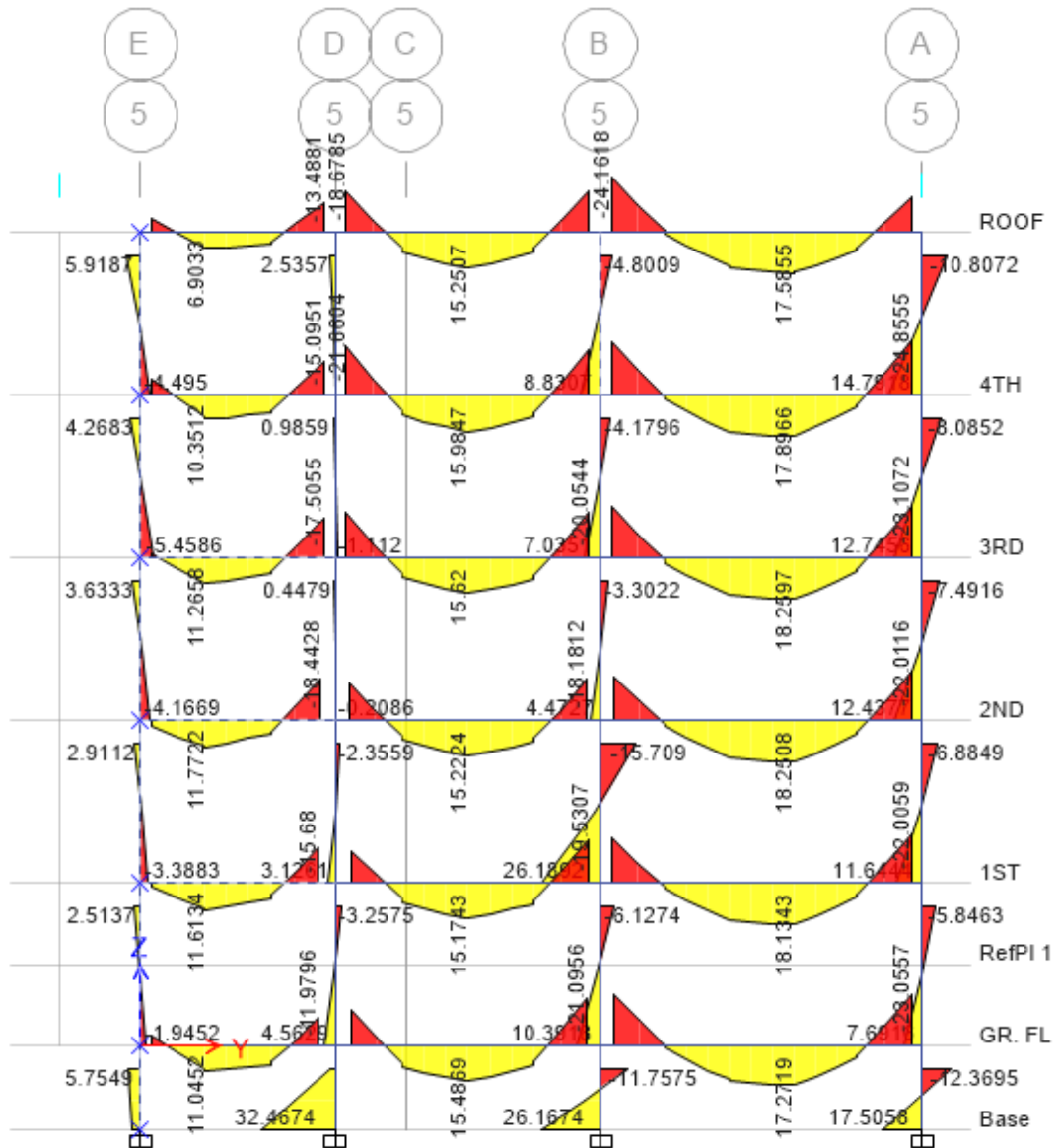
axis 4 3rd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	2.64	250	350	0.00	348.66	single	125.125	3500	21.77	125.125	1.6	2ø14
span	E-D	12.6	250	350	0.02	343.53	single	125.125	3500	105.45	125.125	1.6	2ø14
support	D	20.21	250	350	0.03	339.49	single	125.125	3500	171.15	171.1486	2.2	3ø14
span	D-B	17.19	250	350	0.03	341.11	single	125.125	3500	144.89	144.8856	1.8	2ø14
support	B	33.17	250	350	0.05	332.39	single	125.125	3500	286.91	286.9062	3.7	4ø14
span	B-A	23.6	250	350	0.04	337.66	single	125.125	3500	200.94	200.9394	2.6	3ø14
support	A	27.81	250	350	0.05	335.36	single	125.125	3500	238.41	238.409	3.0	3ø14

axis 4 2nd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	0.98	250	350	0.00	349.51	single	125.125	3500	8.06	125.125	1.6	2ø14
span	E-D	12.55	250	350	0.02	343.55	single	125.125	3500	105.02	125.125	1.6	2ø14
support	D	17.98	250	350	0.03	340.69	single	125.125	3500	151.73	151.7311	1.9	2ø14
span	D-B	16.63	250	350	0.03	341.40	single	125.125	3500	140.04	140.0436	1.8	2ø14
support	B	31.23	250	350	0.05	333.47	single	125.125	3500	269.25	269.2479	3.4	4ø14
span	B-A	23.49	250	350	0.04	337.72	single	125.125	3500	199.97	199.9674	2.5	3ø14
support	A	26.88	250	350	0.04	335.87	single	125.125	3500	230.09	230.0858	2.9	3ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 4 1st floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.79	250	350	0.00	349.09	single	125.125	3500	14.74	125.125	1.6	2ø14
span	E-D	12.41	250	350	0.02	343.63	single	125.125	3500	103.83	125.125	1.6	2ø14
support	D	15.06	250	350	0.02	342.23	single	125.125	3500	126.51	126.5146	1.6	2ø14
span	D-B	16.88	250	350	0.03	341.27	single	125.125	3500	142.20	142.2042	1.8	2ø14
support	B	22.1	250	350	0.04	338.48	single	125.125	3500	187.72	187.7165	2.4	3ø14
span	B-A	23.11	250	350	0.04	337.93	single	125.125	3500	196.61	196.6126	2.5	3ø14
support	A	30.55	250	350	0.05	333.85	single	125.125	3500	263.09	263.087	3.4	3ø14

axis 4 GR floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	5.23	250	350	0.01	347.34	single	125.125	3500	43.29	125.125	1.6	2ø14
span	E-D	11.91	250	350	0.02	343.89	single	125.125	3500	99.57	125.125	1.6	2ø14
support	D	13.47	250	350	0.02	343.07	single	125.125	3500	112.88	125.125	1.6	2ø14
span	D-B	17.23	250	350	0.03	341.08	single	125.125	3500	145.23	145.2318	1.9	2ø14
support	B	21.95	250	350	0.04	338.56	single	125.125	3500	186.40	186.3978	2.4	3ø14
span	B-A	22.35	250	350	0.04	338.34	single	125.125	3500	189.92	189.9157	2.4	3ø14
support	A	32.32	250	350	0.05	332.86	single	125.125	3500	279.15	279.1544	3.6	4ø14



Bending moment diagram of beam in axis 5

axis 5 Roof slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	6.61	250	350	0.01	346.63	single	125.125	3500	54.82	125.125	1.6	2ø14
span	E-D	6.9	250	350	0.01	346.49	single	125.125	3500	57.25	125.125	1.6	2ø14
support	D	18.68	250	350	0.03	340.31	single	125.125	3500	157.81	157.8112	2.0	2ø14
span	D-B	15.25	250	350	0.02	342.13	single	125.125	3500	128.15	128.1483	1.6	2ø14
support	B	24.16	250	350	0.04	337.36	single	125.125	3500	205.89	205.8928	2.6	3ø14
span	B-A	17.59	250	350	0.03	340.89	single	125.125	3500	148.35	148.3495	1.9	2ø14
support	A	15.95	250	350	0.03	341.76	single	125.125	3500	134.18	134.1757	1.7	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 5 4th floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	6.88	250	350	0.01	346.50	single	125.125	3500	57.09	125.125	1.6	2ø14
span	E-D	10.35	250	350	0.02	344.70	single	125.125	3500	86.32	125.125	1.6	2ø14
support	D	21.66	250	350	0.04	338.71	single	125.125	3500	183.85	183.8502	2.3	3ø14
span	D-B	15.98	250	350	0.03	341.75	single	125.125	3500	134.43	134.4343	1.7	2ø14
support	B	20.19	250	350	0.03	339.50	single	125.125	3500	170.97	170.9738	2.2	3ø14
span	B-A	17.9	250	350	0.03	340.73	single	125.125	3500	151.04	151.0371	1.9	2ø14
support	A	24.86	250	350	0.04	336.98	single	125.125	3500	212.10	212.0978	2.7	3ø14

axis 5 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.77	250	350	0.00	349.11	single	125.125	3500	14.58	125.125	1.6	2ø14
span	E-D	11.27	250	350	0.02	344.22	single	125.125	3500	94.13	125.125	1.6	2ø14
support	D	17.51	250	350	0.03	340.94	single	125.125	3500	147.66	147.6564	1.9	2ø14
span	D-B	15.62	250	350	0.03	341.94	single	125.125	3500	131.33	131.3325	1.7	2ø14
support	B	20.05	250	350	0.03	339.58	single	125.125	3500	169.75	169.7507	2.2	3ø14
span	B-A	18.26	250	350	0.03	340.54	single	125.125	3500	154.16	154.1615	2.0	2ø14
support	A	23.11	250	350	0.04	337.93	single	125.125	3500	196.61	196.6126	2.5	3ø14

axis 5 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	2.51	250	350	0.00	348.73	single	125.125	3500	20.69	125.125	1.6	2ø14
span	E-D	12.77	250	350	0.02	343.44	single	125.125	3500	106.90	125.125	1.6	2ø14
support	D	18.44	250	350	0.03	340.44	single	125.125	3500	155.73	155.725	2.0	2ø14
span	D-B	15.22	250	350	0.02	342.15	single	125.125	3500	127.89	127.8903	1.6	2ø14
support	B	18.18	250	350	0.03	340.58	single	125.125	3500	153.47	153.4668	2.0	3ø14
span	B-A	18.25	250	350	0.03	340.54	single	125.125	3500	154.07	154.0746	2.0	2ø14
support	A	22.01	250	350	0.04	338.52	single	125.125	3500	186.93	186.9252	2.4	3ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 5 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.77	250	350	0.00	349.11	single	125.125	3500	14.58	125.125	1.6	2ø14
span	E-D	11.61	250	350	0.02	344.04	single	125.125	3500	97.02	125.125	1.6	2ø14
support	D	15.68	250	350	0.03	341.91	single	125.125	3500	131.85	131.8493	1.7	2ø14
span	D-B	15.17	250	350	0.02	342.18	single	125.125	3500	127.46	127.4603	1.6	2ø14
support	B	19.53	250	350	0.03	339.86	single	125.125	3500	165.21	165.2127	2.1	3ø14
span	B-A	18.13	250	350	0.03	340.61	single	125.125	3500	153.03	153.0328	1.9	2ø14
support	A	22.01	250	350	0.04	338.52	single	125.125	3500	186.93	186.9252	2.4	3ø14

axis 5 GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.24	250	350	0.01	347.85	single	125.125	3500	35.04	125.125	1.6	2ø14
span	E-D	11.05	250	350	0.02	344.34	single	125.125	3500	92.26	125.125	1.6	2ø14
support	D	11.98	250	350	0.02	343.85	single	125.125	3500	100.17	125.125	1.6	2ø14
span	D-B	15.49	250	350	0.03	342.01	single	125.125	3500	130.21	130.2133	1.7	2ø14
support	B	21.1	250	350	0.03	339.01	single	125.125	3500	178.94	178.9376	2.3	3ø14
span	B-A	17.27	250	350	0.03	341.06	single	125.125	3500	145.58	145.578	1.9	2ø14
support	A	23.06	250	350	0.04	337.96	single	125.125	3500	196.17	196.1715	2.5	3ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 6 4th floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	14.7	250	350	0.02	342.42	single	125.125	3500	123.42	125.125	1.6	2ø14
span	E-D	7.07	250	350	0.01	346.40	single	125.125	3500	58.68	125.125	1.6	2ø14
support	D	18.25	250	350	0.03	340.54	single	125.125	3500	154.07	154.0746	2.0	2ø14
span	D-B	16.35	250	350	0.03	341.55	single	125.125	3500	137.63	137.6259	1.8	2ø14
support	B	15.8	250	350	0.03	341.84	single	125.125	3500	132.88	132.883	1.7	2ø14

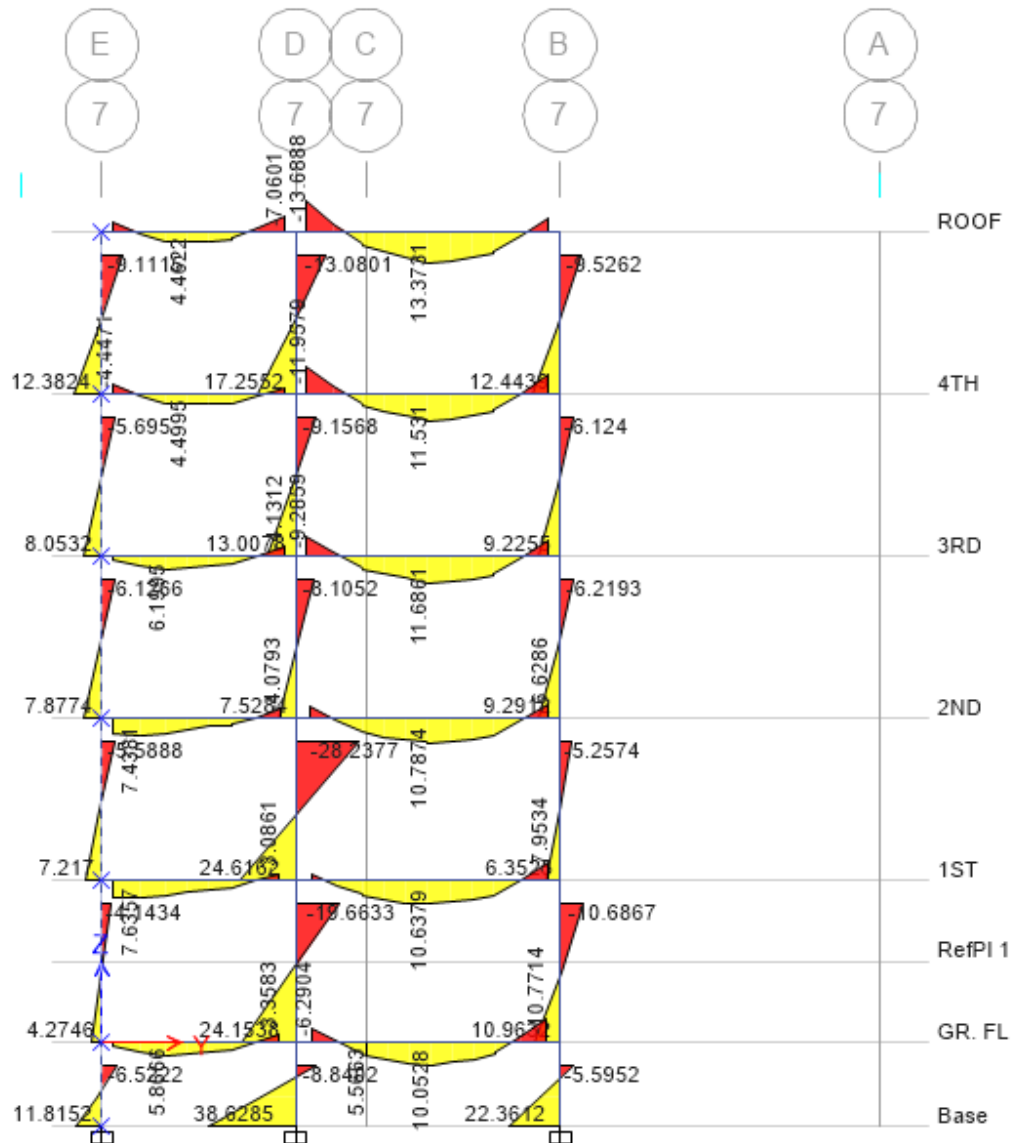
axis 6 3rd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	0.0129	250	350	0.00	349.99	single	125.125	3500	0.11	125.125	1.6	2ø14
span	E-D	8.31	250	350	0.01	345.76	single	125.125	3500	69.10	125.125	1.6	2ø14
support	D	17.17	250	350	0.03	341.12	single	125.125	3500	144.71	144.7125	1.8	2ø14
span	D-B	16.4	250	350	0.03	341.52	single	125.125	3500	138.06	138.0575	1.8	2ø14
support	B	17.17	250	350	0.03	341.12	single	125.125	3500	144.71	144.7125	1.8	2ø14

axis 6 2nd floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.93	250	350	0.01	347.50	single	125.125	3500	40.79	125.125	1.6	2ø14
span	E-D	8.66	250	350	0.01	345.58	single	125.125	3500	72.05	125.125	1.6	2ø14
support	D	14.75	250	350	0.02	342.40	single	125.125	3500	123.85	125.125	1.6	2ø14
span	D-B	15.5	250	350	0.03	342.00	single	125.125	3500	130.30	130.2994	1.7	2ø14
support	B	14.75	250	350	0.02	342.40	single	125.125	3500	123.85	125.125	1.6	2ø14

axis 6 1st floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.25	250	350	0.01	347.84	single	125.125	3500	35.13	125.125	1.6	2ø14
span	E-D	8.63	250	350	0.01	345.59	single	125.125	3500	71.79	125.125	1.6	2ø14
support	D	10.73	250	350	0.02	344.50	single	125.125	3500	89.55	125.125	1.6	2ø14
span	D-B	15.42	250	350	0.03	342.04	single	125.125	3500	129.61	129.6109	1.7	2ø14
support	B	13.87	250	350	0.02	342.86	single	125.125	3500	116.30	125.125	1.6	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 6 GR floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.51	250	350	0.00	349.24	single	125.125	3500	12.43	125.125	1.6	2ø14
span	E-D	8.04	250	350	0.01	345.90	single	125.125	3500	66.83	125.125	1.6	2ø14
support	D	8.07	250	350	0.01	345.88	single	125.125	3500	67.08	125.125	1.6	2ø14
span	D-B	15.74	250	350	0.03	341.87	single	125.125	3500	132.37	132.3661	1.7	2ø14
support	B	18.36	250	350	0.03	340.48	single	125.125	3500	155.03	155.03	2.0	2ø14



Bending moment diagram of beam in axis 7

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 7 Roof Slab													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.31	250	350	0.01	347.81	single	125.125	3500	35.63	125.125	1.6	2ø14
span	E-D	4.46	250	350	0.01	347.74	single	125.125	3500	36.87	125.125	1.6	2ø14
support	D	13.69	250	350	0.02	342.95	single	125.125	3500	114.76	125.125	1.6	2ø14
span	D-B	13.37	250	350	0.02	343.12	single	125.125	3500	112.03	125.125	1.6	2ø14
support	B	6.63	250	350	0.01	346.62	single	125.125	3500	54.99	125.125	1.6	2ø14

axis 7 4th Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	4.45	250	350	0.01	347.74	single	125.125	3500	36.79	125.125	1.6	2ø14
span	E-D	4.5	250	350	0.01	347.72	single	125.125	3500	37.21	125.125	1.6	2ø14
support	D	17.25	250	350	0.03	341.07	single	125.125	3500	145.40	145.4049	1.9	2ø14
span	D-B	11.53	250	350	0.02	344.09	single	125.125	3500	96.34	125.125	1.6	2ø14
support	B	8.67	250	350	0.01	345.57	single	125.125	3500	72.13	125.125	1.6	2ø14

axis 7 3rd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	1.86	250	350	0.00	349.06	single	125.125	3500	15.32	125.125	1.6	2ø14
span	E-D	6.2	250	350	0.01	346.84	single	125.125	3500	51.39	125.125	1.6	2ø14
support	D	13.01	250	350	0.02	343.31	single	125.125	3500	108.95	125.125	1.6	2ø14
span	D-B	11.69	250	350	0.02	344.00	single	125.125	3500	97.70	125.125	1.6	2ø14
support	B	6.88	250	350	0.01	346.50	single	125.125	3500	57.09	125.125	1.6	2ø14

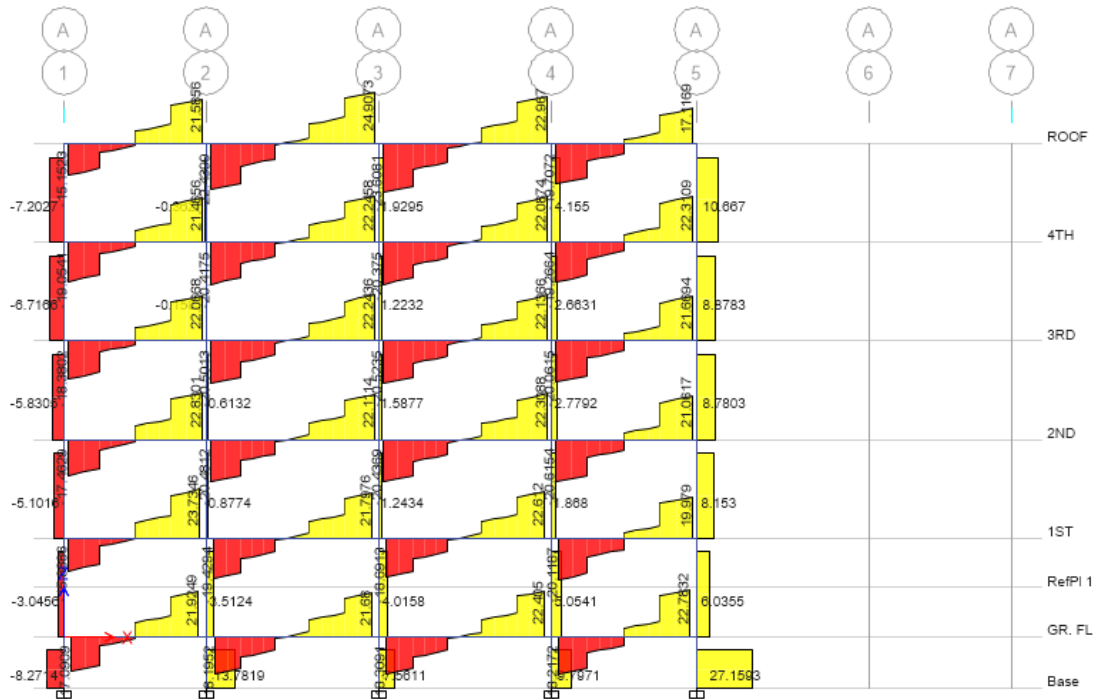
axis 7 2nd Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	7.02	250	350	0.01	346.42	single	125.125	3500	58.26	125.125	1.6	2ø14
span	E-D	7.44	250	350	0.01	346.21	single	125.125	3500	61.78	125.125	1.6	2ø14
support	D	3.09	250	350	0.01	348.43	single	125.125	3500	25.50	125.125	1.6	2ø14
span	D-B	10.79	250	350	0.02	344.47	single	125.125	3500	90.05	125.125	1.6	2ø14
support	B	7.93	250	350	0.01	345.95	single	125.125	3500	65.90	125.125	1.6	2ø14

Structural design of G+4 Apartment + shop building B.Sc Thesis project

axis 7 1st Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	7.42	250	350	0.01	346.22	single	125.125	3500	61.62	125.125	1.6	2ø14
span	E-D	7.64	250	350	0.01	346.10	single	125.125	3500	63.46	125.125	1.6	2ø14
support	D	3.87	250	350	0.01	348.04	single	125.125	3500	31.97	125.125	1.6	2ø14
span	D-B	10.64	250	350	0.02	344.55	single	125.125	3500	88.78	125.125	1.6	2ø14
support	B	7.95	250	350	0.01	345.94	single	125.125	3500	66.07	125.125	1.6	2ø14

axis 7 GR Floor													
Type	Loc	Moment (KNm)	b(mm)	d(mm)	K	Z(mm)	Beam type	As,min	As,max	As,cal	As,prov	No of	Remark
support	E	2.004	250	350	0.00	348.99	single	125.125	3500	16.51	125.125	1.6	2ø14
span	E-D	5.87	250	350	0.01	347.01	single	125.125	3500	48.63	125.125	1.6	2ø14
support	D	3.36	250	350	0.01	348.30	single	125.125	3500	27.73	125.125	1.6	2ø14
span	D-B	10.05	250	350	0.02	344.86	single	125.125	3500	83.78	125.125	1.6	2ø14
support	B	10.77	250	350	0.02	344.48	single	125.125	3500	89.88	125.125	1.6	2ø14

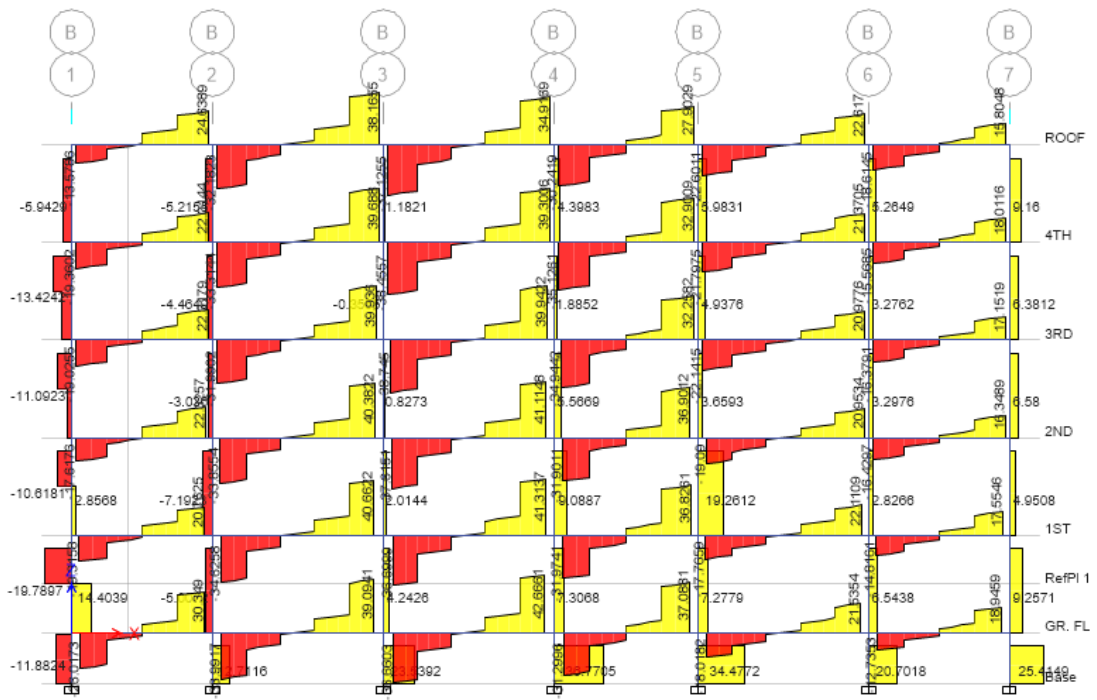
D-2 shear reinforcement calculation



Axis -A shear result

Structural design of G+4 Apartment + shop building B.Sc Thesis project

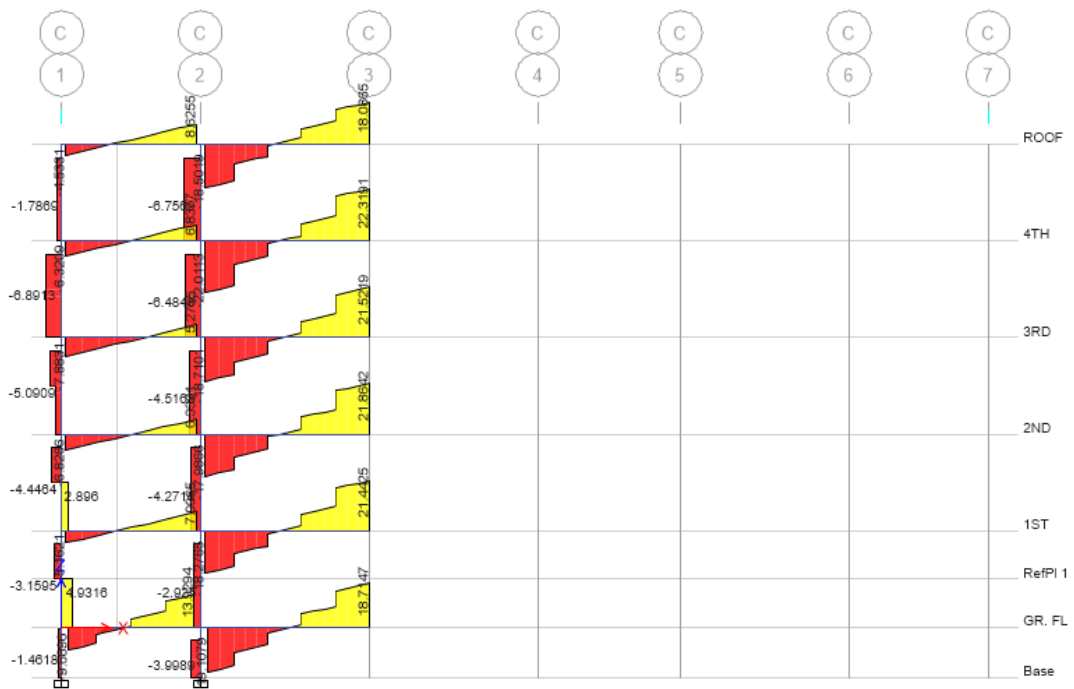
floor level	Axis - A						Remark
	span	Ved	Sbmax	Scal	Smin	Spro	
roof slab	1-2	17.15	262.5	802.4	224.6801	262.5	ø8 c/c 260
	2-3	20.21	262.5	680.9	224.6801	262.5	ø8 c/c 260
	3-4	18.99	262.5	724.7	224.6801	262.5	ø8 c/c 260
	4-5	15.38	262.5	894.8	224.6801	262.5	ø8 c/c 260
4th floor	1-2	16.59	262.5	829.5	224.6801	262.5	ø8 c/c 260
	2-3	18.02	262.5	763.7	224.6801	262.5	ø8 c/c 260
	3-4	17.88	262.5	769.7	224.6801	262.5	ø8 c/c 260
	4-5	17.43	262.5	789.5	224.6801	262.5	ø8 c/c 260
3rd floor	1-2	17.2	262.5	800.1	224.6801	262.5	ø8 c/c 260
	2-3	18.02	262.5	763.7	224.6801	262.5	ø8 c/c 260
	3-4	17.91	262.5	768.4	224.6801	262.5	ø8 c/c 260
	4-5	16.77	262.5	820.6	224.6801	262.5	ø8 c/c 260
2nd floor	1-2	17.98	262.5	765.4	224.6801	262.5	ø8 c/c 260
	2-3	17.89	262.5	769.2	224.6801	262.5	ø8 c/c 260
	3-4	18.07	262.5	761.6	224.6801	262.5	ø8 c/c 260
	4-5	16.23	262.5	847.9	224.6801	262.5	ø8 c/c 260
1st floor	1-2	18.96	262.5	725.8	224.6801	262.5	ø8 c/c 260
	2-3	17.71	262.5	777.0	224.6801	262.5	ø8 c/c 260
	3-4	18.51	262.5	743.5	224.6801	262.5	ø8 c/c 260
	4-5	15.4	262.5	893.6	224.6801	262.5	ø8 c/c 260
GR floor	1-2	17.22	262.5	799.2	224.6801	262.5	ø8 c/c 260
	2-3	17.72	262.5	776.6	224.6801	262.5	ø8 c/c 260
	3-4	18.37	262.5	749.1	224.6801	262.5	ø8 c/c 260
	4-5	17.96	262.5	766.2	224.6801	262.5	ø8 c/c 260



Axis -B shear result

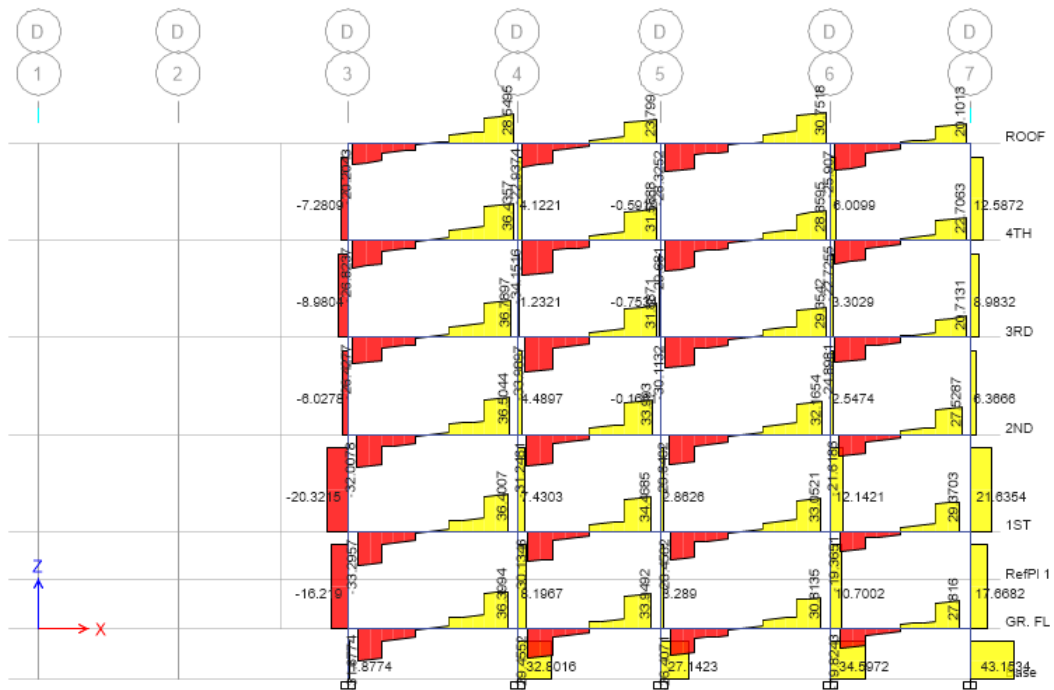
Structural design of G+4 Apartment + shop building B.Sc Thesis project

floor level	Axis - B						Remark
	span	Ved	Sbmax	Scal	Smin	Spro	
roof slab	1-2	20.04	262.5	686.7	224.6801	262.5	ø8 c/c 260
	2-3	31.19	262.5	441.2	224.6801	262.5	ø8 c/c 260
	3-4	29.98	262.5	459.0	224.6801	262.5	ø8 c/c 260
	4-5	23.4	262.5	588.1	224.6801	262.5	ø8 c/c 260
	5-6	18.13	262.5	759.0	224.6801	262.5	ø8 c/c 260
	6-7	14.46	262.5	951.7	224.6801	262.5	ø8 c/c 260
4th floor	1-2	17.03	262.5	808.1	224.6801	262.5	ø8 c/c 260
	2-3	32.45	262.5	424.1	224.6801	262.5	ø8 c/c 260
	3-4	31.59	262.5	435.6	224.6801	262.5	ø8 c/c 260
	4-5	27.13	262.5	507.2	224.6801	262.5	ø8 c/c 260
	5-6	17.52	262.5	785.5	224.6801	262.5	ø8 c/c 260
	6-7	13.96	262.5	985.8	224.6801	262.5	ø8 c/c 260
3rd floor	1-2	17.37	262.5	792.3	224.6801	262.5	ø8 c/c 260
	2-3	32.8	262.5	419.6	224.6801	262.5	ø8 c/c 260
	3-4	32.93	262.5	417.9	224.6801	262.5	ø8 c/c 260
	4-5	27.03	262.5	509.1	224.6801	262.5	ø8 c/c 260
	5-6	17.86	262.5	770.5	224.6801	262.5	ø8 c/c 260
	6-7	13.11	262.5	1049.7	224.6801	262.5	ø8 c/c 260
2nd floor	1-2	17.52	262.5	785.5	224.6801	262.5	ø8 c/c 260
	2-3	33.01	262.5	416.9	224.6801	262.5	ø8 c/c 260
	3-4	33.3	262.5	413.3	224.6801	262.5	ø8 c/c 260
	4-5	28.81	262.5	477.7	224.6801	262.5	ø8 c/c 260
	5-6	16.98	262.5	810.4	224.6801	262.5	ø8 c/c 260
	6-7	12.48	262.5	1102.7	224.6801	262.5	ø8 c/c 260
1st floor	1-2	15.93	262.5	863.9	224.6801	262.5	ø8 c/c 260
	2-3	33.19	262.5	414.6	224.6801	262.5	ø8 c/c 260
	3-4	33.57	262.5	409.9	224.6801	262.5	ø8 c/c 260
	4-5	28.74	262.5	478.8	224.6801	262.5	ø8 c/c 260
	5-6	18.15	262.5	758.2	224.6801	262.5	ø8 c/c 260
	6-7	13.75	262.5	1000.8	224.6801	262.5	ø8 c/c 260
GR floor	1-2	23.56	262.5	584.1	224.6801	262.5	ø8 c/c 260
	2-3	31.84	262.5	432.2	224.6801	262.5	ø8 c/c 260
	3-4	34.84	262.5	395.0	224.6801	262.5	ø8 c/c 260
	4-5	29.04	262.5	473.9	224.6801	262.5	ø8 c/c 260
	5-6	17.62	262.5	781.0	224.6801	262.5	ø8 c/c 260
	6-7	15.13	262.5	909.5	224.6801	262.5	ø8 c/c 260



Axis -C shear result

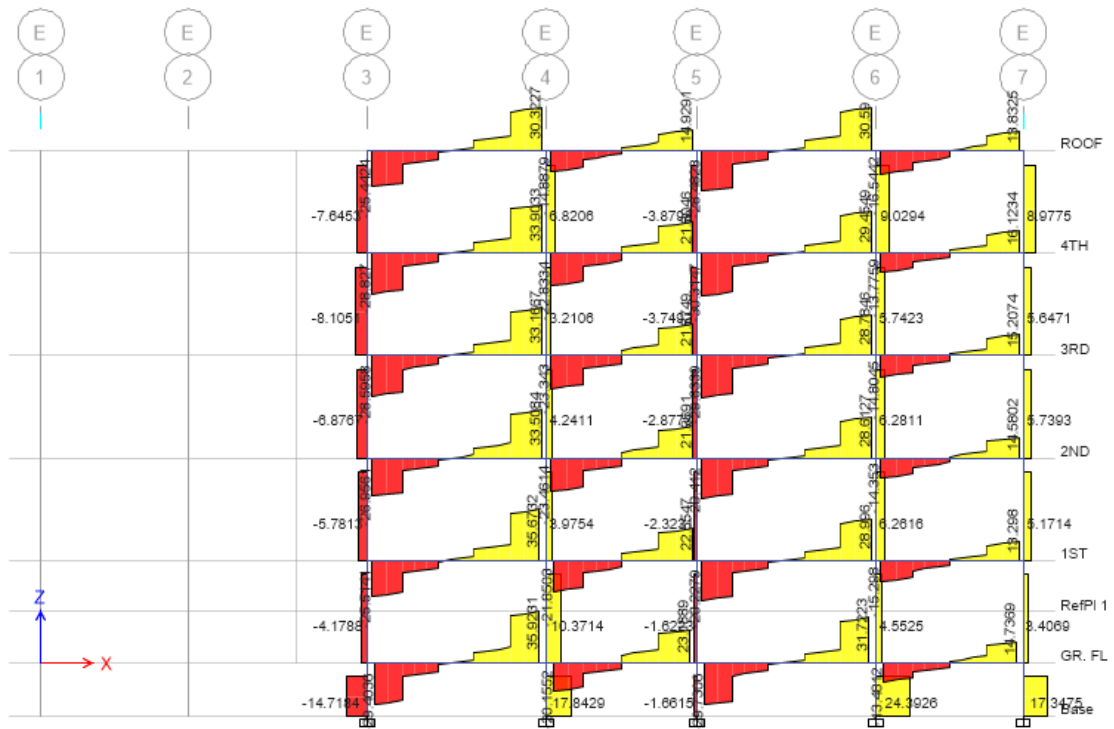
floor level	Axis - C						Remark
	span	Ved	Sbmax	Scal	Smin	Spro	
roof slab	1-2	7.04	262.5	1954.7	224.6801	262.5	ø8 c/c 260
	2-3	14.87	262.5	925.4	224.6801	262.5	ø8 c/c 260
4th floor	1-2	5.25	262.5	2621.2	224.6801	262.5	ø8 c/c 260
	2-3	17.92	262.5	767.9	224.6801	262.5	ø8 c/c 260
3rd floor	1-2	6.29	262.5	2187.8	224.6801	262.5	ø8 c/c 260
	2-3	17.53	262.5	785.0	224.6801	262.5	ø8 c/c 260
2nd floor	1-2	5.24	262.5	2626.2	224.6801	262.5	ø8 c/c 260
	2-3	17.91	262.5	768.4	224.6801	262.5	ø8 c/c 260
1st floor	1-2	6.41	262.5	2146.9	224.6801	262.5	ø8 c/c 260
	2-3	17.5	262.5	786.4	224.6801	262.5	ø8 c/c 260
GR floor	1-2	11.08	262.5	1242.0	224.6801	262.5	ø8 c/c 260
	2-3	15.36	262.5	895.9	224.6801	262.5	ø8 c/c 260



Axis -D shear result

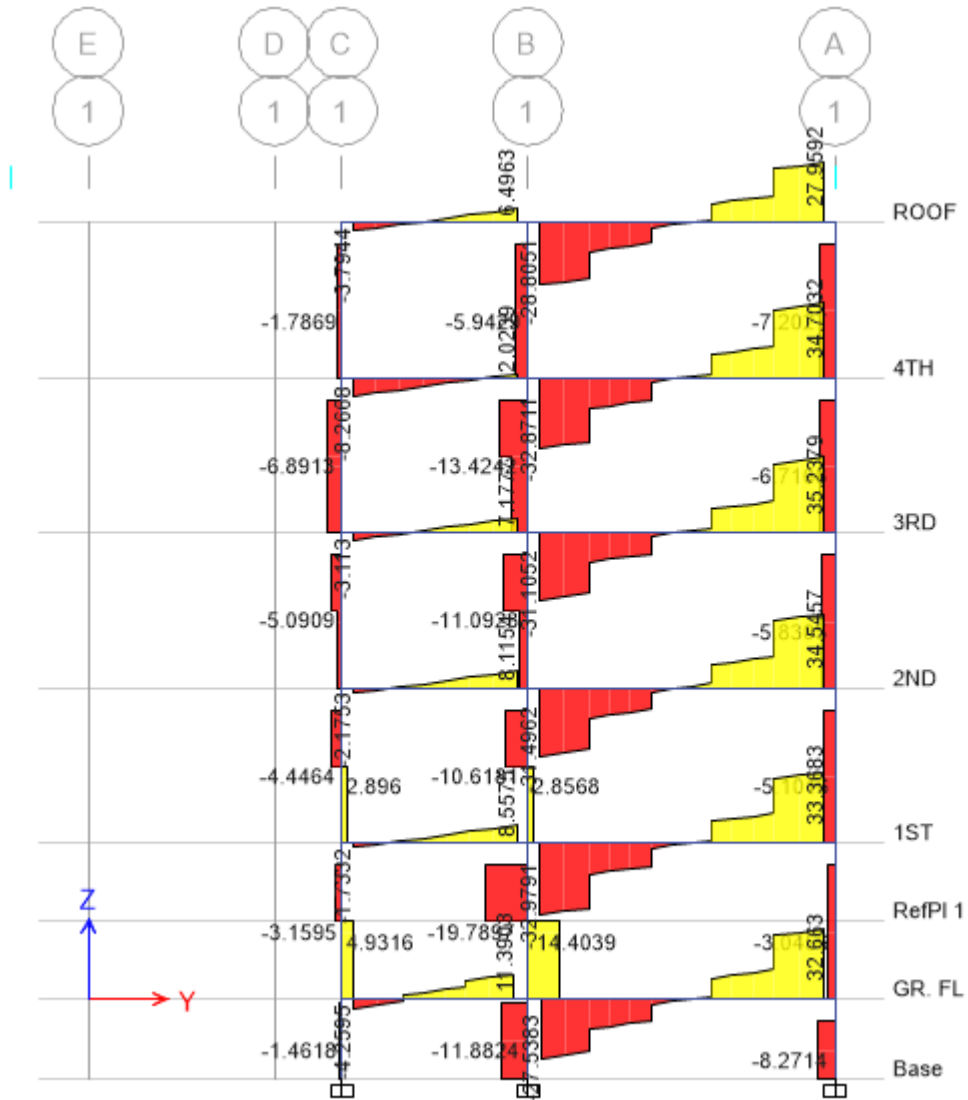
Structural design of G+4 Apartment + shop building B.Sc Thesis project

Axis - D							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	3-4	23.71	262.5	580.4	224.6801	262.5	ø8 c/c 260
	4-5	18.3	262.5	752.0	224.6801	262.5	ø8 c/c 260
	5-6	24.89	262.5	552.9	224.6801	262.5	ø8 c/c 260
	6-7	20.37	262.5	675.6	224.6801	262.5	ø8 c/c 260
4th floor	3-4	30.16	262.5	456.3	224.6801	262.5	ø8 c/c 260
	4-5	26.42	262.5	520.9	224.6801	262.5	ø8 c/c 260
	5-6	23.87	262.5	576.5	224.6801	262.5	ø8 c/c 260
	6-7	17.25	262.5	797.8	224.6801	262.5	ø8 c/c 260
3rd floor	3-4	30.52	262.5	450.9	224.6801	262.5	ø8 c/c 260
	4-5	26.17	262.5	525.8	224.6801	262.5	ø8 c/c 260
	5-6	26.04	262.5	528.5	224.6801	262.5	ø8 c/c 260
	6-7	19.48	262.5	706.4	224.6801	262.5	ø8 c/c 260
2nd floor	3-4	29.7	262.5	463.3	224.6801	262.5	ø8 c/c 260
	4-5	26.31	262.5	523.0	224.6801	262.5	ø8 c/c 260
	5-6	32.93	262.5	417.9	224.6801	262.5	ø8 c/c 260
	6-7	21.61	262.5	636.8	224.6801	262.5	ø8 c/c 260
1st floor	3-4	29.49	262.5	466.6	224.6801	262.5	ø8 c/c 260
	4-5	26.87	262.5	512.1	224.6801	262.5	ø8 c/c 260
	5-6	26.95	262.5	510.6	224.6801	262.5	ø8 c/c 260
	6-7	23.5	262.5	585.6	224.6801	262.5	ø8 c/c 260
GR floor	3-4	29.65	262.5	464.1	224.6801	262.5	ø8 c/c 260
	4-5	26.49	262.5	519.5	224.6801	262.5	ø8 c/c 260
	5-6	25.13	262.5	547.6	224.6801	262.5	ø8 c/c 260
	6-7	22.08	262.5	623.3	224.6801	262.5	ø8 c/c 260



Axis -E shear result

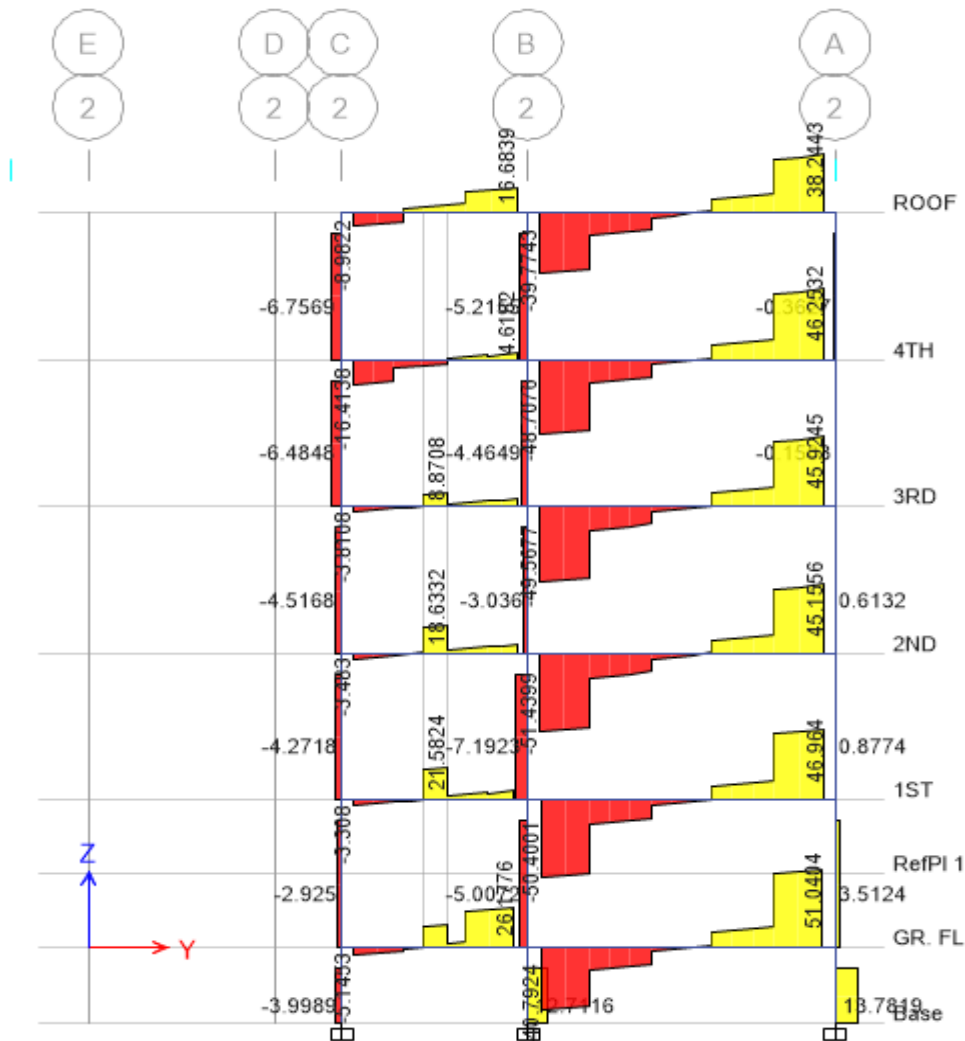
Axis - E							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	3-4	24.79	262.5	555.1	224.6801	262.5	ø8 c/c 260
	4-5	11.53	262.5	1193.5	224.6801	262.5	ø8 c/c 260
	5-6	24.73	262.5	556.5	224.6801	262.5	ø8 c/c 260
	6-7	12.88	262.5	1068.4	224.6801	262.5	ø8 c/c 260
4th floor	3-4	27.68	262.5	497.2	224.6801	262.5	ø8 c/c 260
	4-5	17.58	262.5	782.8	224.6801	262.5	ø8 c/c 260
	5-6	24.38	262.5	564.5	224.6801	262.5	ø8 c/c 260
	6-7	12.52	262.5	1099.2	224.6801	262.5	ø8 c/c 260
3rd floor	3-4	27.04	262.5	508.9	224.6801	262.5	ø8 c/c 260
	4-5	18.02	262.5	763.7	224.6801	262.5	ø8 c/c 260
	5-6	23.84	262.5	577.2	224.6801	262.5	ø8 c/c 260
	6-7	11.59	262.5	1187.4	224.6801	262.5	ø8 c/c 260
2nd floor	3-4	27.51	262.5	500.2	224.6801	262.5	ø8 c/c 260
	4-5	18.11	262.5	759.9	224.6801	262.5	ø8 c/c 260
	5-6	22.95	262.5	599.6	224.6801	262.5	ø8 c/c 260
	6-7	11.09	262.5	1240.9	224.6801	262.5	ø8 c/c 260
1st floor	3-4	29.6	262.5	464.9	224.6801	262.5	ø8 c/c 260
	4-5	17.33	262.5	794.1	224.6801	262.5	ø8 c/c 260
	5-6	23.32	262.5	590.1	224.6801	262.5	ø8 c/c 260
	6-7	11.85	262.5	1161.3	224.6801	262.5	ø8 c/c 260
GR floor	3-4	29.44	262.5	467.4	224.6801	262.5	ø8 c/c 260
	4-5	18.09	262.5	760.7	224.6801	262.5	ø8 c/c 260
	5-6	25.62	262.5	537.1	224.6801	262.5	ø8 c/c 260
	6-7	11.34	262.5	1213.5	224.6801	262.5	ø8 c/c 260



Shear force diagram of beam in axis 1

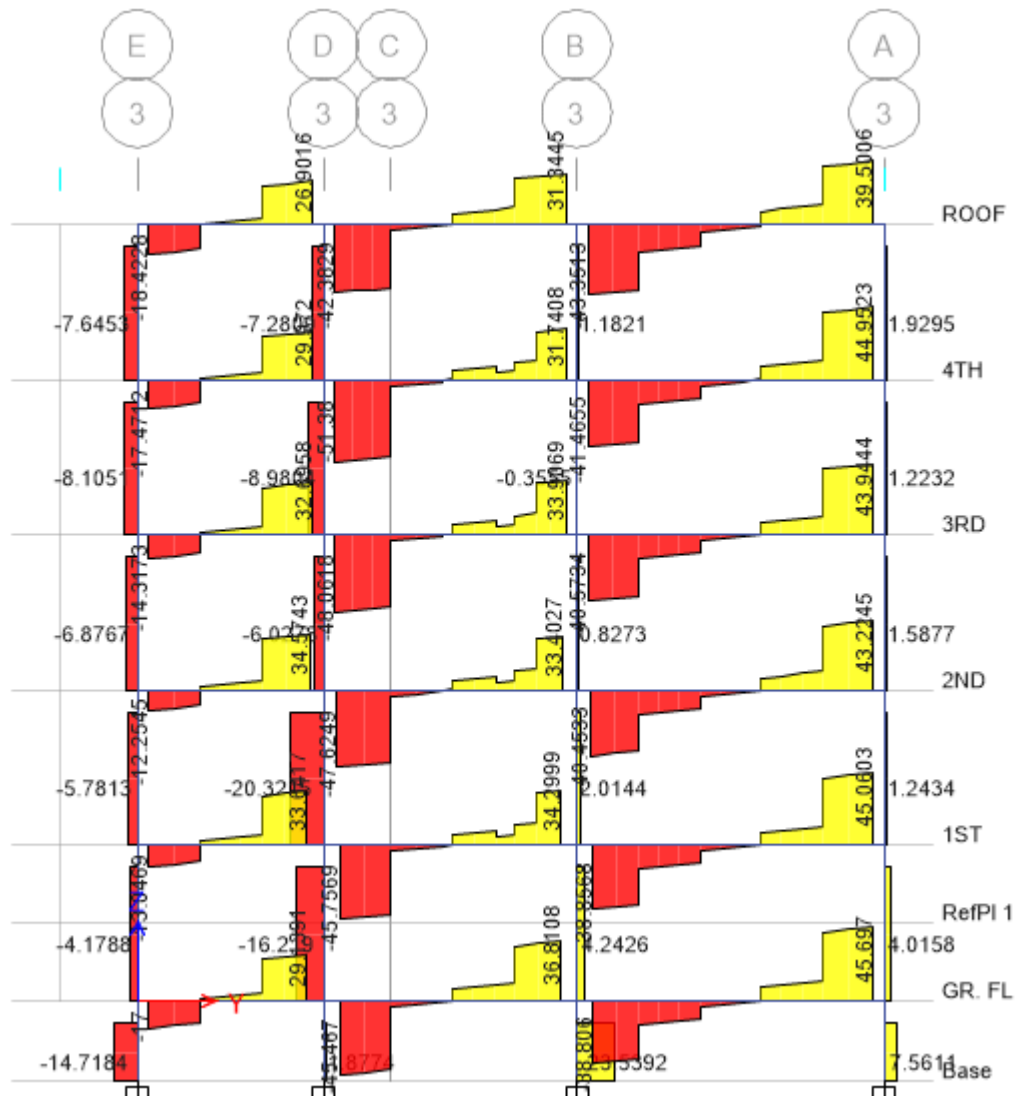
Structural design of G+4 Apartment + shop building B.Sc Thesis project

Axis - 1							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	C-B	7.19	262.5	1914.0	187.2334	262.5	ø8 c/c 200
	B-A	25.05	262.5	549.4	187.2334	262.5	ø8 c/c 200
4th floor	C-B	3.22	262.5	4273.7	187.2334	262.5	ø8 c/c 200
	B-A	36.29	262.5	379.2	187.2334	262.5	ø8 c/c 200
3th floor	C-B	7.57	262.5	1817.9	187.2334	262.5	ø8 c/c 200
	B-A	37.28	262.5	369.1	187.2334	262.5	ø8 c/c 200
2th floor	C-B	7.93	262.5	1735.4	187.2334	262.5	ø8 c/c 200
	B-A	36.21	262.5	380.0	187.2334	262.5	ø8 c/c 200
1th floor	C-B	8.04	262.5	1711.6	187.2334	262.5	ø8 c/c 200
	B-A	34.59	262.5	397.8	187.2334	262.5	ø8 c/c 200
GR floor	C-B	10.45	262.5	1316.9	187.2334	262.5	ø8 c/c 200
	B-A	33.79	262.5	407.3	187.2334	262.5	ø8 c/c 200



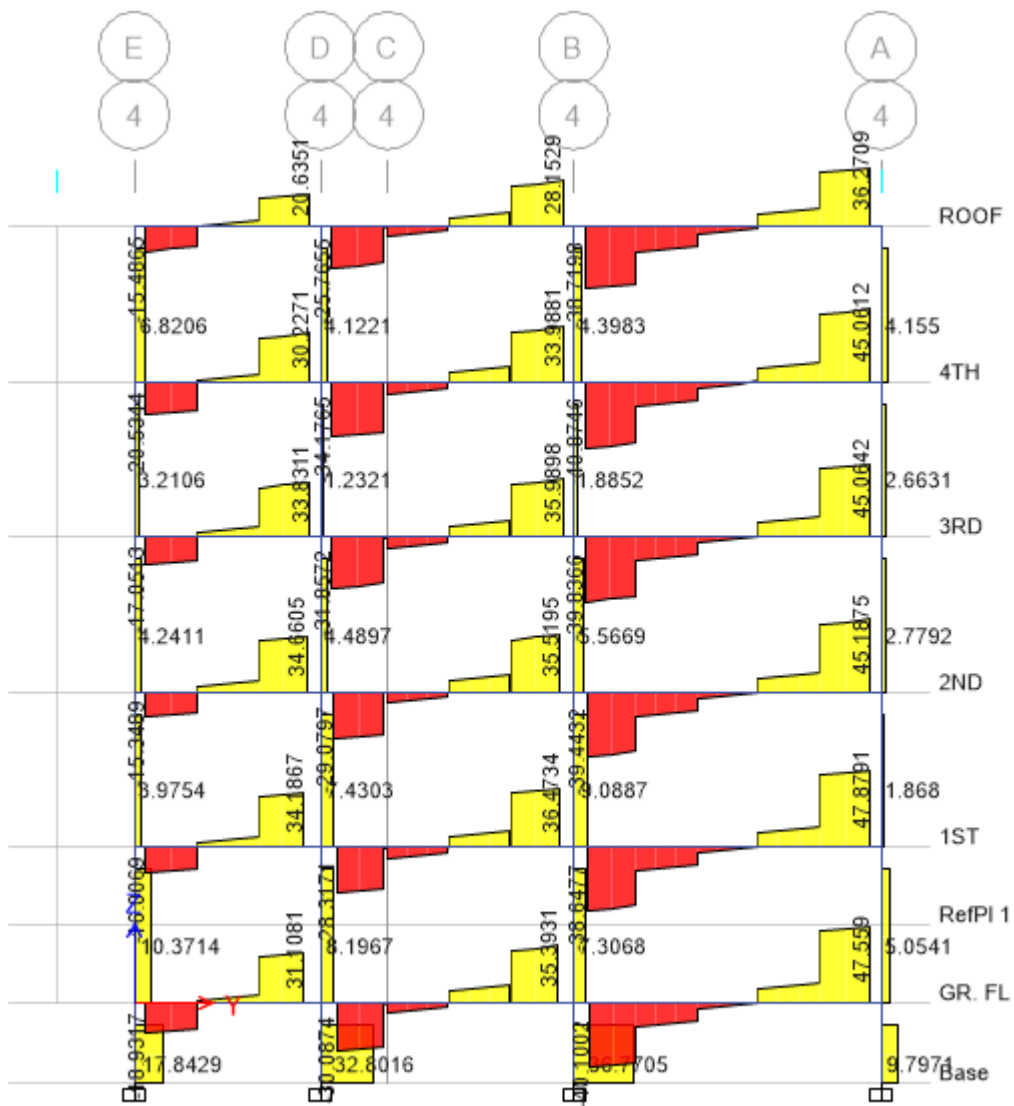
Shear force diagram of beam in axis 2

Axis - 2							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	C-B	13.83	262.5	995.0	187.2334	262.5	ø8 c/c 200
	B-A	32.55	262.5	422.8	187.2334	262.5	ø8 c/c 200
4th floor	C-B	14.07	262.5	978.1	187.2334	262.5	ø8 c/c 200
	B-A	39.91	262.5	344.8	187.2334	262.5	ø8 c/c 200
3th floor	C-B	4.64	262.5	2965.8	187.2334	262.5	ø8 c/c 200
	B-A	40.73	262.5	337.9	187.2334	262.5	ø8 c/c 200
2th floor	C-B	5.13	262.5	2682.5	187.2334	262.5	ø8 c/c 200
	B-A	42.61	262.5	323.0	187.2334	262.5	ø8 c/c 200
1th floor	C-B	5.39	262.5	2553.1	187.2334	262.5	ø8 c/c 200
	B-A	41.39	262.5	332.5	187.2334	262.5	ø8 c/c 200
GR floor	C-B	22.7	262.5	606.2	187.2334	262.5	ø8 c/c 200
	B-A	42.54	262.5	323.5	187.2334	262.5	ø8 c/c 200



Shear force diagram of beam in axis 3

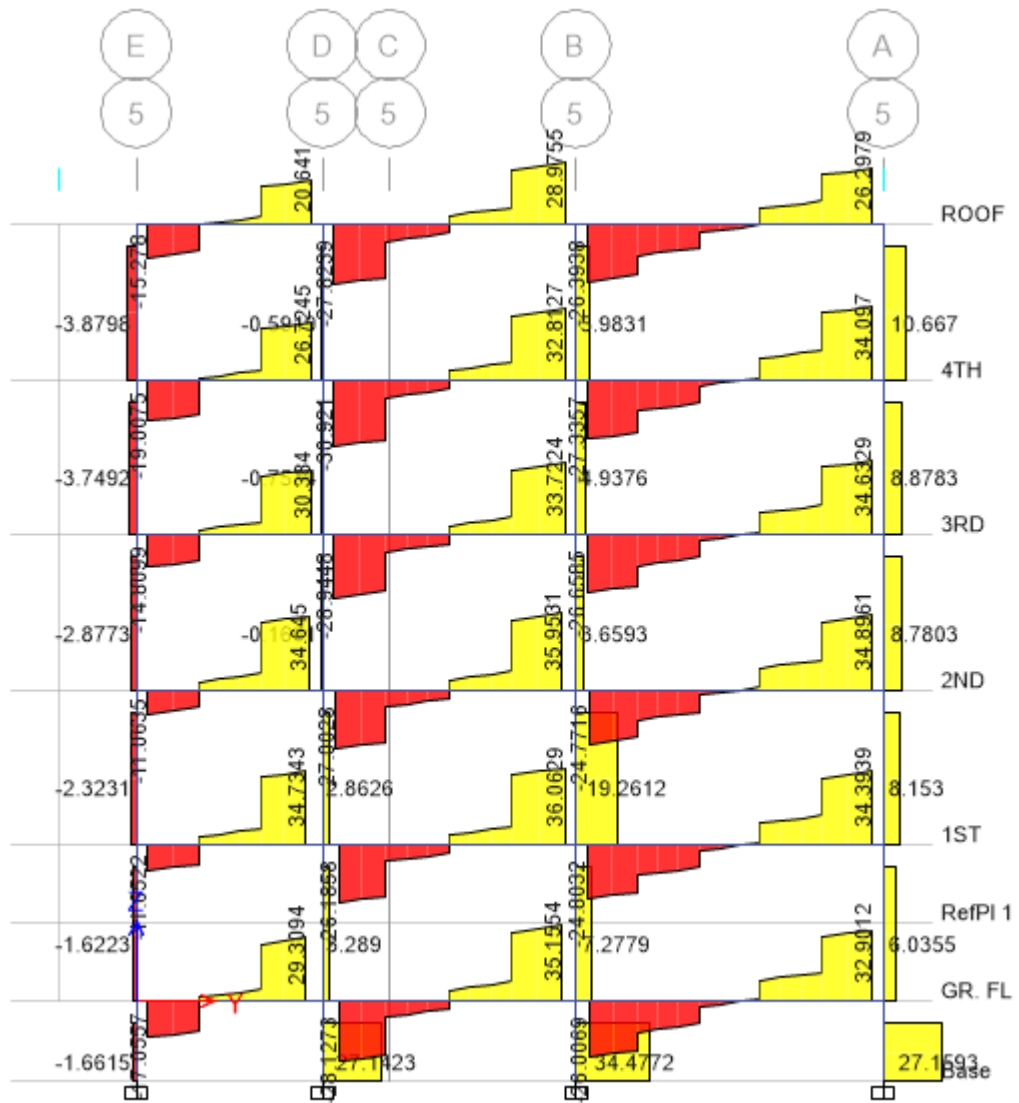
		Axis - 3					
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	E-D	22.33	262.5	616.3	187.2334	262.5	ø8 c/c 200
	D-B	34.19	262.5	402.5	187.2334	262.5	ø8 c/c 200
	B-A	35.68	262.5	385.7	187.2334	262.5	ø8 c/c 200
4th floor	E-D	23.09	262.5	596.0	187.2334	262.5	ø8 c/c 200
	D-B	42.14	262.5	326.6	187.2334	262.5	ø8 c/c 200
	B-A	36.95	262.5	372.4	187.2334	262.5	ø8 c/c 200
3rd floor	E-D	25.89	262.5	531.5	187.2334	262.5	ø8 c/c 200
	D-B	38.97	262.5	353.1	187.2334	262.5	ø8 c/c 200
	B-A	36.12	262.5	381.0	187.2334	262.5	ø8 c/c 200
2nd floor	E-D	27.78	262.5	495.4	187.2334	262.5	ø8 c/c 200
	D-B	39.01	262.5	352.8	187.2334	262.5	ø8 c/c 200
	B-A	36.12	262.5	381.0	187.2334	262.5	ø8 c/c 200
1st floor	E-D	26.87	262.5	512.1	187.2334	262.5	ø8 c/c 200
	D-B	36.86	262.5	373.3	187.2334	262.5	ø8 c/c 200
	B-A	37.29	262.5	369.0	187.2334	262.5	ø8 c/c 200
GR floor	E-D	22.45	262.5	613.0	187.2334	262.5	ø8 c/c 200
	D-B	36.33	262.5	378.8	187.2334	262.5	ø8 c/c 200
	B-A	37.88	262.5	363.3	187.2334	262.5	ø8 c/c 200



Shear force diagram of beam in axis 4

Structural design of G+4 Apartment + shop building B.Sc Thesis project

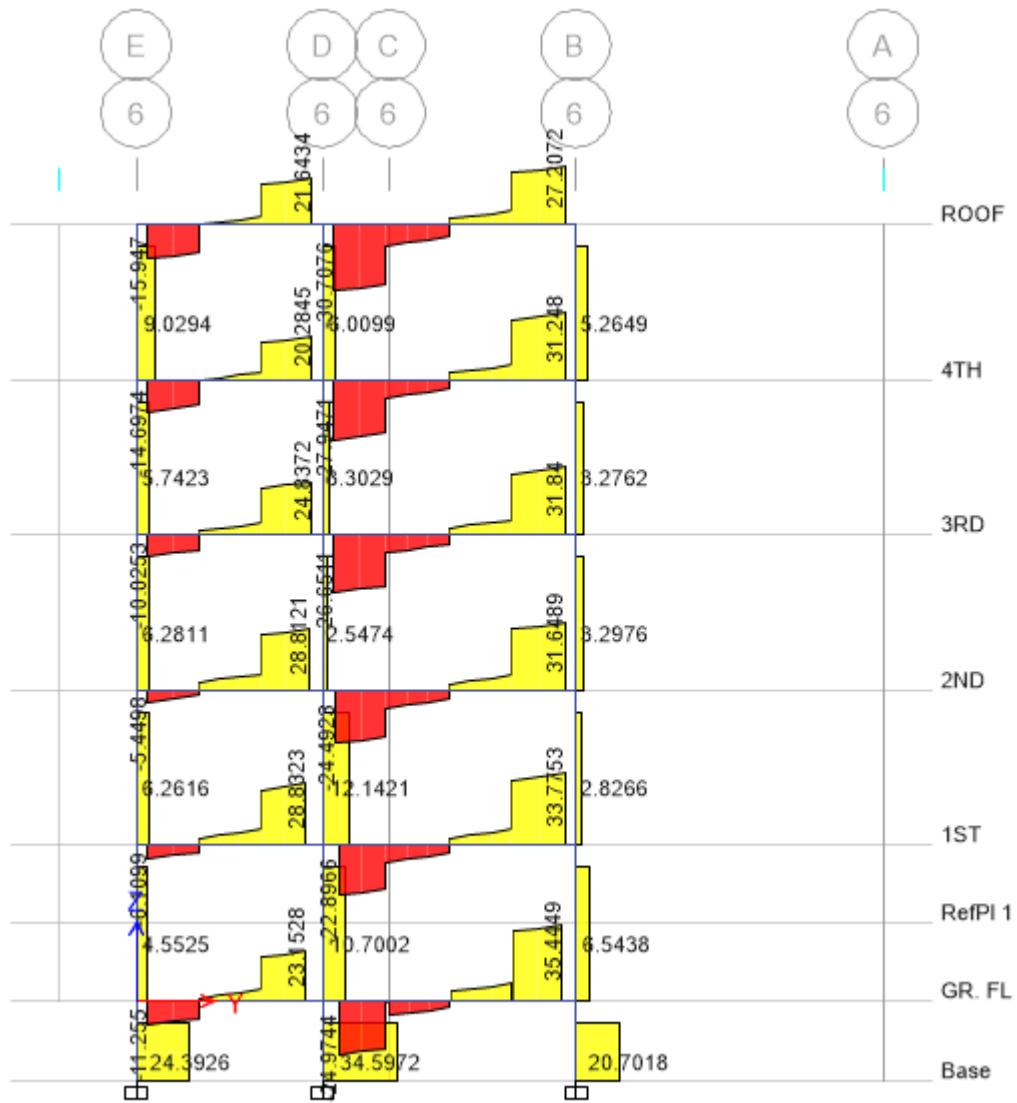
Axis - 4							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	E-D	15.4	262.5	893.6	187.2334	262.5	ø8 c/c 200
	D-B	22.16	262.5	621.0	187.2334	262.5	ø8 c/c 200
	B-A	31.78	262.5	433.0	187.2334	262.5	ø8 c/c 200
4th floor	E-D	22.87	262.5	601.7	187.2334	262.5	ø8 c/c 200
	D-B	26.6	262.5	517.3	187.2334	262.5	ø8 c/c 200
	B-A	37.1	262.5	370.9	187.2334	262.5	ø8 c/c 200
3rd floor	E-D	26.46	262.5	520.1	187.2334	262.5	ø8 c/c 200
	D-B	28.44	262.5	483.9	187.2334	262.5	ø8 c/c 200
	B-A	37.2	262.5	369.9	187.2334	262.5	ø8 c/c 200
2nd floor	E-D	27.42	262.5	501.9	187.2334	262.5	ø8 c/c 200
	D-B	29.34	262.5	469.0	187.2334	262.5	ø8 c/c 200
	B-A	36.12	262.5	381.0	187.2334	262.5	ø8 c/c 200
1st floor	E-D	29.91	262.5	460.1	187.2334	262.5	ø8 c/c 200
	D-B	30.16	262.5	456.3	187.2334	262.5	ø8 c/c 200
	B-A	39.87	262.5	345.2	187.2334	262.5	ø8 c/c 200
GR floor	E-D	23.85	262.5	577.0	187.2334	262.5	ø8 c/c 200
	D-B	28.12	262.5	489.4	187.2334	262.5	ø8 c/c 200
	B-A	39.44	262.5	348.9	187.2334	262.5	ø8 c/c 200



Shear force diagram of beam in axis 5

Structural design of G+4 Apartment + shop building B.Sc Thesis project

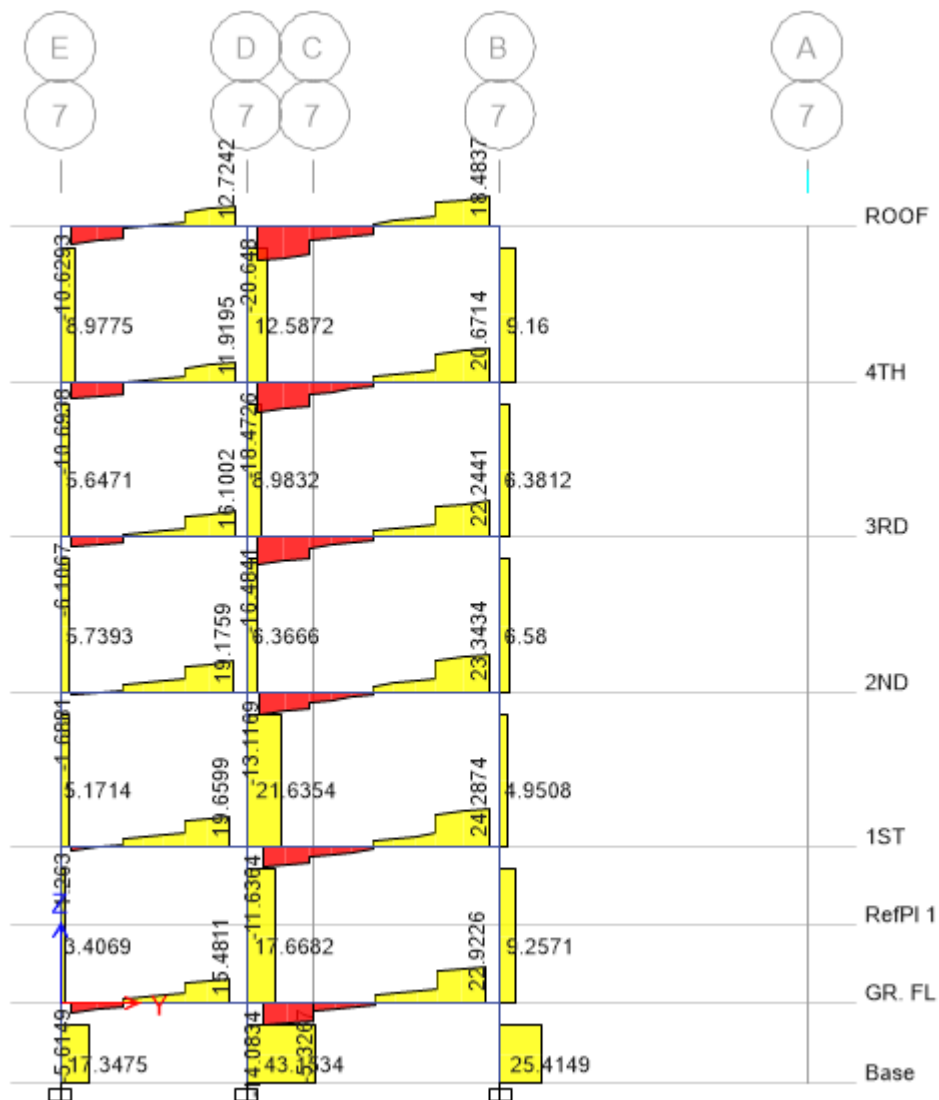
Axis - 5							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	E-D	15.43	262.5	891.9	187.2334	262.5	ø8 c/c 200
	D-B	22.67	262.5	607.0	187.2334	262.5	ø8 c/c 200
	B-A	21.88	262.5	628.9	187.2334	262.5	ø8 c/c 200
4th floor	E-D	20.1	262.5	684.6	187.2334	262.5	ø8 c/c 200
	D-B	25.73	262.5	534.8	187.2334	262.5	ø8 c/c 200
	B-A	28.41	262.5	484.4	187.2334	262.5	ø8 c/c 200
3rd floor	E-D	24.31	262.5	566.1	187.2334	262.5	ø8 c/c 200
	D-B	26.8	262.5	513.5	187.2334	262.5	ø8 c/c 200
	B-A	29.57	262.5	465.4	187.2334	262.5	ø8 c/c 200
2nd floor	E-D	28.02	262.5	491.1	187.2334	262.5	ø8 c/c 200
	D-B	28.96	262.5	475.2	187.2334	262.5	ø8 c/c 200
	B-A	28.91	262.5	476.0	187.2334	262.5	ø8 c/c 200
1st floor	E-D	28.01	262.5	491.3	187.2334	262.5	ø8 c/c 200
	D-B	29.14	262.5	472.3	187.2334	262.5	ø8 c/c 200
	B-A	39.87	262.5	345.2	187.2334	262.5	ø8 c/c 200
GR floor	E-D	22.58	262.5	609.5	187.2334	262.5	ø8 c/c 200
	D-B	28.12	262.5	489.4	187.2334	262.5	ø8 c/c 200
	B-A	39.44	262.5	348.9	187.2334	262.5	ø8 c/c 200



Shear force diagram of beam in axis 6

Structural design of G+4 Apartment + shop building B.Sc Thesis project

Axis - 6							
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	E-D	16.19	262.5	850.0	187.2334	262.5	ø8 c/c 200
	D-B	24.27	262.5	567.0	187.2334	262.5	ø8 c/c 200
4th floor	E-D	15.21	262.5	904.8	187.2334	262.5	ø8 c/c 200
	D-B	24.67	262.5	557.8	187.2334	262.5	ø8 c/c 200
3rd floor	E-D	19.79	262.5	695.4	187.2334	262.5	ø8 c/c 200
	D-B	25.34	262.5	543.1	187.2334	262.5	ø8 c/c 200
2nd floor	E-D	23.84	262.5	577.2	187.2334	262.5	ø8 c/c 200
	D-B	25.41	262.5	541.6	187.2334	262.5	ø8 c/c 200
1st floor	E-D	23.77	262.5	578.9	187.2334	262.5	ø8 c/c 200
	D-B	27.47	262.5	501.0	187.2334	262.5	ø8 c/c 200
GR floor	E-D	18.16	262.5	757.8	187.2334	262.5	ø8 c/c 200
	D-B	28.73	262.5	479.0	187.2334	262.5	ø8 c/c 200



Shear force diagram of beam in axis 7

		Axis - 7					
floor level	span	Ved	Sbmax	Scal	Smin	Spro	Remark
roof slab	E-D	9.34	262.5	1473.4	187.2334	262.5	ø8 c/c 200
	D-B	16.3	262.5	844.3	187.2334	262.5	ø8 c/c 200
4th floor	E-D	8.64	262.5	1592.8	187.2334	262.5	ø8 c/c 200
	D-B	16.32	262.5	843.2	187.2334	262.5	ø8 c/c 200
3rd floor	E-D	12.88	262.5	1068.4	187.2334	262.5	ø8 c/c 200
	D-B	17.97	262.5	765.8	187.2334	262.5	ø8 c/c 200
2nd floor	E-D	16.15	262.5	852.1	187.2334	262.5	ø8 c/c 200
	D-B	19.29	262.5	713.4	187.2334	262.5	ø8 c/c 200
1st floor	E-D	16.63	262.5	827.5	187.2334	262.5	ø8 c/c 200
	D-B	20.29	262.5	678.2	187.2334	262.5	ø8 c/c 200
GR floor	E-D	12.42	262.5	1108.0	187.2334	262.5	ø8 c/c 200
	D-B	18.81	262.5	731.6	187.2334	262.5	ø8 c/c 200

Appendix F – (column design)

F-1 Effective length calculation

location	colu mn	bc	hc	bb	hb	Lc	Lbx left	Lbx right	Lby left	Lby right	k1x	k2x	k1y	k2y	Lox	Loy
ROOF	C1	250	400	250	400	2800	0	4150	0	5700	1.48	1.48	2.04	2.04	2473.94	2546.55
ROOF	C2	250	400	250	400	2800	0	4150	3450	0	1.48	1.48	1.23	1.23	2473.94	2425.48
ROOF	C3	250	400	250	400	2800	0	4150	5700	3450	1.48	1.48	0.77	0.77	2473.94	2282.57
ROOF	C4	250	400	250	400	2800	4150	5040	3450	0	0.81	0.81	1.23	1.23	2301.12	2425.48
ROOF	C5	250	400	250	400	2800	4150	5040	5700	3450	0.81	0.81	0.77	0.77	2301.12	2282.57
ROOF	C6	250	400	250	400	2800	4150	5040	0	5700	0.81	0.81	2.04	2.04	2301.12	2546.55
ROOF	C8	250	400	250	400	2800	5040	5040	5700	4700	0.90	0.90	0.92	0.92	2333.33	2340.14
ROOF	C9	250	400	250	400	2800	5040	5040	0	5700	0.90	0.90	2.04	2.04	2333.33	2546.55
ROOF	C10	250	400	250	400	2800	5040	4250	5700	4700	0.82	0.82	0.92	0.92	2305.29	2340.14
ROOF	C11	250	400	250	400	2800	5040	4250	0	5700	0.82	0.82	2.04	2.04	2305.29	2546.55
ROOF	C12	250	400	250	400	2800	4250	5040	5700	4700	0.82	0.82	0.92	0.92	2305.29	2340.14
ROOF	C13	250	400	300	400	2800	4250	0	0	5700	1.26	1.26	1.70	1.70	2432.63	2506.49
ROOF	C14	250	400	300	400	2800	5040	4150	3450	0	0.68	0.68	1.03	1.03	2241.18	2373.40
ROOF	C15	250	400	300	400	2800	5040	4150	4700	3450	0.68	0.68	0.59	0.59	2241.18	2195.47
ROOF	C17	250	400	300	400	2800	5040	4150	0	4700	0.68	0.68	1.40	1.40	2241.18	2459.24
ROOF	C18	250	400	300	400	2800	4150	0	3450	0	1.24	1.24	1.03	1.03	2426.14	2373.40
ROOF	C19	250	400	300	400	2800	4150	0	4700	3450	1.24	1.24	0.59	0.59	2426.14	2195.47
ROOF	C21	250	400	300	400	2800	4150	0	0	4700	1.24	1.24	1.40	1.40	2426.14	2459.24
ROOF	C22	250	400	300	400	2800	0	5040	3450	0	1.50	1.50	1.03	1.03	2476.92	2373.40
ROOF	C23	250	400	300	400	2800	0	5040	4700	3450	1.50	1.50	0.59	0.59	2476.92	2195.47
ROOF	C24	250	400	300	400	2800	5040	4250	3450	0	0.69	0.69	1.03	1.03	2245.53	2373.40
ROOF	C25	250	400	300	400	2800	5040	4250	4700	3450	0.69	0.69	0.59	0.59	2245.53	2195.47
ROOF	C26	250	400	300	400	2800	4250	5040	3450	0	0.69	0.69	1.03	1.03	2245.53	2373.40
ROOF	C27	250	400	300	400	3200	4250	5040	4700	3450	0.60	0.60	0.52	0.52	2514.58	2456.29
4TH	C1	250	400	300	400	3200	0	4150	0	5700	1.08	1.08	1.48	1.48	2729.64	2827.79
4TH	C2	250	400	300	400	3200	0	4150	3450	0	1.08	1.08	0.90	0.90	2729.64	2666.05
4TH	C3	250	400	300	400	3200	0	4150	5700	3450	1.08	1.08	0.56	0.56	2729.64	2486.90
4TH	C4	250	400	300	400	3200	4150	5040	3450	0	0.59	0.59	0.90	0.90	2509.48	2666.05
4TH	C5	250	400	300	400	3200	4150	5040	5700	3450	0.59	0.59	0.56	0.56	2509.48	2486.90
4TH	C6	250	400	300	400	3200	4150	5040	0	5700	0.59	0.59	1.48	1.48	2509.48	2827.79
4TH	C8	250	400	300	400	3200	5040	5040	5700	4700	0.66	0.66	0.67	0.67	2549.15	2557.62
4TH	C9	250	400	300	400	3200	5040	5040	0	5700	0.66	0.66	1.48	1.48	2549.15	2827.79
4TH	C10	250	400	300	400	3200	5040	4250	5700	4700	0.60	0.60	0.67	0.67	2514.58	2557.62
4TH	C11	250	400	300	400	3200	5040	4250	0	5700	0.60	0.60	1.48	1.48	2514.58	2827.79
4TH	C12	250	400	300	400	3200	4250	5040	5700	4700	0.60	0.60	0.67	0.67	2514.58	2557.62
4TH	C13	250	400	300	400	3200	4250	0	0	5700	1.11	1.11	1.48	1.48	2737.50	2827.79
4TH	C14	250	400	300	400	3200	5040	4150	3450	0	0.59	0.59	0.90	0.90	2509.48	2666.05
4TH	C15	250	400	300	400	3200	5040	4150	4700	3450	0.59	0.59	0.52	0.52	2509.48	2456.29
4TH	C17	250	400	300	400	3200	5040	4150	0	4700	0.59	0.59	1.22	1.22	2509.48	2769.88
4TH	C18	250	400	300	400	3200	4150	0	3450	0	1.08	1.08	0.90	0.90	2729.64	2666.05
4TH	C19	250	400	300	400	3200	4150	0	4700	3450	1.08	1.08	0.52	0.52	2729.64	2456.29
4TH	C21	250	400	300	400	3200	4150	0	0	4700	1.08	1.08	1.22	1.22	2729.64	2769.88
4TH	C22	250	400	300	400	3200	0	5040	3450	0	1.31	1.31	0.90	0.90	2791.49	2666.05
4TH	C23	250	400	300	400	3200	0	5040	4700	3450	1.31	1.31	0.52	0.52	2791.49	2456.29
4TH	C24	250	400	300	400	3200	5040	4250	3450	0	0.60	0.60	0.90	0.90	2514.58	2666.05
4TH	C25	250	400	300	400	3200	5040	4250	4700	3450	0.60	0.60	0.52	0.52	2514.58	2456.29
4TH	C26	250	400	300	400	3200	4250	5040	3450	0	0.60	0.60	0.90	0.90	2514.58	2666.05
4TH	C27	250	400	300	400	3200	4250	5040	4700	3450	0.60	0.60	0.52	0.52	2514.58	2456.29

Structural design of G+4 Apartment + shop building B.Sc Thesis project

location	column	bc	hc	bb	hb	Lc	Lbx left	lbr right	lby left	lby right	k1x	k2x	k1y	k2y	Lox	Loy
3RD	C1	250	400	300	400	3200	0	4150	0	5700	1.08	1.08	1.48	1.48	2729.64	2827.79
3RD	C2	250	400	300	400	3200	0	4150	3450	0	1.08	1.08	0.90	0.90	2729.64	2666.05
3RD	C3	250	400	300	400	3200	0	4150	5700	3450	1.08	1.08	0.56	0.56	2729.64	2486.90
3RD	C4	250	400	300	400	3200	4150	5040	3450	0	0.59	0.59	0.90	0.90	2509.48	2666.05
3RD	C5	250	400	300	400	3200	4150	5040	5700	3450	0.59	0.59	0.56	0.56	2509.48	2486.90
3RD	C6	250	400	300	400	3200	4150	5040	0	5700	0.59	0.59	1.48	1.48	2509.48	2827.79
3RD	C8	250	400	300	400	3200	5040	5040	5700	4700	0.66	0.66	0.67	0.67	2549.15	2557.62
3RD	C9	250	400	300	400	3200	5040	5040	0	5700	0.66	0.66	1.48	1.48	2549.15	2827.79
3RD	C10	250	400	300	400	3200	5040	4250	5700	4700	0.60	0.60	0.67	0.67	2514.58	2557.62
3RD	C11	250	400	300	400	3200	5040	4250	0	5700	0.60	0.60	1.48	1.48	2514.58	2827.79
3RD	C12	250	400	300	400	3200	4250	5040	5700	4700	0.60	0.60	0.67	0.67	2514.58	2557.62
3RD	C13	250	400	300	400	3200	4250	0	0	5700	1.11	1.11	1.48	1.48	2737.50	2827.79
3RD	C14	250	400	300	400	3200	5040	4150	3450	0	0.59	0.59	0.90	0.90	2509.48	2666.05
3RD	C15	250	400	300	400	3200	5040	4150	4700	3450	0.59	0.59	0.52	0.52	2509.48	2456.29
3RD	C17	250	400	300	400	3200	5040	4150	0	4700	0.59	0.59	1.22	1.22	2509.48	2769.88
3RD	C18	250	400	300	400	3200	4150	0	3450	0	1.08	1.08	0.90	0.90	2729.64	2666.05
3RD	C19	250	400	300	400	3200	4150	0	4700	3450	1.08	1.08	0.52	0.52	2729.64	2456.29
3RD	C21	250	400	300	400	3200	4150	0	0	4700	1.08	1.08	1.22	1.22	2729.64	2769.88
3RD	C22	250	400	300	400	3200	0	5040	3450	0	1.31	1.31	0.90	0.90	2791.49	2666.05
3RD	C23	250	400	300	400	3200	0	5040	4700	3450	1.31	1.31	0.52	0.52	2791.49	2456.29
3RD	C24	250	400	300	400	3200	5040	4250	3450	0	0.60	0.60	0.90	0.90	2514.58	2666.05
3RD	C25	250	400	300	400	3200	5040	4250	4700	3450	0.60	0.60	0.52	0.52	2514.58	2456.29
3RD	C26	250	400	300	400	3200	4250	5040	3450	0	0.60	0.60	0.90	0.90	2514.58	2666.05
3RD	C27	250	400	300	400	3200	4250	5040	4700	3450	0.60	0.60	0.52	0.52	2514.58	2456.29
2ND	C1	400	400	300	400	3200	0	4150	0	5700	1.73	1.73	2.38	2.38	2869.60	2945.13
2ND	C2	400	400	300	400	3200	0	4150	3450	0	1.73	1.73	1.44	1.44	2869.60	2818.54
2ND	C3	400	400	300	400	3200	0	4150	5700	3450	1.73	1.73	0.90	0.90	2869.60	2664.88
2ND	C4	400	400	300	400	3200	4150	5040	3450	0	0.95	0.95	1.44	1.44	2685.09	2818.54
2ND	C5	400	400	300	400	3200	4150	5040	5700	3450	0.95	0.95	0.90	0.90	2685.09	2664.88
2ND	C6	400	400	300	400	3200	4150	5040	0	5700	0.95	0.95	2.38	2.38	2685.09	2945.13
2ND	C8	400	400	300	400	3200	5040	5040	5700	4700	1.05	1.05	1.07	1.07	2720.00	2727.35
2ND	C9	400	400	300	400	3200	5040	5040	0	5700	1.05	1.05	2.38	2.38	2720.00	2945.13
2ND	C10	400	400	300	400	3200	5040	4250	5700	4700	0.96	0.96	1.07	1.07	2689.62	2727.35
2ND	C11	400	400	300	400	3200	5040	4250	0	5700	0.96	0.96	2.38	2.38	2689.62	2945.13
2ND	C12	400	400	300	400	3200	4250	5040	5700	4700	0.96	0.96	1.07	1.07	2689.62	2727.35
2ND	C13	400	400	300	400	3200	4250	0	0	5700	1.77	1.77	2.38	2.38	2875.80	2945.13
2ND	C14	400	400	300	400	3200	5040	4150	3450	0	0.95	0.95	1.44	1.44	2685.09	2818.54
2ND	C15	400	400	300	400	3200	5040	4150	4700	3450	0.95	0.95	0.83	0.83	2685.09	2637.05
2ND	C17	400	400	300	400	3200	5040	4150	0	4700	0.95	0.95	1.96	1.96	2685.09	2901.04
2ND	C18	400	400	300	400	3200	4150	0	3450	0	1.73	1.73	1.44	1.44	2869.60	2818.54
2ND	C19	400	400	300	400	3200	4150	0	4700	3450	1.73	1.73	0.83	0.83	2869.60	2637.05
2ND	C21	400	400	300	400	3200	4150	0	0	4700	1.73	1.73	1.96	1.96	2869.60	2901.04
2ND	C22	400	400	300	400	3200	0	5040	3450	0	2.10	2.10	1.44	1.44	2917.65	2818.54
2ND	C23	400	400	300	400	3200	0	5040	4700	3450	2.10	2.10	0.83	0.83	2917.65	2637.05
2ND	C24	400	400	300	400	3200	5040	4250	3450	0	0.96	0.96	1.44	1.44	2689.62	2818.54
2ND	C25	400	400	300	400	3200	5040	4250	4700	3450	0.96	0.96	0.83	0.83	2689.62	2637.05
2ND	C26	400	400	300	400	3200	4250	5040	3450	0	0.96	0.96	1.44	1.44	2689.62	2818.54
2ND	C27	400	400	300	400	3200	4250	5040	4700	3450	0.96	0.96	0.83	0.83	2689.62	2637.05

Structural design of G+4 Apartment + shop building B.Sc Thesis project

location	column	bc	hc	bb	hb	Lc	Lbx left	Lbx right	Lby left	Lby right	k1x	k2x	k1y	k2y	Lox	Loy
1ST	C1	400	400	300	400	3200	0	4150	0	5700	1.73	1.73	2.38	2.38	2869.60	2945.13
1ST	C2	400	400	300	400	3200	0	4150	3450	0	1.73	1.73	1.44	1.44	2869.60	2818.54
1ST	C3	400	400	300	400	3200	0	4150	5700	3450	1.73	1.73	0.90	0.90	2869.60	2664.88
1ST	C4	400	400	300	400	3200	4150	5040	3450	0	0.95	0.95	1.44	1.44	2685.09	2818.54
1ST	C5	400	400	300	400	3200	4150	5040	5700	3450	0.95	0.95	0.90	0.90	2685.09	2664.88
1ST	C6	400	400	300	400	3200	4150	5040	0	5700	0.95	0.95	2.38	2.38	2685.09	2945.13
1ST	C8	400	400	300	400	3200	5040	5040	5700	4700	1.05	1.05	1.07	1.07	2720.00	2727.35
1ST	C9	400	400	300	400	3200	5040	5040	0	5700	1.05	1.05	2.38	2.38	2720.00	2945.13
1ST	C10	400	400	300	400	3200	5040	4250	5700	4700	0.96	0.96	1.07	1.07	2689.62	2727.35
1ST	C11	400	400	300	400	3200	5040	4250	0	5700	0.96	0.96	2.38	2.38	2689.62	2945.13
1ST	C12	400	400	300	400	3200	4250	5040	5700	4700	0.96	0.96	1.07	1.07	2689.62	2727.35
1ST	C13	400	400	300	400	3200	4250	0	0	5700	1.77	1.77	2.38	2.38	2875.80	2945.13
1ST	C14	400	400	300	400	3200	5040	4150	3450	0	0.95	0.95	1.44	1.44	2685.09	2818.54
1ST	C15	400	400	300	400	3200	5040	4150	4700	3450	0.95	0.95	0.83	0.83	2685.09	2637.05
1ST	C17	400	400	300	400	3200	5040	4150	0	4700	0.95	0.95	1.96	1.96	2685.09	2901.04
1ST	C18	400	400	300	400	3200	4150	0	3450	0	1.73	1.73	1.44	1.44	2869.60	2818.54
1ST	C19	400	400	300	400	3200	4150	0	4700	3450	1.73	1.73	0.83	0.83	2869.60	2637.05
1ST	C21	400	400	300	400	3200	4150	0	0	4700	1.73	1.73	1.96	1.96	2869.60	2901.04
1ST	C22	400	400	300	400	3200	0	5040	3450	0	2.10	2.10	1.44	1.44	2917.65	2818.54
1ST	C23	400	400	300	400	3200	0	5040	4700	3450	2.10	2.10	0.83	0.83	2917.65	2637.05
1ST	C24	400	400	300	400	3200	5040	4250	3450	0	0.96	0.96	1.44	1.44	2689.62	2818.54
1ST	C25	400	400	300	400	3200	5040	4250	4700	3450	0.96	0.96	0.83	0.83	2689.62	2637.05
1ST	C26	400	400	300	400	3200	4250	5040	3450	0	0.96	0.96	1.44	1.44	2689.62	2818.54
1ST	C27	400	400	300	400	3200	4250	5040	4700	3450	0.96	0.96	0.83	0.83	2689.62	2637.05
GR. FL	C1	500	500	300	400	3200	0	4150	0	5700	4.22	4.22	5.80	5.80	3045.88	3084.77
GR. FL	C2	500	500	300	400	3200	0	4150	3450	0	4.22	4.22	3.51	3.51	3045.88	3018.16
GR. FL	C3	500	500	300	400	3200	0	4150	5700	3450	4.22	4.22	2.19	2.19	3045.88	2926.89
GR. FL	C4	500	500	300	400	3200	4150	5040	3450	0	2.32	2.32	3.51	3.51	2939.62	3018.16
GR. FL	C5	500	500	300	400	3200	4150	5040	5700	3450	2.32	2.32	2.19	2.19	2939.62	2926.89
GR. FL	C6	500	500	300	400	3200	4150	5040	0	5700	2.32	2.32	5.80	5.80	2939.62	3084.77
GR. FL	C8	500	500	300	400	3200	5040	5040	5700	4700	2.56	2.56	2.62	2.62	2961.07	2965.50
GR. FL	C9	500	500	300	400	3200	5040	5040	0	5700	2.56	2.56	5.80	5.80	2961.07	3084.77
GR. FL	C10	500	500	300	400	3200	5040	4250	5700	4700	2.35	2.35	2.62	2.62	2942.44	2965.50
GR. FL	C11	500	500	300	400	3200	5040	4250	0	5700	2.35	2.35	5.80	5.80	2942.44	3084.77
GR. FL	C12	500	500	300	400	3200	4250	5040	5700	4700	2.35	2.35	2.62	2.62	2942.44	2965.50
GR. FL	C13	500	500	300	400	3200	4250	0	0	5700	4.32	4.32	5.80	5.80	3049.16	3084.77
GR. FL	C14	500	500	300	400	3200	5040	4150	3450	0	2.32	2.32	3.51	3.51	2939.62	3018.16
GR. FL	C15	500	500	300	400	3200	5040	4150	4700	3450	2.32	2.32	2.02	2.02	2939.62	2908.96
GR. FL	C17	500	500	300	400	3200	5040	4150	0	4700	2.32	2.32	4.78	4.78	2939.62	3062.36
GR. FL	C18	500	500	300	400	3200	4150	0	3450	0	4.22	4.22	3.51	3.51	3045.88	3018.16
GR. FL	C19	500	500	300	400	3200	4150	0	4700	3450	4.22	4.22	2.02	2.02	3045.88	2908.96
GR. FL	C21	500	500	300	400	3200	4150	0	0	4700	4.22	4.22	4.78	4.78	3045.88	3062.36
GR. FL	C22	500	500	300	400	3200	0	5040	3450	0	5.13	5.13	3.51	3.51	3070.90	3018.16
GR. FL	C23	500	500	300	400	3200	0	5040	4700	3450	5.13	5.13	2.02	2.02	3070.90	2908.96
GR. FL	C24	500	500	300	400	3200	5040	4250	3450	0	2.35	2.35	3.51	3.51	2942.44	3018.16
GR. FL	C25	500	500	300	400	3200	5040	4250	4700	3450	2.35	2.35	2.02	2.02	2942.44	2908.96
GR. FL	C26	500	500	300	400	3200	4250	5040	3450	0	2.35	2.35	3.51	3.51	2942.44	3018.16
GR. FL	C27	500	500	300	400	3200	4250	5040	4700	3450	2.35	2.35	2.02	2.02	2942.44	2908.96

F-2 Slenderness check

location	column	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λ_x	λ_y	λ_{lim}	λ_{ylim}	slenderness(x)	slenderness(y)
ROOF	C1	24.34	25.60	8.91	9.95	66.17	25.66	10.23	26.93	11.27	21.42	22.05	47.60	50.49	short	short
ROOF	C2	4.88	2.11	2.77	1.88	17.66	2.46	2.24	5.23	3.12	21.42	21.01	151.65	121.37	short	short
ROOF	C3	23.48	23.04	8.15	7.87	71.35	24.47	9.30	24.91	9.57	21.42	19.77	44.04	44.71	short	short
ROOF	C4	8.40	12.79	8.36	9.42	55.05	9.51	9.46	13.89	10.52	19.93	21.01	70.96	55.91	short	short
ROOF	C5	27.95	22.01	7.20	6.67	197.73	25.96	10.63	31.91	11.15	19.93	19.77	32.67	27.55	short	short
ROOF	C6	38.03	41.63	1.89	0.22	139.01	40.81	3.00	44.41	4.67	19.93	22.05	34.34	46.53	short	short
ROOF	C8	16.39	12.86	0.48	1.49	292.26	18.71	6.33	22.23	7.34	20.21	20.27	26.03	25.40	short	short
ROOF	C9	39.42	41.39	0.91	2.69	147.48	42.37	3.86	44.34	5.64	20.21	22.05	31.78	43.33	short	short
ROOF	C10	15.12	12.86	3.75	6.01	259.83	18.06	8.95	20.31	11.21	19.96	20.27	26.07	29.00	short	short
ROOF	C11	36.06	40.17	3.52	5.57	134.22	38.74	6.20	42.85	8.26	19.96	22.05	35.62	42.46	short	short
ROOF	C12	0.70	3.66	4.80	8.83	180.46	4.31	8.41	7.27	12.44	19.96	20.27	42.70	39.51	short	short
ROOF	C13	23.21	25.80	10.81	14.79	66.76	24.54	12.14	27.13	16.13	21.07	21.71	50.46	60.08	short	short
ROOF	C14	7.54	7.72	9.02	12.47	123.84	10.02	11.50	10.20	14.94	19.41	20.55	33.42	43.35	short	short
ROOF	C15	11.57	10.47	5.50	8.37	216.67	14.80	9.84	15.91	12.70	19.41	19.01	27.10	32.60	short	short
ROOF	C17	21.95	24.32	4.56	7.35	117.90	24.31	6.92	26.67	9.70	19.41	21.30	37.66	47.12	short	short
ROOF	C18	8.88	9.48	9.11	12.38	42.52	9.73	9.96	10.33	13.23	21.01	20.55	60.28	75.29	short	short
ROOF	C19	10.20	9.11	13.08	17.26	95.39	11.02	14.99	12.11	19.16	21.01	19.01	41.93	48.71	short	short
ROOF	C21	11.66	12.32	9.53	12.44	54.62	12.76	10.62	13.42	13.54	21.01	21.30	52.55	64.22	short	short
ROOF	C22	6.80	7.38	9.38	10.28	111.21	9.03	11.61	9.60	12.51	21.45	20.55	37.33	37.95	short	short
ROOF	C23	9.89	11.16	8.52	10.46	172.66	13.35	11.97	14.61	13.91	21.45	19.01	31.04	33.12	short	short
ROOF	C24	6.95	9.40	7.06	8.99	120.16	9.35	9.46	11.80	11.39	19.45	20.55	42.92	41.10	short	short
ROOF	C25	9.57	9.35	3.65	5.54	197.46	13.30	7.60	13.52	9.49	19.45	19.01	26.41	33.17	short	short
ROOF	C26	6.97	8.92	5.92	4.50	117.59	9.32	6.85	11.27	8.27	19.45	20.55	41.73	41.69	short	short
ROOF	C27	10.22	8.99	2.54	0.06	201.04	13.01	4.09	14.25	6.56	21.78	21.27	28.77	39.37	short	short
4TH	C1	19.40	27.40	8.39	9.59	148.47	22.37	11.36	30.37	12.56	23.64	24.49	40.98	33.83	short	short
4TH	C2	1.38	15.79	2.85	4.59	72.14	2.82	4.29	17.23	6.04	23.64	23.09	93.76	60.34	short	short
4TH	C3	14.39	0.42	7.60	7.49	179.68	4.02	11.08	17.98	11.20	23.64	21.54	57.11	27.46	short	short
4TH	C4	5.14	5.11	7.70	8.63	125.57	7.62	10.21	7.65	11.14	21.73	23.09	32.59	36.24	short	short
4TH	C5	3.25	1.14	5.58	6.25	369.57	8.53	12.97	10.64	13.64	21.73	21.54	24.21	20.19	short	slender
4TH	C6	27.28	40.09	1.94	0.15	293.63	33.15	6.02	45.96	7.81	21.73	24.49	29.61	28.11	short	short
4TH	C8	3.30	1.97	1.21	0.40	615.72	14.29	12.72	15.61	13.52	22.08	22.15	16.39	15.87	slender	slender
4TH	C9	25.11	37.52	0.38	1.77	299.45	31.10	6.37	43.51	7.76	22.08	24.49	29.52	26.34	short	short
4TH	C10	6.21	4.00	1.34	2.46	574.40	15.48	12.82	17.70	13.95	21.78	22.15	17.85	16.88	slender	slender
4TH	C11	25.02	38.14	1.10	3.78	285.53	30.73	6.82	43.85	9.49	21.78	24.49	30.65	30.13	short	short
4TH	C12	1.63	8.72	4.18	7.04	389.19	9.42	11.96	16.50	14.82	21.78	22.15	29.68	23.46	short	short
4TH	C13	16.10	25.51	8.09	12.75	151.18	19.12	11.11	28.53	15.77	23.71	24.49	43.41	41.97	short	short
4TH	C14	4.80	0.28	5.72	8.22	245.26	5.18	10.62	9.70	13.12	21.73	23.09	38.59	29.47	short	short
4TH	C15	8.08	1.66	3.27	4.68	429.09	10.24	11.86	16.66	13.26	21.73	21.27	27.16	20.17	short	slender
4TH	C17	12.74	23.53	2.56	4.63	237.49	17.49	7.31	28.28	9.38	21.73	23.99	36.39	30.98	short	short
4TH	C18	6.67	3.73	5.70	8.05	89.81	5.52	7.49	8.46	9.85	23.64	23.09	57.31	51.39	short	short
4TH	C19	8.03	1.36	9.16	13.01	194.52	5.25	13.05	11.92	16.90	23.64	21.27	46.82	34.49	short	short
4TH	C21	5.20	12.65	6.12	9.23	113.54	7.48	8.39	14.92	11.50	23.64	23.99	58.32	47.18	short	short
4TH	C22	1.56	0.05	8.86	11.68	230.26	4.66	13.47	6.16	16.28	24.18	23.09	32.26	29.82	short	short
4TH	C23	5.35	2.39	9.63	12.82	387.21	10.14	17.38	13.10	20.57	24.18	21.27	24.39	22.53	short	short
4TH	C24	5.10	2.57	2.93	4.70	275.49	8.08	8.44	10.61	10.21	21.78	23.09	29.31	27.28	short	short
4TH	C25	4.94	0.40	1.11	1.72	487.04	10.14	10.85	14.68	11.46	21.78	21.27	23.71	17.70	short	slender
4TH	C26	5.44	2.19	4.27	5.46	262.26	7.44	9.51	10.68	10.70	21.78	23.09	32.13	25.97	short	short
4TH	C27	5.59	0.23	0.99	1.11	455.10	9.34	10.09	14.69	10.21	21.78	21.27	25.87	17.31	short	slender

Structural design of G+4 Apartment + shop building B.Sc Thesis project

locati on	colu mn	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λ_x	λ_y	λ_{lim}	λ_{ylim}	slende rnes(x)	slender nes(y)
3RD	C1	18.22	26.45	8.22	8.24	231.13	22.84	12.84	31.07	12.87	23.64	24.49	32.90	23.94	short	slender
3RD	C2	7.85	14.30	2.46	1.86	113.53	10.12	4.13	16.58	4.73	23.64	23.09	53.01	40.18	short	short
3RD	C3	20.95	1.42	6.75	4.47	288.61	7.19	10.24	26.72	12.52	23.64	21.54	43.66	26.92	short	short
3RD	C4	4.69	2.23	4.90	6.52	265.24	7.54	10.21	10.00	11.82	21.73	23.09	30.10	26.63	short	short
3RD	C5	2.42	2.33	4.58	4.10	554.69	13.43	15.19	13.51	15.67	21.73	21.54	15.54	16.08	slender	slender
3RD	C6	26.74	40.06	2.06	0.99	448.70	35.72	9.96	49.03	11.04	21.73	24.49	23.78	19.51	short	slender
3RD	C8	7.97	2.20	2.18	0.87	949.26	21.18	19.86	26.96	21.16	22.08	22.15	15.38	12.82	slender	slender
3RD	C9	24.56	38.45	0.78	2.26	448.86	33.53	9.76	47.43	11.23	22.08	24.49	24.29	20.33	short	slender
3RD	C10	11.39	3.02	3.07	7.10	897.97	20.98	21.03	29.35	25.06	21.78	22.15	17.04	14.89	slender	slender
3RD	C11	24.70	40.36	0.38	3.91	436.19	33.42	9.10	49.08	12.64	21.78	24.49	25.29	24.32	short	slender
3RD	C12	0.16	7.17	3.30	4.47	597.49	12.11	15.25	19.12	16.42	21.78	22.15	22.62	16.36	short	slender
3RD	C13	15.83	26.99	7.49	12.44	233.99	20.51	12.17	31.67	17.12	23.71	24.49	35.66	33.51	short	short
3RD	C14	3.34	8.04	6.20	8.85	359.50	10.53	13.39	15.23	16.04	21.73	23.09	27.56	23.65	short	short
3RD	C15	7.76	3.01	3.01	3.09	652.19	16.05	16.06	20.80	16.13	21.73	21.27	18.84	14.30	slender	slender
3RD	C17	12.50	25.92	2.20	4.60	354.72	19.59	9.29	33.01	11.69	21.73	23.99	30.45	24.92	short	short
3RD	C18	5.79	3.05	6.13	7.88	130.46	5.66	8.74	8.40	10.49	23.64	23.09	46.57	39.35	short	short
3RD	C19	7.54	3.17	8.11	7.53	292.30	9.01	13.37	13.39	13.95	23.64	21.27	31.14	22.48	short	short
3RD	C21	5.90	16.72	6.22	9.29	170.59	9.31	9.63	20.13	12.70	23.64	23.99	49.11	37.38	short	short
3RD	C22	2.12	3.30	7.81	9.81	345.27	9.02	14.72	10.20	16.71	24.18	23.09	22.76	22.86	slender	slender
3RD	C23	4.64	0.47	7.58	7.63	599.84	12.47	19.58	16.63	19.62	24.18	21.27	20.12	14.86	slender	slender
3RD	C24	4.65	1.44	3.95	6.02	425.01	9.94	12.45	13.15	14.52	21.78	23.09	23.74	21.19	short	slender
3RD	C25	6.14	4.14	5.16	5.31	784.53	19.83	20.85	21.84	21.00	21.78	21.27	14.65	13.09	slender	slender
3RD	C26	4.11	4.85	3.63	4.17	399.30	12.10	11.62	12.84	12.15	21.78	23.09	19.65	19.30	slender	slender
3RD	C27	5.16	4.20	0.45	0.21	712.34	18.45	14.46	19.40	14.69	21.78	21.27	14.55	13.91	slender	slender
2ND	C1	17.70	25.21	7.36	7.14	312.07	23.94	13.38	31.45	13.60	24.85	25.51	34.84	26.60	short	short
2ND	C2	6.39	13.39	2.76	1.52	142.99	9.25	4.38	16.25	5.62	24.85	24.41	62.02	50.44	short	short
2ND	C3	19.55	2.84	6.39	1.80	395.35	10.75	9.70	27.46	14.29	24.85	23.08	43.16	33.67	short	short
2ND	C4	1.08	0.75	4.56	6.28	371.66	8.19	11.99	8.51	13.71	23.25	24.41	25.12	28.08	short	short
2ND	C5	0.13	6.91	11.46	9.68	781.87	15.77	25.32	22.55	27.09	23.25	23.08	23.46	17.95	short	slender
2ND	C6	25.72	36.22	1.44	1.29	603.68	37.80	13.36	48.29	13.51	23.25	25.51	24.48	18.97	short	slender
2ND	C8	10.69	2.21	1.87	2.09	1288.4	27.98	27.64	36.46	27.86	23.56	23.62	17.04	12.93	slender	slender
2ND	C9	23.48	33.85	0.77	1.66	596.05	35.40	12.69	45.77	13.58	23.56	25.51	24.88	20.57	short	slender
2ND	C10	14.66	2.49	5.74	11.57	1223.97	26.97	30.22	39.14	36.05	23.29	23.62	18.95	16.15	slender	slender
2ND	C11	24.30	36.86	0.04	2.34	587.12	36.04	11.78	48.61	14.08	23.29	25.51	25.94	23.35	short	slender
2ND	C12	0.39	21.17	15.71	26.19	823.03	16.85	32.17	37.63	42.65	23.29	23.62	28.62	21.61	short	slender
2ND	C13	15.97	28.22	6.88	11.64	314.96	22.27	13.18	34.52	17.94	24.91	25.51	38.97	35.66	short	short
2ND	C14	0.10	10.41	6.30	8.86	464.88	9.40	15.60	19.70	18.16	23.25	24.41	37.19	25.57	short	short
2ND	C15	11.11	11.61	14.32	14.14	891.83	28.95	31.98	29.44	32.16	23.25	22.84	15.74	15.50	slender	slender
2ND	C17	12.03	25.49	1.82	3.83	469.41	21.42	11.21	34.87	13.21	23.25	25.12	32.86	25.78	short	short
2ND	C18	3.08	6.01	5.59	7.22	165.23	6.38	8.89	9.32	10.52	24.85	24.41	51.79	43.60	short	short
2ND	C19	9.32	12.39	28.24	24.62	412.24	17.57	32.86	20.64	36.48	24.85	22.84	27.41	25.81	short	short
2ND	C21	6.54	16.98	5.26	6.35	226.41	11.07	9.79	21.51	10.88	24.85	25.12	51.66	34.89	short	short
2ND	C22	0.30	3.43	6.55	8.22	455.19	9.41	15.66	12.53	17.33	25.27	24.41	29.18	24.48	short	short
2ND	C23	7.35	0.92	27.90	23.85	836.85	17.66	40.59	24.09	44.63	25.27	22.84	21.91	17.92	slender	slender
2ND	C24	3.42	1.33	4.17	5.34	572.92	12.79	15.63	14.88	16.80	23.29	24.41	23.04	21.09	slender	slender
2ND	C25	5.73	6.49	8.13	8.84	1080.33	27.34	29.74	28.10	30.45	23.29	22.84	14.50	14.43	slender	slender
2ND	C26	1.11	6.50	2.91	3.39	528.05	11.68	13.47	17.06	13.95	23.29	24.41	28.98	20.95	short	slender
2ND	C27	5.83	11.56	2.36	3.13	992.85	25.69	22.21	31.42	22.98	23.29	22.84	18.36	15.26	slender	slender

Structural design of G+4 Apartment + shop building B.Sc Thesis project

locatio n	colu mn	Mx top	Mx botm	My top	My bot	NED	M01x	M01y	M02x	M02y	λ_x	λ_y	λ_{lim}	λ_{ylim}	slende rnes(x)	slender nes(y)
1ST	C1	17.02	20.92	5.44	3.65	389.44	24.81	11.43	28.70	13.23	24.85	25.51	27.77	27.77	short	short
1ST	C2	4.82	13.37	2.67	2.47	161.87	8.06	5.70	16.61	5.91	24.85	24.41	62.60	37.90	short	short
1ST	C3	23.00	22.18	12.61	6.48	510.77	32.40	16.69	33.22	22.82	24.85	23.08	21.03	28.10	slender	short
1ST	C4	0.02	2.52	3.90	3.50	462.87	9.28	12.76	11.78	13.16	23.25	24.41	27.81	22.25	short	slender
1ST	C5	2.51	4.79	12.35	3.29	1019.49	22.90	23.68	25.18	32.74	23.25	23.08	16.24	20.06	slender	slender
1ST	C6	33.53	37.36	1.06	4.73	767.38	48.88	16.40	52.70	20.07	23.25	25.51	18.29	20.90	slender	slender
1ST	C8	6.20	14.05	0.13	6.42	1635.97	38.92	32.85	46.77	39.14	23.56	23.62	14.07	13.95	slender	slender
1ST	C9	31.21	44.26	0.62	5.63	748.43	46.18	15.59	59.23	20.60	23.56	25.51	22.06	22.61	slender	slender
1ST	C10	8.44	9.51	4.68	10.39	1550.61	39.45	35.70	40.52	41.40	23.29	23.62	12.10	13.95	slender	slender
1ST	C11	32.47	47.44	0.14	7.30	747.79	47.42	15.10	62.39	22.26	23.29	25.51	22.54	24.49	slender	slender
1ST	C12	0.68	12.69	6.13	10.39	1037.30	21.42	26.87	33.43	31.14	23.29	23.62	21.56	17.04	slender	slender
1ST	C13	15.11	23.73	5.85	7.69	394.72	23.00	13.74	31.63	15.59	24.91	25.51	32.10	27.01	short	short
1ST	C14	1.84	7.04	5.14	5.88	572.97	13.30	16.60	18.50	17.33	23.25	24.41	26.87	20.34	short	slender
1ST	C15	10.08	30.42	12.15	14.05	1136.12	32.80	34.87	53.14	36.77	23.25	22.84	21.06	14.63	slender	slender
1ST	C17	15.91	29.89	5.50	7.77	591.94	27.75	17.34	41.73	19.61	23.25	25.12	27.90	21.98	short	slender
1ST	C18	3.89	5.81	4.14	4.27	197.63	7.84	8.10	9.76	8.23	24.85	24.41	41.80	33.39	short	short
1ST	C19	7.34	36.22	19.66	24.15	544.13	18.22	30.55	47.11	35.04	24.85	22.84	36.91	23.28	short	short
1ST	C21	7.74	21.48	10.69	10.96	289.45	13.53	16.48	27.27	16.75	24.85	25.12	46.39	27.61	short	short
1ST	C22	1.30	1.84	5.42	5.33	564.47	12.58	16.62	13.13	16.71	25.27	24.41	20.47	19.46	slender	slender
1ST	C23	5.12	13.88	18.10	22.12	1081.26	26.75	39.73	35.50	43.74	25.27	22.84	18.88	15.79	slender	slender
1ST	C24	6.01	1.51	11.23	13.80	727.93	16.07	25.79	20.57	28.36	23.29	24.41	22.33	19.21	slender	slender
1ST	C25	6.45	22.79	8.74	11.00	1381.52	34.08	36.37	50.42	38.63	23.29	22.84	18.07	13.38	slender	slender
1ST	C26	2.79	3.60	2.51	1.95	658.33	15.95	15.11	16.76	15.68	23.29	24.41	19.12	18.81	slender	slender
1ST	C27	6.36	27.78	3.26	4.56	1281.28	31.99	28.88	53.41	30.19	23.29	22.84	20.17	13.61	slender	slender
GR. FL	C1	15.85	19.63	10.70	1.60	464.29	25.13	10.89	28.91	19.99	21.10	21.37	31.60	43.94	short	short
GR. FL	C2	8.83	5.81	8.23	4.22	210.60	10.03	8.43	13.05	12.44	21.10	20.91	52.61	57.72	short	short
GR. FL	C3	25.76	11.53	25.08	7.97	650.92	24.55	20.99	38.78	38.10	21.10	20.28	34.27	36.92	short	short
GR. FL	C4	11.65	1.01	7.42	2.67	615.76	13.32	14.99	23.97	19.73	20.37	20.91	37.79	31.06	short	short
GR. FL	C5	17.01	10.36	3.04	17.03	1323.08	36.83	29.50	43.47	43.49	20.37	20.28	19.22	23.02	slender	short
GR. FL	C6	33.54	41.10	3.61	18.77	929.20	52.13	22.19	59.69	37.35	20.37	21.37	22.23	29.73	short	short
GR. FL	C8	22.17	30.27	7.40	33.29	1962.32	61.41	46.65	69.52	72.54	20.51	20.55	15.11	19.56	slender	slender
GR. FL	C9	22.48	24.65	0.39	8.71	894.74	40.37	18.29	42.54	26.60	20.51	21.37	20.58	27.75	short	short
GR. FL	C10	22.68	32.00	2.06	38.38	1874.65	60.18	39.56	69.49	75.88	20.39	20.55	15.79	22.31	slender	short
GR. FL	C11	22.78	25.29	1.06	9.71	898.88	40.76	19.04	43.27	27.69	20.39	21.37	20.72	27.68	short	short
GR. FL	C12	0.91	23.84	11.76	26.17	1253.24	25.97	36.82	48.90	51.23	20.39	20.55	27.06	22.72	short	short
GR. FL	C13	14.94	20.25	12.37	17.51	475.42	24.45	21.88	29.76	27.01	21.13	21.37	33.02	33.46	short	short
GR. FL	C14	4.77	7.11	10.88	14.98	688.70	18.54	24.66	20.88	28.75	20.37	20.91	25.36	26.31	short	short
GR. FL	C15	30.11	31.70	2.69	35.34	1355.41	57.22	29.80	58.81	62.44	20.37	20.15	16.19	27.22	slender	short
GR. FL	C17	15.45	33.84	2.39	20.38	713.74	29.73	16.67	48.12	34.66	20.37	21.22	33.20	37.40	short	short
GR. FL	C18	5.82	6.74	6.52	11.82	234.90	10.51	11.22	11.44	16.51	21.10	20.91	41.77	54.57	short	short
GR. FL	C19	29.99	33.42	8.84	38.63	656.74	43.12	21.98	46.56	51.76	21.10	20.15	24.75	40.79	short	short
GR. FL	C21	2.92	26.94	5.60	22.36	353.37	9.99	12.66	34.01	29.43	21.10	21.22	61.31	55.36	short	short
GR. FL	C22	6.24	5.01	13.89	2.58	681.56	18.65	16.21	19.87	27.52	21.28	20.91	23.92	34.89	short	short
GR. FL	C23	22.90	29.88	20.68	22.75	1298.64	48.87	46.65	55.85	48.72	21.28	20.15	18.76	16.88	slender	slender
GR. FL	C24	7.67	4.30	6.23	12.04	876.95	21.84	23.77	25.21	29.58	20.39	20.91	23.08	24.81	short	short
GR. FL	C25	22.86	31.91	1.36	34.65	1663.05	56.12	34.62	65.17	67.91	20.39	20.15	16.86	23.92	slender	short
GR. FL	C26	7.18	5.01	5.75	3.28	798.65	20.98	19.26	23.16	21.73	20.39	20.91	23.03	23.60	short	short
GR. FL	C27	25.46	32.20	2.61	32.47	1541.33	56.29	33.44	63.03	63.29	20.39	20.15	16.85	24.46	slender	short

F-3 Reinforcement calculation

colm n	loca tion	MED x	MED y	Vsd	μ sd y	μ sd x	ω	As	As, min	As, max	As, prov	ϕ	no of	longtu dinal	shear reinforcme	
ROOF	C1	26.93	11.27	0.1	0.02	0.06	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C2	5.23	3.12	0.0	0.01	0.01	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C3	24.91	9.57	0.1	0.02	0.05	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C4	13.89	10.52	0.0	0.02	0.03	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c	250
ROOF	C5	31.91	11.15	0.2	0.02	0.07	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C6	44.41	4.67	0.1	0.01	0.10	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
ROOF	C8	22.23	7.34	0.3	0.02	0.05	0.01	46	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C9	44.34	5.64	0.1	0.01	0.10	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
ROOF	C10	20.31	11.21	0.2	0.02	0.04	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C11	42.85	8.26	0.1	0.02	0.09	0.5	1916	200	4000	1916	24	4.24	6 ϕ 16	ϕ 8 c/c	250
ROOF	C12	7.27	12.44	0.2	0.03	0.02	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C13	27.13	16.13	0.1	0.04	0.06	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C14	10.20	14.94	0.1	0.03	0.02	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C15	15.91	12.70	0.2	0.03	0.04	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C17	26.67	9.70	0.1	0.02	0.06	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C18	10.33	13.23	0.0	0.03	0.02	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C19	12.11	19.16	0.1	0.04	0.03	0.01	38.3	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C21	13.42	13.54	0.0	0.03	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C22	9.60	12.51	0.1	0.03	0.02	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c	250
ROOF	C23	14.61	13.91	0.2	0.03	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C24	11.80	11.39	0.1	0.03	0.03	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
ROOF	C25	13.52	9.49	0.2	0.02	0.03	0.01	46	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
ROOF	C26	11.27	8.27	0.1	0.02	0.02	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
ROOF	C27	14.25	6.56	0.2	0.01	0.03	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C1	30.37	12.56	0.1	0.03	0.07	0.5	1916	200	4000	1916	24	4.24	6 ϕ 16	ϕ 8 c/c	250
4TH	C2	17.23	6.04	0.1	0.01	0.04	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C3	17.98	11.20	0.2	0.02	0.04	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C4	7.65	11.14	0.1	0.02	0.02	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C5	10.64	13.64	0.3	0.03	0.02	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C6	45.96	7.81	0.3	0.02	0.10	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C8	15.61	13.52	0.5	0.03	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C9	43.51	7.76	0.3	0.02	0.10	0.01	38.3	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C10	17.70	13.95	0.5	0.03	0.04	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C11	43.85	9.49	0.3	0.02	0.10	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c	250
4TH	C12	16.50	14.82	0.3	0.03	0.04	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C13	28.53	15.77	0.1	0.03	0.06	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
4TH	C14	9.70	13.12	0.2	0.03	0.02	0.01	46	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C15	16.66	13.26	0.4	0.03	0.04	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c	250
4TH	C17	28.28	9.38	0.2	0.02	0.06	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C18	8.46	9.85	0.1	0.02	0.02	0.5	1916	200	4000	1916	24	4.24	6 ϕ 16	ϕ 8 c/c	250
4TH	C19	11.92	16.90	0.2	0.04	0.03	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C21	14.92	11.50	0.1	0.03	0.03	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C22	6.16	16.28	0.2	0.04	0.01	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C23	13.10	20.57	0.3	0.05	0.03	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C24	10.61	10.21	0.2	0.02	0.02	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C25	14.68	11.46	0.4	0.03	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C26	10.68	10.70	0.2	0.02	0.02	0.01	38.3	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250
4TH	C27	14.69	10.21	0.4	0.02	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c	250

Structural design of G+4 Apartment + shop building B.Sc Thesis project

colm n	loca tion	MED x	MED y	Vsd	μ sd y	μ sd x	ω	As	As, min	As, max	As, prov	ϕ	no of	longtu dinal	shear reinforcme
3RD	C1	31.07	12.87	0.2	0.03	0.07	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c 250
3RD	C2	16.58	4.73	0.1	0.01	0.04	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C3	26.72	12.52	0.3	0.03	0.06	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c 250
3RD	C4	10.00	11.82	0.2	0.03	0.02	0.01	46	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C5	13.51	15.67	0.5	0.03	0.03	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c 250
3RD	C6	49.03	11.04	0.4	0.02	0.11	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C8	26.96	21.16	0.8	0.05	0.06	0.5	1916	200	4000	1916	24	4.24	6 ϕ 16	ϕ 8 c/c 250
3RD	C9	47.43	11.23	0.4	0.02	0.10	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C10	29.35	25.06	0.8	0.06	0.06	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C11	49.08	12.64	0.4	0.03	0.11	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C12	19.12	16.42	0.5	0.04	0.04	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C13	31.67	17.12	0.2	0.04	0.07	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C14	15.23	16.04	0.3	0.04	0.03	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C15	20.80	16.13	0.6	0.04	0.05	0.01	38.3	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C17	33.01	11.69	0.3	0.03	0.07	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c 250
3RD	C18	8.40	10.49	0.1	0.02	0.02	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C19	13.39	13.95	0.3	0.03	0.03	0.02	76.6	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C21	20.13	12.70	0.2	0.03	0.04	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C22	10.20	16.71	0.3	0.04	0.02	0.01	38.3	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C23	16.63	19.62	0.5	0.04	0.04	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C24	13.15	14.52	0.4	0.03	0.03	0.1	383	200	4000	383	16	1.91	4 ϕ 16	ϕ 8 c/c 250
3RD	C25	21.84	21.00	0.7	0.05	0.05	0	0	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
3RD	C26	12.84	12.15	0.4	0.03	0.03	0.15	575	200	4000	575	16	2.86	4 ϕ 16	ϕ 8 c/c 250
3RD	C27	19.40	14.69	0.6	0.03	0.04	0.01	46	200	4000	200	16	1.00	4 ϕ 16	ϕ 8 c/c 250
2ND	C1	31.45	13.60	0.2	0.02	0.04	0.15	920	320	6400	920	16	4.58	4 ϕ 16	ϕ 8 c/c 320
2ND	C2	16.25	5.62	0.1	0.01	0.02	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C3	27.46	14.29	0.2	0.02	0.04	0.15	920	320	6400	920	20	2.93	4 ϕ 20	ϕ 8 c/c 400
2ND	C4	8.51	13.71	0.2	0.02	0.01	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C5	22.55	27.09	0.4	0.04	0.03	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C6	48.29	13.51	0.3	0.02	0.07	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C8	36.46	27.86	0.7	0.04	0.05	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C9	45.77	13.58	0.3	0.02	0.06	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C10	39.14	36.05	0.7	0.05	0.05	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C11	48.61	14.08	0.3	0.02	0.07	0.01	61.3	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C12	37.63	42.65	0.5	0.06	0.05	0.1	613	320	6400	613	16	3.05	4 ϕ 16	ϕ 8 c/c 320
2ND	C13	34.52	17.94	0.2	0.02	0.05	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C14	19.70	18.16	0.3	0.03	0.03	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C15	29.44	32.16	0.5	0.04	0.04	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C17	34.87	13.21	0.3	0.02	0.05	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C18	9.32	10.52	0.1	0.01	0.01	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C19	20.64	36.48	0.2	0.05	0.03	0.01	73.6	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C21	21.51	10.88	0.1	0.02	0.03	0.01	61.3	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C22	12.53	17.33	0.3	0.02	0.02	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C23	24.09	44.63	0.5	0.06	0.03	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C24	14.88	16.80	0.3	0.02	0.02	0.02	123	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C25	28.10	30.45	0.6	0.04	0.04	0.05	307	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C26	17.06	13.95	0.3	0.02	0.02	0	0	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320
2ND	C27	31.42	22.98	0.5	0.03	0.04	0.01	61.3	320	6400	320	16	1.59	4 ϕ 16	ϕ 8 c/c 320

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colmn	location	MED x	MED y	Vsd	μsd y	μsd x	ω	As	As, min	As, max	As, prov	φ	no of	longitudinal	shear reinforcement
1ST	C1	28.70	13.23	0.2	0.02	0.04	0.03	184	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C2	16.61	5.91	0.1	0.01	0.02	0.01	61.3	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C3	33.22	22.82	0.3	0.03	0.05	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C4	11.78	13.16	0.3	0.02	0.02	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C5	25.18	32.74	0.6	0.05	0.03	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C6	52.70	20.07	0.4	0.03	0.07	0.02	123	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C8	46.77	39.14	0.9	0.05	0.06	0.01	73.6	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C9	59.23	20.60	0.4	0.03	0.08	0.01	61.3	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C10	40.52	41.40	0.9	0.06	0.06	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C11	62.39	22.26	0.4	0.03	0.09	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C12	33.43	31.14	0.6	0.04	0.05	0.02	123	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C13	31.63	15.59	0.2	0.02	0.04	0.05	307	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C14	18.50	17.33	0.3	0.02	0.03	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C15	53.14	36.77	0.6	0.05	0.07	0.01	61.3	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C17	41.73	19.61	0.3	0.03	0.06	0.03	184	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C18	9.76	8.23	0.1	0.01	0.01	0.01	61.3	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C19	47.11	35.04	0.3	0.05	0.06	0.1	613	320	6400	613	16	3.05	4φ16	ø 10 c/c 320
1ST	C21	27.27	16.75	0.2	0.02	0.04	0.02	123	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C22	13.13	16.71	0.3	0.02	0.02	0.02	123	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C23	35.50	43.74	0.6	0.06	0.05	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C24	20.57	28.36	0.4	0.04	0.03	0.01	61.3	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C25	50.42	38.63	0.8	0.05	0.07	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
1ST	C26	16.76	15.68	0.4	0.02	0.02	0.1	613	320	6400	613	16	3.05	4φ16	ø 10 c/c 320
1ST	C27	53.41	30.19	0.7	0.04	0.07	0	0	320	6400	320	16	1.59	4φ16	ø 10 c/c 320
GR. FL	C1	28.91	19.99	0.2	0.01	0.02	0.15	1437	500	10000	1437	24	3.18	4φ24	ø 10 c/c 400
GR. FL	C2	13.05	12.44	0.1	0.01	0.01	0.01	115	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C3	38.78	38.10	0.2	0.03	0.03	0.01	95.8	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C4	23.97	19.73	0.2	0.01	0.02	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C5	43.47	43.49	0.5	0.03	0.03	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C6	59.69	37.35	0.3	0.03	0.04	0.02	192	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C8	69.52	72.54	0.7	0.05	0.05	0.05	479	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C9	42.54	26.60	0.3	0.02	0.03	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C10	69.49	75.88	0.7	0.05	0.05	0.01	95.8	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C11	43.27	27.69	0.3	0.02	0.03	0.03	287	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C12	48.90	51.23	0.4	0.04	0.03	0.01	95.8	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C13	29.76	27.01	0.2	0.02	0.02	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C14	20.88	28.75	0.2	0.02	0.01	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C15	58.81	62.44	0.5	0.04	0.04	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C17	48.12	34.66	0.3	0.02	0.03	0.02	192	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C18	11.44	16.51	0.1	0.01	0.01	0.01	115	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C19	46.56	51.76	0.2	0.04	0.03	0.01	95.8	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C21	34.01	29.43	0.1	0.02	0.02	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C22	19.87	27.52	0.2	0.02	0.01	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C23	55.85	48.72	0.5	0.03	0.04	0.02	192	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C24	25.21	29.58	0.3	0.02	0.02	0.05	479	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C25	65.17	67.91	0.6	0.05	0.05	0	0	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C26	23.16	21.73	0.3	0.02	0.02	0.01	95.8	500	10000	500	16	2.49	4φ16	ø 10 c/c 320
GR. FL	C27	63.03	63.29	0.5	0.04	0.04	0.03	287	500	10000	500	16	2.49	4φ16	ø 10 c/c 320

F-4 Biaxial chart

