



Civil Engineering Department

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Structural design of G+4 Hotel building Project

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5Wolkite University, Civil Engineering Department, ID Number ENGR700/09

6Wolkite University, Civil Engineering Department, ID Number ENGR 829/09

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Submission Date; 06/08/2021

Wolkite University, College of Engineering and Technology

Civil Engineering Department

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 **Belete Maschal Tarekegn, Dawud Shemsu Sherahe, Hanan Bedru Alemu, Kalkidan Molla Mekonnen, Nurhasen Mehamed Amin, Teha Umer Eba, Hermen Negash Biru,** entitled **Structural design of G+4 Hotel building Project,** 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 Hotel building project 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.

Acknowledgment

From the beginning to the end, we want to thank the almighty GOD for everlasting love and protection. Next for our general concern we would like to thank Department of Civil Engineering for facilitating & preparing this thesis project for our successive upgrading and continuous follow up with appreciable comments.

For the major succession of our project paper we would like to give our gratitude from the bottom of the heart to our advisor Mr. Bekele Tamru, who gave us valuable suggestion & consistent comments and also our thesis coordinator Mr. Kibret G. while advising & guiding us till the end of project, and we have received a great deal of assistance, encouragement from their consulting.

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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 tF
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
<i>Latin lower case letters</i>	
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_{tp}	Tensile strength of prestressing steel

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
f_{pk}	Characteristic tensile strength of prestressing steel
f_t	Tensile strength of reinforcement
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
v	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
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_n	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 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 3.00m each. The building area covers total of 401.5 m². all floor plans are typical excluding ground floor . plan. Ground floor is designed for Restaurant and Bank service. 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

1.1 Main objective

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

1.2 Sub-objectives

When we do this project, we will be able to improve our skills on

- ✓ To work and practice the general concepts of designing methodology
- ✓ Software related to structural design like ETABS, SAP, Auto Cad, MS-excel etc
- ✓ Reading and preparing detailed drawings of structure
- ✓ Knowing design code and design criteria
- ✓ To develop the habit of working together

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 is considered

The choice of the two procedures is based on the height of the building and the dynamic coefficient, C_d . 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 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
Location	Wolkite
Building height	15.0m
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
orthography factor	1: recommended
Turbulence factor	1: recommended
Air density	1.25 kg/m ³ : recommended

Reference height	14.4m : for $h < b$
Live load	0.4kN/m: recommended value for roof not accessible except for normal maintenance and repair

1.4. Wind load analysis

- ✓ Determination of basic wind velocity

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

Where

C_{dir} = directional factor

C_{sea} = seasonal factor

V_{bo} = fundamental basic wind velocity

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

- ✓ Roughness factor $C_r(Z)$ (Wind Action ES EN 1991:2015,2018, P 8)

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

Where

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

$$Z_0 = 0.05 \text{ m (terrain category II)}$$

Z_{min} = minimum height

Z_{max} = maximum height taken as 200m

$$K_r = 0.19 \left(\frac{0.03}{0.05}\right)^{0.07} = 0.215$$

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

$$= 0.215 \ln\left(\frac{15}{0.3}\right)$$

$$C_r(Z) = 0.841$$

- ✓ 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

$$V_m(Z) = 0.841 * 1 * 22 \text{ m/s} = \mathbf{18.50 \text{ m/s}}$$

✓ Wind turbulence $I_v(Z)$

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

Where K_1 = turbulence factor

$C_0(Z)$ = orography factor

$$I_v(Z) = \frac{1}{1 + \ln\left(\frac{15}{0.3}\right)} = 0.257$$

➤ Peak velocity pressure $q_p(Z)$

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

where ρ = air density

$$Q_p(Z) = [1 + 7 * 0.257] * \frac{1}{2} * 1.25 * 18.50^2 = \mathbf{598.72N = 0.598KN}$$

➤ 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

Table 2.2 external pressure coefficients for flat roofs with mansard eaves angle of 60°

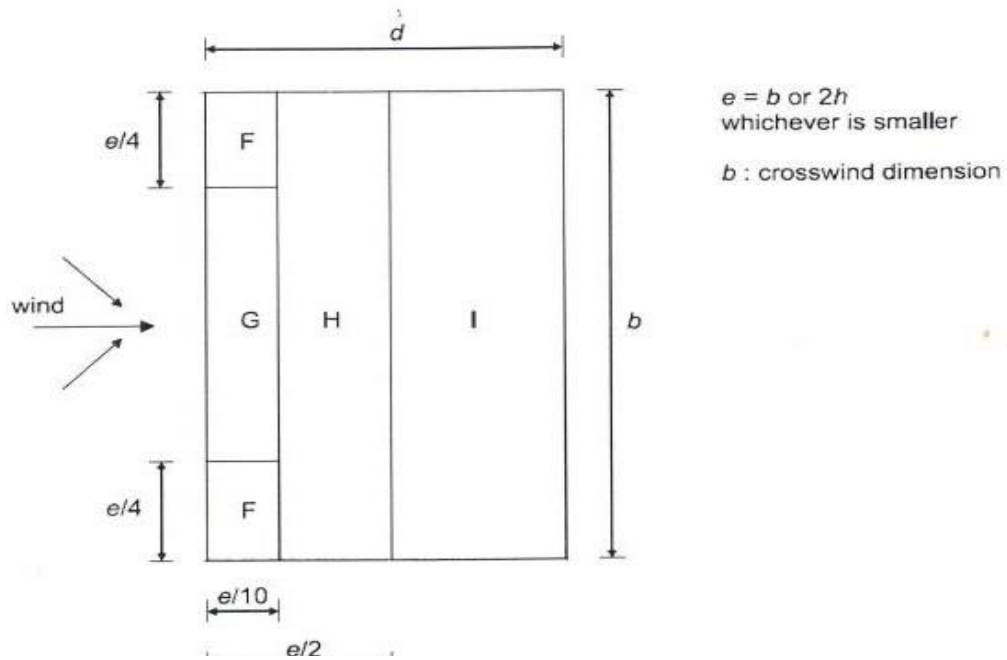
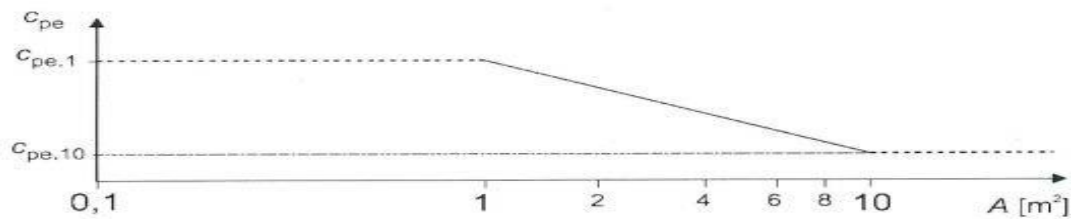


Figure 2.1 key for flat roofs

Roof type	Zone							
	F		G		H		I	
	C _{pe,10}	C _{pe,1}	C _{pe,10}	C _{pe,1}	C _{pe,10}	C _{pe,1}	C _{pe,10}	C _{pe,1}
Sharp Eaves	-1.8	-2.5	-1.2	-2.0	-0.7	-1.2	+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 1m^2 and 10m^2

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

-1 $\Theta = 90^\circ$

$b = 22.40$ $d = 18.25\text{m}$, $h = 15\text{m}$

$$e = \min \left\{ \begin{array}{l} b = 22.40 \\ 2h = 2 * 15 = 30 \end{array} \right. \quad e = 22.40 \text{ m}$$

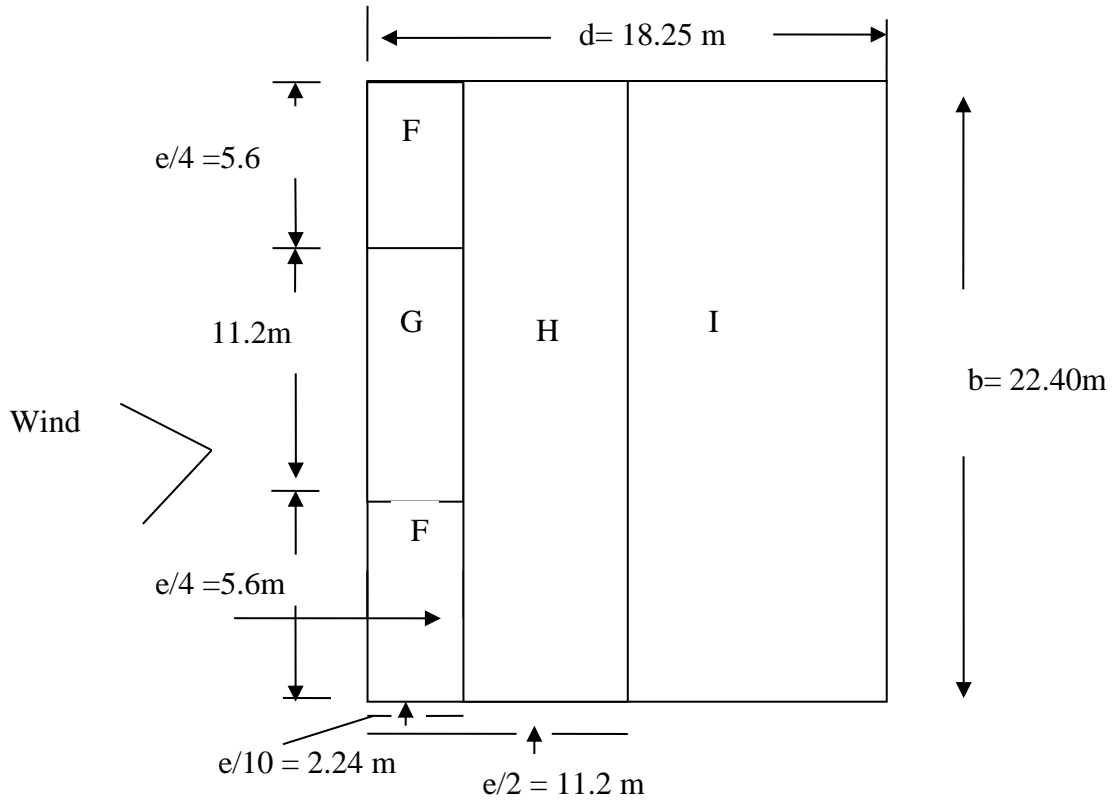


Figure 2.3 Pressure Distribution zones $\Theta = 90^\circ$

Table 2.3 External wind pressure $\Theta = 90^\circ$

zone	Cpe values		cpe	$W_e = q_p(Z) \cdot C_{pe}$
F	cpe,10	-1.8	-1.8	-1.08
	cpe,1	-2.5		
G	cpe,10	-1.2	-1.2	-0.718
	cpe,1	-2		
H	cpe,10	-0.7	-0.70	-0.419
	cpe,1	-1.2		
I	cpe,10	0.2	0.20	0.120
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.120
	cpe,1	0.2		

Critical external pressure; - pressure = **0.120** and suction = **-1.08**

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

$b = 18.25\text{m}$, $d = 22.40\text{m}$, $h=15$

$$e = \min \left\{ \begin{array}{l} b = 18.25 \\ 2h = 2 * 15 = 30 \end{array} \right. \quad e = 18.25 \text{ m}$$

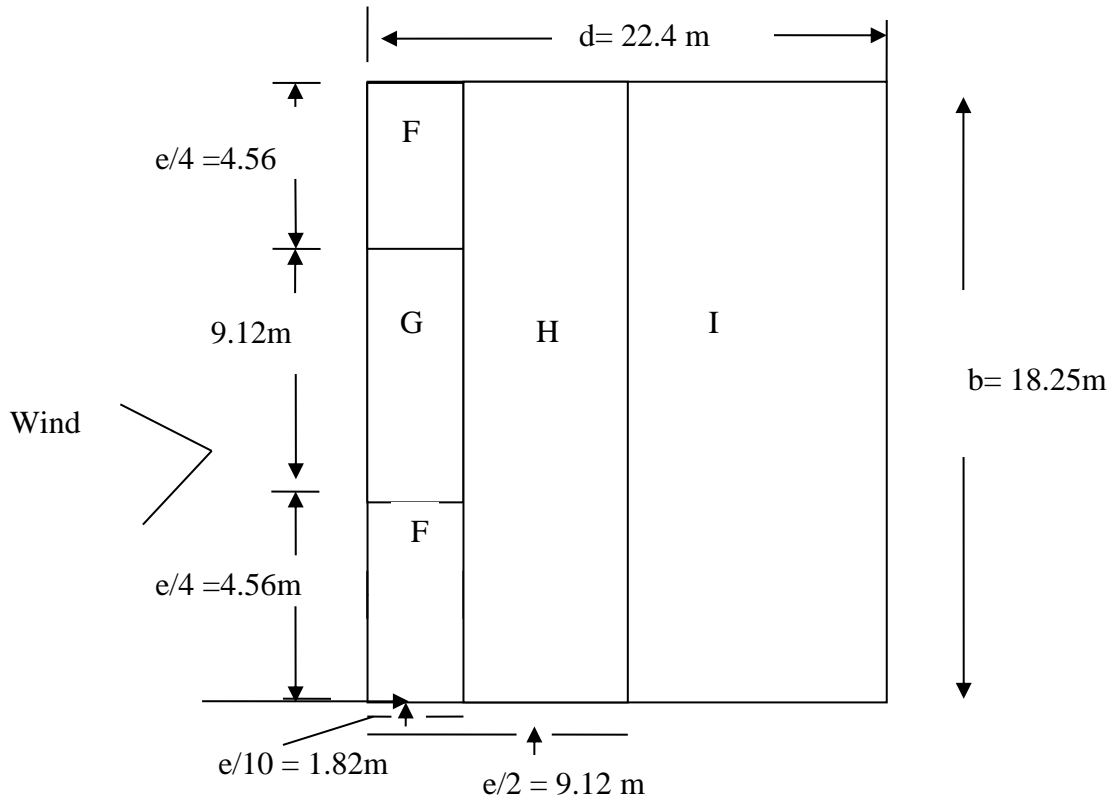


Figure 2.4 Pressure Distribution zones $\Theta=0^\circ$

Table 2.4 External wind pressure $\Theta=0^\circ$

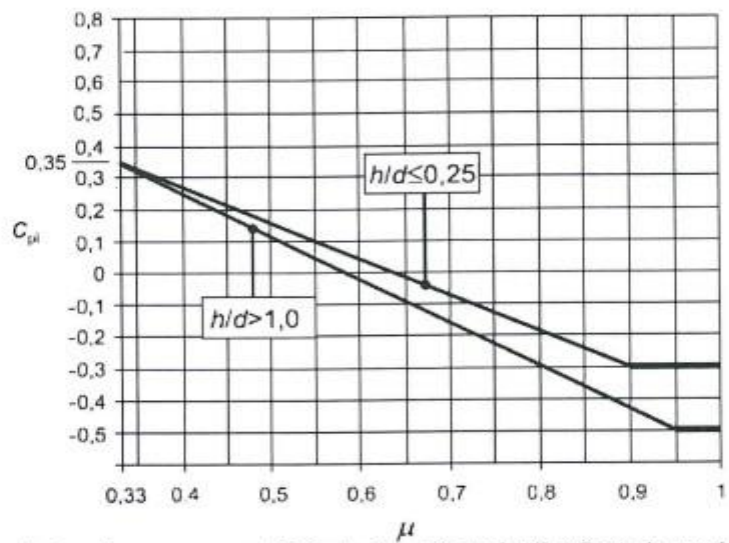
zone	Cpe values		cpe	w_e
F	cpe,10	-1.8	-1.86	-1.11
	cpe,1	-2.5		
G	cpe,10	-1.2	-1.20	-0.718
	cpe,1	-2		
H	cpe,10	-0.7	-0.70	-0.419
	cpe,1	-1.2		
I	cpe,10	0.2	0.20	0.120
	cpe,1	-0.2		
	cpe,10	-0.2	-0.20	-0.120
	cpe,1	0.2		

Critical external pressures: - pressure = **0.120** and suction = **-1.11**

- Internal pressure

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

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

Take C_{pi} 0.8 for pressure and $C_{pi} = -0.5$ for suction

$$W_i = 0.598 * 0.8 = 0.4784$$

$$= 0.598 * -0.5 = -0.299$$

➤ Net pressure = $W_e - W_i$

W_e	W_i	W_{net}
0.120	0.4784	-0.3584
-1.08	-0.299	-0.781
0.120	0.4784	-0.3584
-1.11	-0.299	-0.811

$$W_{net \text{ min}} = -0.3584$$

$$W_{net \text{ max}} = -0.811$$

2.1 Concrete cover design for roof slab

- 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 (Appendix B, Table B1).
- Minimum concrete cover for bond/durability = 12 mm

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

$C_{min, b} = 12$ mm, assumed diameter of bar (Appendix B, Table B4). $C_{min, dur} = 10$ 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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

- Nominal cover = 22mm

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

$\Delta c_{dev} = 10\text{mm}$, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.(ES EN 1992-1-1:2015, -)(4.4.1.3,1)

- Minimum cover Design for Fire = 20mm ((Endris Setegn, et al, 2020) (Table C1)
- Governing concrete cover = 25mm

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

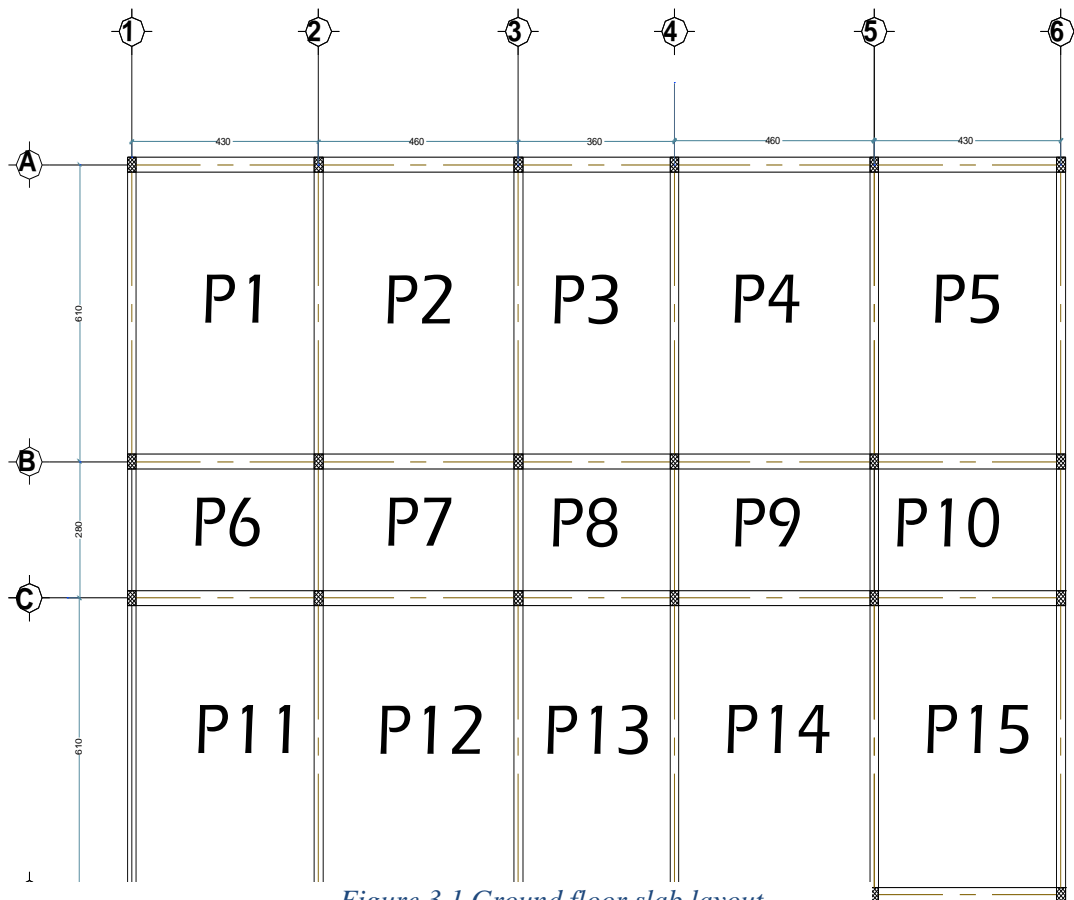


Figure 3.1 Ground floor slab layout

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.(ES EN 1992-1-1:2015, -)
- b. Minimum strength class = C20/25, for exposure class XC1 (Appendix B, Table B1).
- c. Minimum concrete cover for bond/durability = 12 mm

$$C_{min} = \max \begin{pmatrix} C_{min, B} \\ C_{min, Dur} \\ 10 \text{ mm} \end{pmatrix} = \max \begin{pmatrix} 12 \text{ mm} \\ 10 \text{ mm} \\ 10 \text{ mm} \end{pmatrix} = 12$$

$C_{min,b} = 12$ mm, assumed diameter of bar (Appendix B, Table B4). $C_{min,dur} = 10$ 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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

- d. Nominal cover = 22mm

$$C_{nom} = C_{min} + \delta_{cdev} = 12\text{mm} + 10\text{mm} = 22\text{mm}$$

$\Delta_{cdev} = 10\text{mm}$, the value of Δ_{cdev} for use in a Country may be found in its National Annex. The recommended value is 10 mm. (ES EN 1992-1-1:2015, -)(4.4.1.3,1)

- e. Minimum cover Design for Fire = 20mm

- f. Governing concrete cover = 22mm

3.1.2 Depth determination

We use C20/25 concrete and S400 rebar

Serviceability requirement

$$\frac{l}{d} = N * K * F1 * F2 * F3 \text{ (Yohannes, 2021)}$$

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

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

$$F = \text{factor of safety } F1 = \frac{500}{f_{yk}} = 1.25$$

$$F2 = 1, F3 = 1$$

$$\frac{l}{d} = N * K * F1 * F2 * F3 = 1.3 * 17.71 * 1.25 * 1 * 1 = 28.78 \text{ for end span}$$

Depth, $D = d + d'$ where $d' = \text{cover} + \frac{\phi_{lon}}{2}$ use $\phi_{lon} = 10\text{mm}$

$$d = 22 + \frac{10}{2} = 27\text{mm}$$

$$\frac{l}{d} = 28.78$$

$$L = lx = 4300, \text{ for span 1}$$

$$d = \frac{4300}{28.78} = 149.41$$

Similar steps for the rest panels will be calculated as follow

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	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.41
P-2	6100	4600	1.33	two way	end span	1.3	17.71	28.78	159.83
P-3	6100	3600	1.69	two way	end span	1.3	17.71	28.78	125.09
P-4	6100	4600	1.33	two way	end span	1.3	17.71	28.78	159.83
P-5	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.41
P-6	4300	2800	1.54	two way	end span	1.3	17.71	28.78	97.29
P-7	4600	2800	1.64	two way	continuous	1.5	17.71	33.21	84.31
P-8	3600	2800	1.29	two way	continuous	1.5	17.71	33.21	84.31
P-9	4600	2800	1.64	two way	continuous	1.5	17.71	33.21	84.31
P-10	4300	2800	1.54	two way	end span	1.3	17.71	28.78	97.29
P-11	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.41
P-12	6100	4600	1.33	two way	end span	1.3	17.71	28.78	159.83
P-13	6100	3600	1.69	two way	end span	1.3	17.71	28.78	125.09
P-14	6100	4600	1.33	two way	end span	1.3	17.71	28.78	159.83
P15	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.41
Governing panel depth(mm)									159.83
clear cover(mm)									25mm
Assuming reinforcements ϕ									10mm
D(mm)=d+c+(ϕ /2)									189.833
use D minimum									200mm

3.1.3 Reinforcement

➤ Area of steel

$$max = \max \left\{ \begin{array}{l} 0.26 \left(\frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left(\frac{2.2}{400} \right) 1000 * 200 = 286 \\ 0.0013 * 1000 * 200 = 260 \end{array} \right.$$

$$A_{st, \min} = 286 \text{ mm}^2$$

➤ Spacing

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

Let the ϕ of reinforcement be 12mm

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

$$\text{Spacing of bars } S = \frac{b a_s}{A_s} = \frac{1000 * 113.04}{286} = 395.25 \text{ mm}$$

395.25 mm \approx 390 mm used < 600mm OK!! So ϕ 12 c/c 390 used for whole ground floor slab.

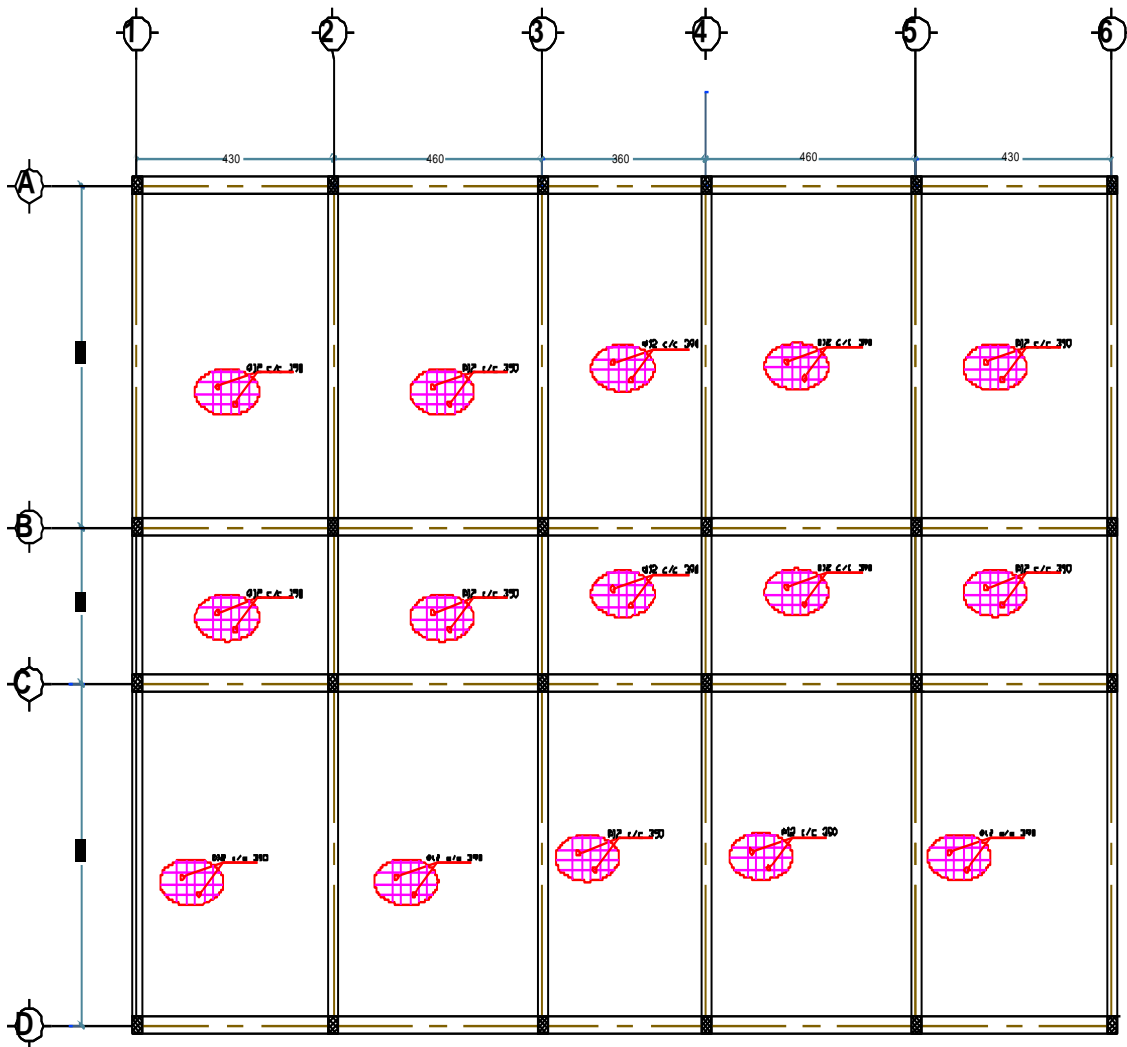


Figure 3.2 Ground Floor Reinforcement Detailing

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

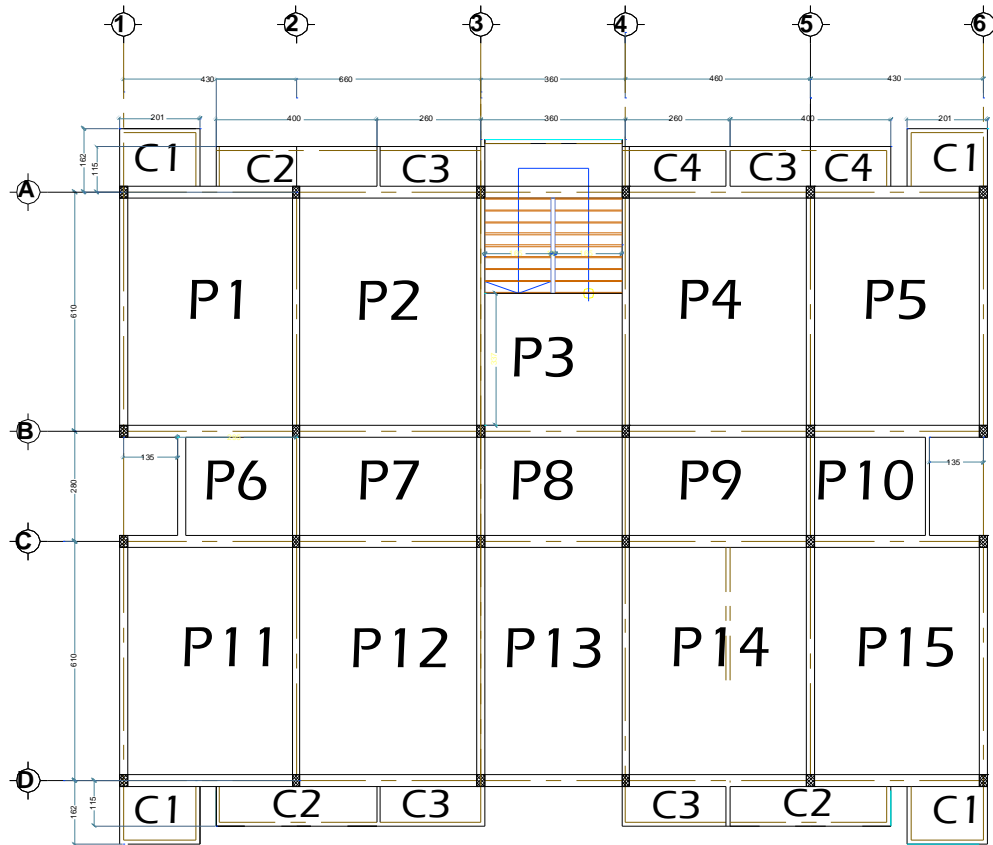


Figure 3.3 floor slab layout for typical 1st -4th floor

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 whererelevant) 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

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

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 is 12 mm

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

$C_{min, b} = 12$ mm, assumed diameter of bar (Appendix B, Table B4). $C_{min, dur} = 10$ 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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

d. 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.

e. Minimum cover Design for Fire = 20mm

f. Governing concrete cover = 22mm

3.2.2 Depth determination

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

$$\frac{l}{d} = N * K * F1 * F2 * F3 \text{ (Yohannes, 2021)}$$

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

F=factor of safety $F1 = \frac{500}{f_{yk}} = 1.25$

$$F2 = 1, F3 = 1$$

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

Table 3.2 Depth determination of typical 1st -4th floor

Panel	ly (mm)	lx (mm)	ly/lx	Type	structural system	k	N	lx/d	d
P-1	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.42
P-2	6100	4600	1.33	two way	continuous	1.5	17.71	33.21	138.53
P-3	3600	3370	1.07	two way	continuous	1.5	17.71	33.21	101.49
P-4	6100	4600	1.33	two way	continuous	1.5	17.71	33.21	138.53
P-5	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.42
P-6	2950	2800	1.05	two way	end span	1.3	17.71	28.78	97.29
P-7	4600	2800	1.64	two way	continuous	1.5	17.71	33.21	84.32
P-8	3600	2800	1.29	two way	continuous	1.5	17.71	33.21	84.32
P-9	4600	2800	1.64	two way	continuous	1.5	17.71	33.21	84.32
P-10	2950	2800	1.05	two way	end span	1.3	17.71	28.78	97.29
P-11	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.42
P-12	6100	4600	1.33	two way	continuous	1.5	17.71	33.21	138.53
P-13	6100	3600	1.69	two way	end span	1.3	17.71	28.78	125.09
P-14	6100	4600	1.33	two way	continuous	1.5	17.71	33.21	138.53
P15	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.42
C1	200	162	1.23	two way	Cantilever	0.4	17.71	8.86	18.29
C2	400	115	3.48	one way	Cantilever	0.4	17.71	8.86	12.99
C3	260	115	2.26	one way	Cantilever	0.4	17.71	8.86	12.99
C4	200	115	1.74	one way	Cantilever	0.4	17.71	8.86	12.99
Governing panel depth(mm)									149.42
clear cover(mm)									25mm
Assuming reinforcements ϕ									10mm
$D(mm)=d+c+(\phi/2)$									179.41
use D minimum									190mm

3.2.3 Determination of loading

When we determine the design load we use the load combination $P_d = 1.35G_k + 1.5Q_k$

The imposed or live loads on the floor based on its specific use is derived from (ES EN1992-1-1, 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
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
Marble tile	27

For typical 1st -4th floor

P-1 & P-11

Panel 1 and 4 has 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

$$\text{Self-weight of slab} = \text{thickness of slab} \times \text{unit weight} = 0.19\text{m} \times 25\text{KN/m}^3 = 4.75\text{KN/m}^2$$

Floor finish = thickness * unit weight

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

$$\text{Cement screed} = 0.03\text{m} * 23\text{KN/m}^3 = \mathbf{0.69\text{KN/m}^2}$$

Ceiling plaster = thickness * unit weight

$$\text{Ceiling plaster} = 0.03\text{m} * 23\text{KN/m}^3 = \mathbf{0.69\text{KN/m}^2}$$

$$\text{Floor finish} = 0.02\text{m} * 27\text{KN/m}^3 = \mathbf{0.54\text{KN/m}^2}$$

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

$$A=L*W= 6.1\text{m}*4.3\text{m} = 26.23\text{m}^2$$

2. Calculate volume of partition wall

$$V=L*W*H= 4.7\text{m}*0.15\text{m}*2.81\text{m} = 1.981\text{m}^3$$

3. Calculate load

$$\begin{aligned} \text{DL} &= \text{unit weight of HCB} * V/A = 14 \\ \text{KN/m}^3 * 1.981\text{m}^3 / 26.23\text{m}^2 &= \mathbf{1.057\text{KN/m}^2} \end{aligned}$$

4. Calculate volume of plastering

$$V=L*W*H= 4.7\text{m}*0.02\text{m}*2.81\text{m} = 0.264\text{m}^3$$

5. Calculate load

$$\begin{aligned} \text{DL} &= \text{unit weight of HCB} * V/A = 23 \\ \text{KN/m}^3 * 0.264\text{m}^3 / 26.23\text{m}^2 &= \mathbf{0.232\text{KN/m}^2} \end{aligned}$$

$$\begin{aligned} \text{Total DL} &= 4.75\text{KN/m}^2 + 0.69\text{KN/m}^2 + 0.69\text{KN/m}^2 + 0.54\text{KN/m}^2 + 1.057\text{KN/m}^2 + 0.232\text{KN/m}^2 \\ &= \mathbf{7.96\text{KN/m}^2} \end{aligned}$$

Live load: - live load is selected with respect to functional use of panel for this panel $LL=2\text{KN/m}^2$

$$PD = 1.35 * 7.96\text{KN/m}^2 + 1.5 * 2\text{KN/m}^2 = \mathbf{13.745\text{KN/m}^2}$$

Panel -1 and Panel 11					Material	Unit weight	Dimension	Load
								KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					Marble tiling	23	0.03	0.69
Floor finishing					Ceramic	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
L	t	h	l	w	HCB	14	0.076	1.057
4.7	0.15	2.81	6.1	4.3				
Dimension of hcb wall plastering					Concrete	23	0.010	0.232
L	t	h						
4.7	0.02	2.81						
Partition wall Volume (l*t*h)				1.98				
Partition wall plastering Volume (l*t*h)				0.2641				
Area of slab (l*w)				26.23				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					7.959	KN/m ²		
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					13.745	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 based 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 be analyzed by coefficient method.

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

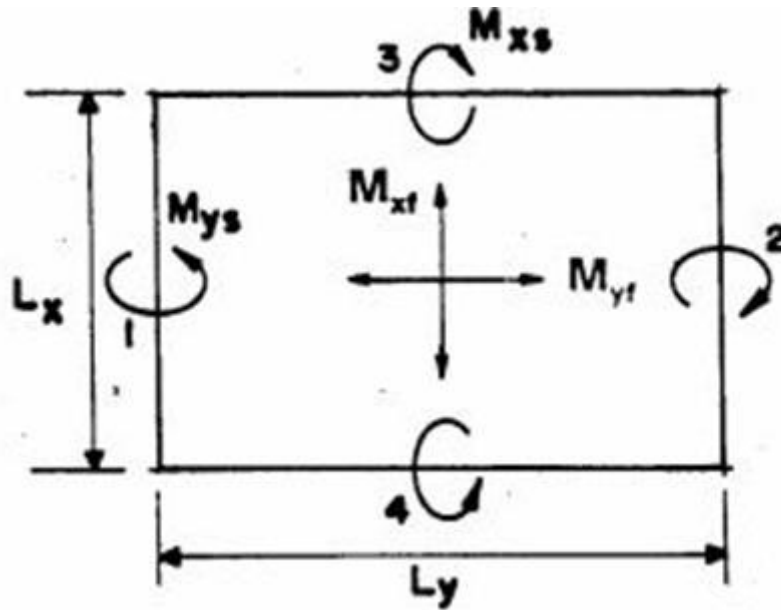


Figure 3-3 Moment direction on panels

Sample calculation

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

$$M_i = 0.0689 * 13.748 * 4.3^2 = 17.52$$

$$M_i = 0.0517 * 13.748 * 4.3^2 = 13.15$$

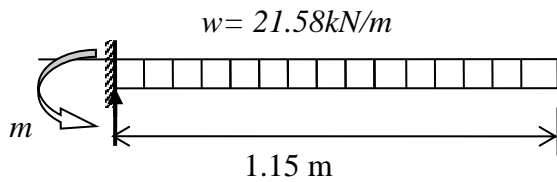
$$M_i = 0.039 * 13.748 * 4.3^2 = 9.91$$

$$M_i = 0.030 * 13.748 * 4.3^2 = 7.63$$

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

Moment analysis for cantilever slabs

C1

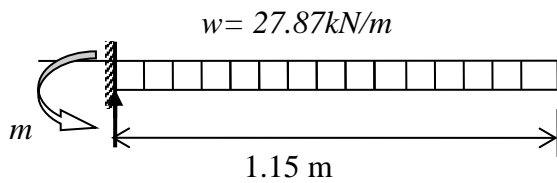


Mxs =

$$WL^2/2 = 21.58 * 1.62^2 / 2 = 28.32 \text{ KNM}$$

$$RA=WL=21.58*1.62=34.96 \text{ KN}$$

C2

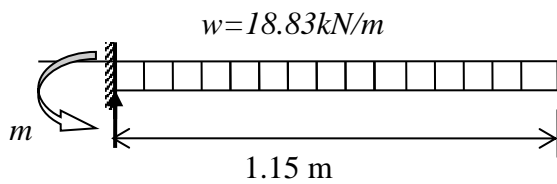


Mxs =

$$WL^2/2 = 27.87 * 1.5^2 / 2 = 18.43 \text{ KNM}$$

$$RA=WL=27.87*1.15=32.05 \text{ KN}$$

C3

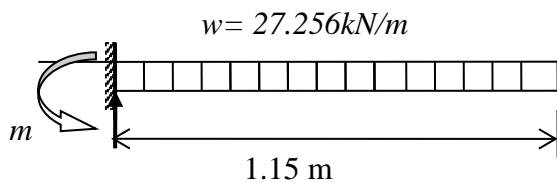


Mxs =

$$WL^2/2 = 18.83 * 1.5^2 / 2 = 12.45 \text{ KNM}$$

$$RA=WL=18.83*1.15=21.65 \text{ KN}$$

C4



Mxs =

$$WL^2/2 = 27.256 * 1.5^2 / 2 = 18.02 \text{ KNM}$$

$$RA=WL=27.256*1.15=31.34 \text{ KN}$$

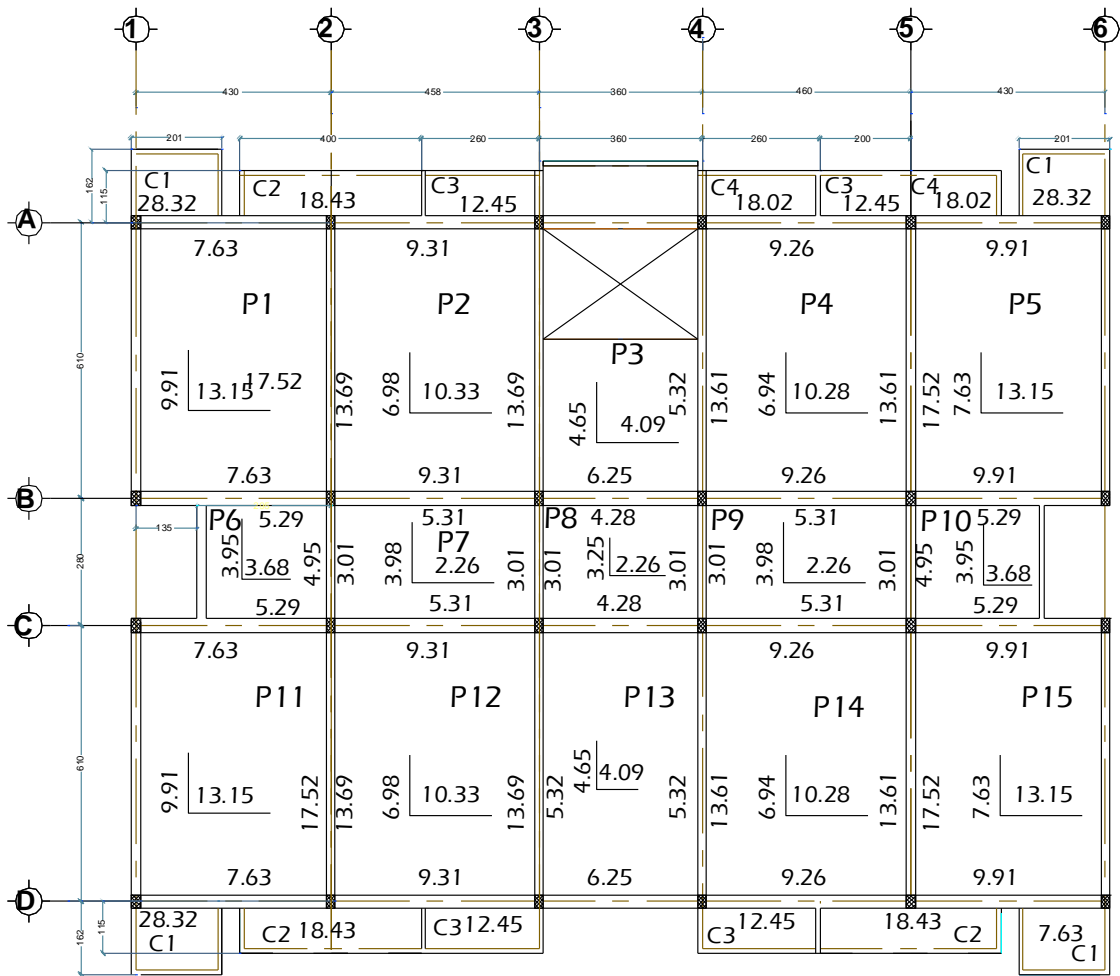


Figure 3.4 Unadjusted Moment for typical 1st -4th floor

Support moment adjustment

Two methods of moment adjustment are used

There are two cases (on EBCS-2015)

Two methods of moment adjustments are used.

Methods (1) if $\Delta N < 20\%$ (M large) we use average method.

Methods (2) if $\Delta N > 20\%$ (M large) we use moment distribution method based on relative stiffness.

- If M_s is decreased, then field moments are increased to allow equilibrium compatibility.
- If M_s is increased, no field moment adjustment.

- 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.

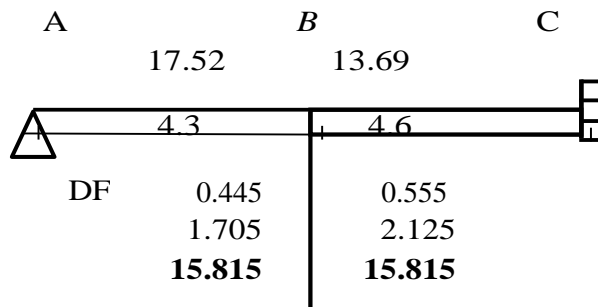
✓ Sample calculation

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

$$\Delta M_s = 17.52 - 13.69 = 3.83$$

$$20\% M_{large} = 0.2 * 17.52 = 3.50$$

Moment distribution method required



$$K_{AB} = \frac{I}{L}, \frac{3I}{4 * 4.3} = 0.174 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.23256}{0.23256 + 0.2174} = 0.445 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.555$$

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

–

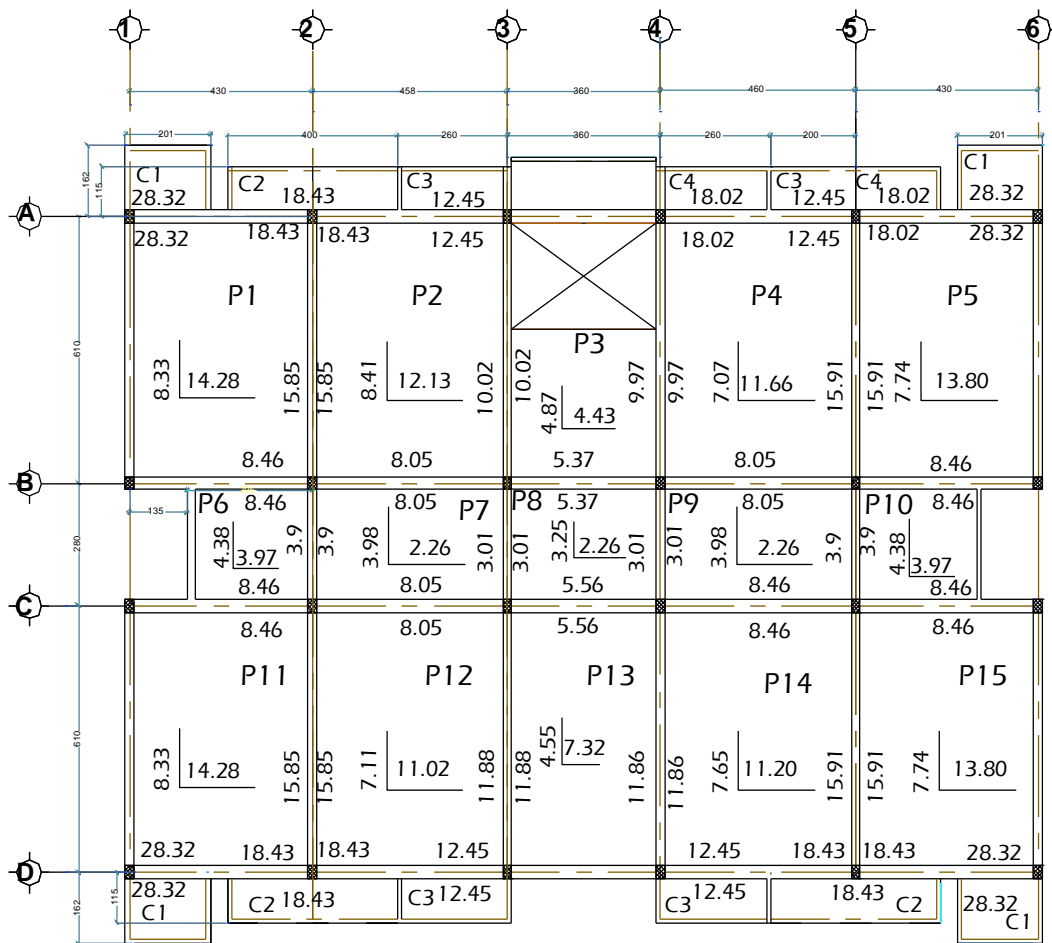


Figure 3.5 Adjusted moment for typical 1st -4th floor

3.2.5 Load transfer to beam Two-way slab shear

The shear force for each panel calculated by coefficient method

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

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

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

Sample calculation for P-1

$$\beta_{vxc} = 0.493 \quad \beta_{vxd} = 0.34$$

$$\beta_{vyc} = 0.36 \quad \beta_{vyd} = 0$$

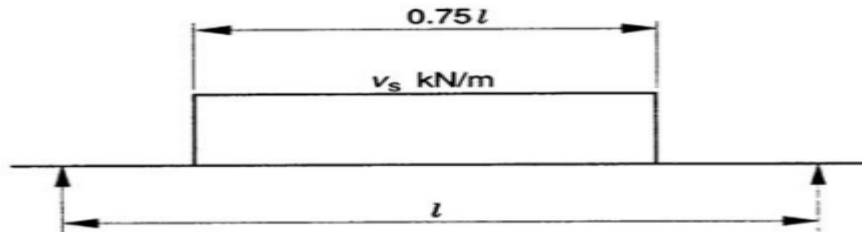
$$V_{xc} = \beta_{vxc} PdL_x = 0.493 * 13.745 * 4.3 = 29.1$$

$$V_{xd} = \beta_{vxd} PdL_x = 0.34 * 13.745 * 4.3 = 20.1$$

$$V_{yc} = \beta_{vyc} PdL_y = 0.36 * 13.745 * 4.3 = 21.3$$

$$V_{yd} = \beta_{vyd} PdL_y = 0$$

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-2]

3.2.6 Check for shear strength

$$VR_{dc} = [0.12K(100\rho_1f_{ck})^{(1/3)}]bd \geq V_{min}$$

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

$$d = 150$$

$$K = 2.15 \quad \text{not} < 2, \text{ so } k=2$$

$$\rho_1 = \frac{Asl}{bd} \leq 0.02$$

$$VR_{dc} = [0.12 * 2(100 * 0.0015 * 20)^{(1/3)}] = 0.346$$

Taking minimum reinforcement $\phi 12 @ 430$

$$\rho_1 = 0.0015$$

$$V_{min} = [0.035 (K)^{(3/2)} (f_{ck})^{(1/2)}] = 0.443$$

$$VR_{dc} = 0.346 \leq V_{min} = 0.443 \text{ Then } VR_{dc} = 0.443$$

Maximum shear force transfer to beam is $V_i = 29.2 \text{ kN/m}$

$$V_i = 29.2 \text{KN/m} = 0.0292 \text{KN/mm}$$

0.443 > 0.0292, the slab thickness is sufficient to support the design load (ES EN 1992-1-1:2015, -)

3.2.7 Reinforcement Calculation for 1st to 4th floor slab

$$\mu_{sd} = \frac{msd}{f_{cd} * b * d} \quad A_{st} = \frac{msd}{Z * f_{yd}}$$

$$A_{st \min} = \max \left\{ 0.26 \left(\frac{f_{ctm}}{f_{yk}} \right) bd \quad S = \frac{bas}{A_{st}} \quad S \max = \max \left(\frac{3D}{400mm} \right) \right\} \quad (\text{Yohannes, 2021})$$

$$A_{st \min} = \max \left\{ 0.26 \left(\frac{2.2}{400} \right) * 1000 * 149 = 213.07 \right. \\ \left. 0.0013 * 1000 * 149 = 193.7 \right.$$

$$A_{st \min} = 214 \text{mm}^2$$

$$A_{st \max} = 0.04bd = 0.04 * 1000 * 149 = 5960 \text{mm}^2$$

$$A_{st} = \frac{msd}{Z * f_{yd}}, \quad F_{yd} = \frac{F_{yk}}{1.5} = \frac{400}{1.5} = 347.83, \quad Z = 0.9d = 0.9 * 149 = 134.1 \text{mm}$$

Panel 1 support moment 15.85KN-m

$$A_{st} = \frac{15.85 * 1000 * 1000}{134.1 * 347.83} = 339.81 \text{mm}^2 > 214 \text{mm}^2 \dots\dots\dots \text{OK!}$$

$$S \max = \max \left(\frac{3D}{400mm} \right) = \frac{3 * 190 = 570 \text{mm}}{400mm} \quad \text{Then } S \max = 570 \text{mm}$$

Let the ϕ of reinforcement be 12mm

$$ast = \frac{\pi d^2}{4} = \frac{3.1412^2}{4} = 113.04 \text{mm}^2$$

$$S = \frac{bas}{A_{st}} = \frac{1000 * 113.04}{339.81} = 332.66 \text{mm}$$

$$332.66 \text{ mm} \approx 320 \text{mm used} < 570 \text{mm} \text{OK!}$$

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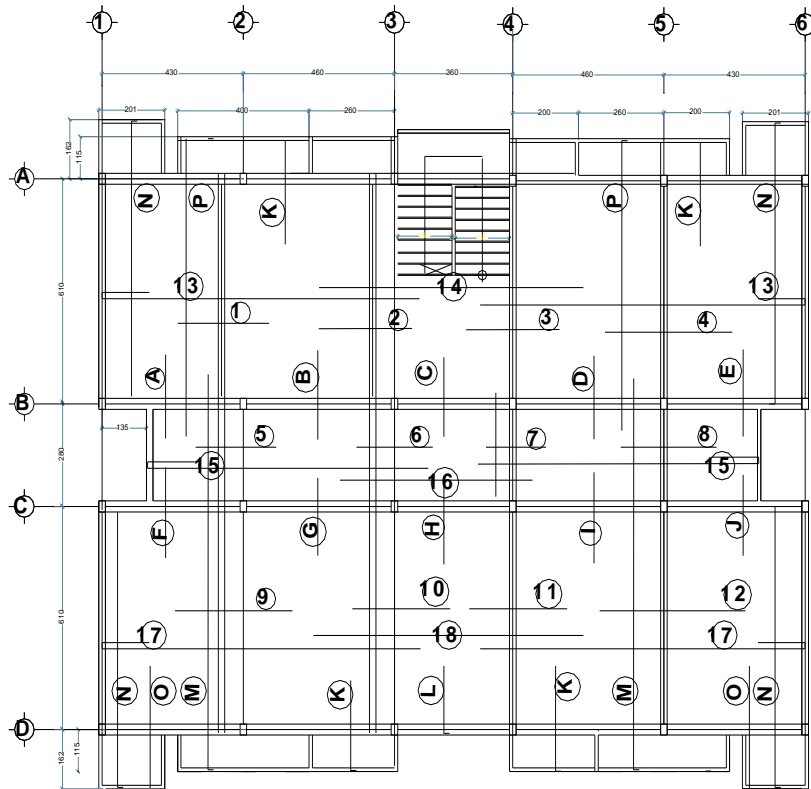
Table 3.5 Reinforcement Detailing

Panel	Depth	Moment		Ast=Msd /Z*f _{yd}	Ø	Spacing	Spacing used	Reinforcement		
		Location	Value	Ast						
P1	149.4 2	M _{xs} =	17.52	374.566	10	209.576	200	10	c/c	200
		M _{xf} =	14.3	305.297	10	257.127	250	10	c/c	250
		M _{ys} =	8.46	180.869	10	434.016	400	10	c/c	400
		M _{yf} =	8.33	178.090	10	440.789	400	10	c/c	400
P2	138.5 3	M _{xs} =	15.85	365.495	10	214.777	200	10	c/c	200
		M _{xf} =	12.13	279.713	10	280.644	250	10	c/c	250
		M _{ys} =	8.05	185.630	10	422.884	400	10	c/c	400
		M _{yf} =	8.41	193.932	10	404.782	400	10	c/c	400
P3	101.4 9	M _{xs} =	10.02	315.390	10	248.898	200	10	c/c	200
		M _{xf} =	4.43	139.439	10	562.970	500	10	c/c	500
		M _{ys} =	5.37	169.026	10	464.424	450	10	c/c	450
		M _{yf} =	4.81	151.400	10	518.495	500	10	c/c	500
P4	138.5 3	M _{xs} =	9.97	229.905	10	341.446	300	10	c/c	300
		M _{xf} =	11.66	268.875	10	291.957	250	10	c/c	250
		M _{ys} =	8.05	185.630	10	422.884	400	10	c/c	400
		M _{yf} =	7.07	163.032	10	481.502	450	10	c/c	450
P5	149.4 2	M _{xs} =	15.91	340.145	10	230.784	200	10	c/c	200
		M _{xf} =	13.80	295.035	10	266.070	250	10	c/c	250
		M _{ys} =	8.46	180.869	10	434.016	400	10	c/c	400
		M _{yf} =	7.74	165.476	10	474.389	450	10	c/c	450
P6	97.29	M _{xs} =	4.95	162.521	10	483.014	450	10	c/c	450
		M _{xf} =	3.97	130.345	10	602.247	550	10	c/c	550
		M _{ys} =	6.89	226.216	10	347.013	500	10	c/c	500
		M _{yf} =	4.38	143.806	10	545.872	500	10	c/c	500
P7	84.32	M _{xs} =	3.90	147.746	10	531.316	500	10	c/c	500
		M _{xf} =	2.26	85.617	10	916.873	550	10	c/c	550
		M _{ys} =	8.05	304.964	10	257.408	250	10	c/c	250
		M _{yf} =	3.97	150.398	10	521.948	500	10	c/c	500
P8	84.32	M _{xs} =	3.01	114.030	10	688.416	550	10	c/c	550
		M _{xf} =	2.26	85.617	10	916.873	550	10	c/c	550
		M _{ys} =	5.56	210.633	10	372.686	350	10	c/c	350
		M _{yf} =	3.25	123.122	10	637.579	550	10	c/c	550
P9	84.32	M _{xs} =	3.01	114.030	10	688.416	550	10	c/c	550
		M _{xf} =	2.26	85.617	10	916.873	550	10	c/c	550
		M _{ys} =	8.46	320.496	10	244.933	200	10	c/c	200

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		Myf =	3.98	150.777	10	520.636	500	10	c/c	500
P10	97.29	Mxs =	3.90	128.047	10	613.057	550	10	c/c	550
		Mxf =	3.97	130.345	10	602.247	550	10	c/c	550
		Mys =	8.46	277.763	10	282.615	250	10	c/c	250
		Myf =	4.38	143.806	10	545.872	500	10	c/c	500
P11	149.4 2	Mxs =	17.52	374.566	10	209.576	200	10	c/c	200
		Mxf =	14.3	305.297	10	257.127	250	10	c/c	250
		Mys =	8.46	180.869	10	434.016	400	10	c/c	400
		Myf =	8.33	178.090	10	440.789	400	10	c/c	400
P12	138.5 3	Mxs =	15.85	365.495	10	214.777	200	10	c/c	200
		Mxf =	11.02	254.117	10	308.913	300	10	c/c	300
		Mys =	8.05	185.630	10	422.884	400	10	c/c	400
		Myf =	7.11	163.954	10	478.793	450	10	c/c	450
P13	125.0 9	Mxs =	11.88	303.373	10	258.758	250	10	c/c	250
		Mxf =	7.32	186.927	10	419.951	400	10	c/c	400
		Mys =	5.56	141.982	10	552.885	500	10	c/c	500
		Myf =	4.55	116.191	10	675.614	550	10	c/c	550
P14	138.5 3	Mxs =	11.86	273.487	10	287.033	250	10	c/c	250
		Mxf =	11.20	258.268	10	303.948	300	10	c/c	300
		Mys =	8.46	195.085	10	402.390	400	10	c/c	400
		Myf =	7.65	176.406	10	444.996	400	10	c/c	400
P15	149.4 2	Mxs =	15.91	340.145	10	230.784	200	10	c/c	200
		Mxf =	13.80	295.035	10	266.070	250	10	c/c	250
		Mys =	8.46	180.869	10	434.016	400	10	c/c	400
		Myf =	7.74	165.476	10	474.389	450	10	c/c	450
C1	18.29	Mxs =	28.32	494.429	10	158.769	150	10	c/c	150
C2	12.99	Mxs =	18.43	453.287	10	173.179	170	10	c/c	170
C3	12.99	Mxs =	12.45	306.209	10	256.361	250	10	c/c	250
C4	12.99	Mxs =	18.02	443.203	10	177.120	170	10	c/c	170

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



		Reinforcement						Reinforcement			
A	L=	2967	10	c/c	400	1	L=	2966	10	c/c	200
B	L=	2967	10	c/c	400	2	L=	2733	10	c/c	200
C	L=	2967	10	c/c	450	3	L=	2733	10	c/c	300
D	L=	2967	10	c/c	400	4	L=	2966	10	c/c	200
E	L=	2967	10	c/c	400	5	L=	2516	10	c/c	450
F	L=	2967	10	c/c	400	6	L=	2733	10	c/c	500
G	L=	2967	10	c/c	400	7	L=	2733	10	c/c	550
H	L=	2967	10	c/c	450	8	L=	2516	10	c/c	550
I	L=	2967	10	c/c	400	9	L=	2966	10	c/c	200
J	L=	2967	10	c/c	400	10	L=	2733	10	c/c	200
K	L=	3183	10	c/c	170	11	L=	2733	10	c/c	250
L	L=	2033	10	c/c	500	12	L=	2966	10	c/c	250
M	L=	10420	10	c/c	400	13	L=	11533	10	c/c	250
N	L=	7420	10	c/c	400	14	L=	6667	10	c/c	250
O	L=	3653	10	c/c	150	15	L=	9733	10	c/c	550
P	L=	8353	10	c/c	450	16	L=	6667	10	c/c	550
						17	L=	11533	10	c/c	300
						18	L=	6667	10	c/c	250

3.2.8 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 lapss

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

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

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

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

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

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

$\eta_1 = 1$ for good quality of bond

$\eta_1 = 0.7$ for other cases

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

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

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

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

$f_{ctk, 0.05} = 1.5$ for C20/25 mpa, from ESEN 1992

Sample calculation for $\phi 10$ reinforcement bar

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

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

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

$\eta_1 = 0.7$ for other cases

$$f_{bd} = 2.25 \cdot 1 \cdot 1 \cdot 0.85 = 1.91$$

$$\sigma_{sd} = \frac{400}{1.15} = 347.83$$

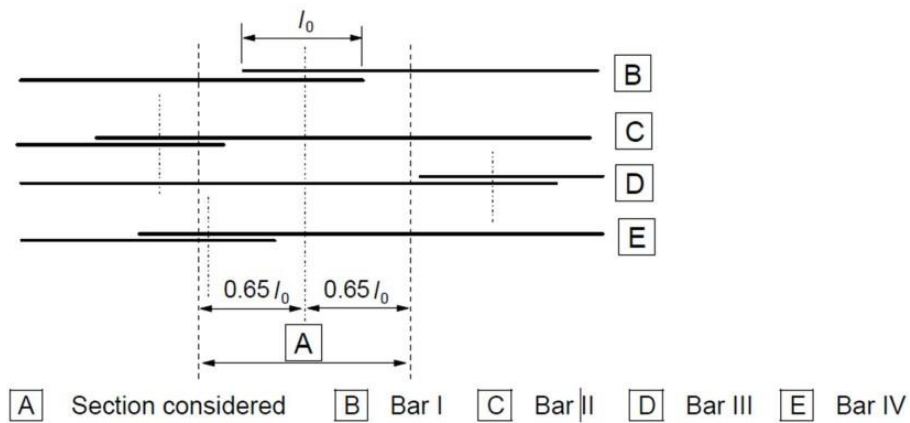
$$l_{b,rqd} = \frac{\sigma_{sd}}{4 \cdot f_{bd}} = \frac{10 \cdot 347.83}{4 \cdot 1.91} = 455.3\text{mm}$$

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq 10, \text{min}$$

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: $\rho_1 = 50\%$ 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 $\rho_1 > 50\%$,

$$l_{0, \min} \geq \max \begin{cases} 0.3\alpha_6 l_{b, rqd} = 0.3 * 1.5 * 453.3 = 204.89\text{mm} \\ 15\phi = 15 * 10 = 150 \\ 200\text{mm} \end{cases}$$

$$l_{0, \min} = 204.89\text{mm}$$

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

$$l_0 = 1 * 1 * 1 * 1.5 * 455.3\text{mm} = 682.95\text{mm} \geq 204.89\text{mm}$$

$$\text{lap length, } l_0 = \mathbf{685\text{mm}}$$

3.2.9 Anchorage length

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

For anchorage in tension

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

$$l_{b,min} \geq \max \begin{cases} 0.3 * 455.3 = 136.59\text{mm} \\ 10 * 10 = 100\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 136.59\text{mm}$$

For anchorage in Compression

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

$$l_{b,min} \geq \max \begin{cases} 0.6 * 455.3 = 273.18\text{mm} \\ 10 * 10 = 100\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 273.28$$

3.2.10 Check depth of flexure

For flexure

$$N = \left[11 + 1.5 \sqrt{f_{ck}} \frac{bd}{\rho} + 3.2 \sqrt{f_{ck}} \left(\frac{bd}{\rho} - 1 \right) \frac{3}{2} \right] \text{ If } \rho \leq \rho_o$$

$$N = 11 + 1.5 \sqrt{f_{ck}} \frac{bd}{\rho} + 1/12 * \sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_o}}$$

If $\rho > \rho_o$ (ES EN 1992-1-1, 2015) (7.4.2)

$$\text{Ast. pro} = \frac{bd}{S_{pro}} \quad \text{Ast. pro} = \frac{1000 * 150}{400} = 375$$

$$\rho = \frac{\text{Ast. pro}}{bd} = \frac{375}{1000 * 150} = 0.0025$$

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

$$N = \left[11 + 1.5 \sqrt{20} * \frac{0.00447}{0.0025} + 3.2 \sqrt{20} \left(\frac{0.00447}{0.0025} - 1 \right) \frac{3}{2} \right]$$

N=33.00

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

$$A_{Scal} = \frac{M_{sd}}{Z * f_{yd}}$$

$$M_{sd} = 28.32 \text{ KN-M}$$

$$Z = 0.9d = 0.9 * 150 = 135$$

$$f_{yd} = \frac{f_{yk}}{1.5} = \frac{400}{1.5} = 347.83$$

$$A_{Scal} = \frac{28.32 * 1000 * 1000}{135 * 347.84} = 603.1 \text{ mm}^2$$

$$F1 = \frac{500}{f_{yk} * \frac{A_{Scal}}{A_{Spro}}} = \frac{500}{400 * \frac{603.1}{375}} = 0.777$$

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

$$\frac{4300}{d} = 33 * 1.3 * 0.777 * 1 * 1$$

$$d = 129$$

D used = 129 + 25 + 10/2 = 159 mm < D used 190 mm then OK!

3.3 Analysis and design of roof floor slab

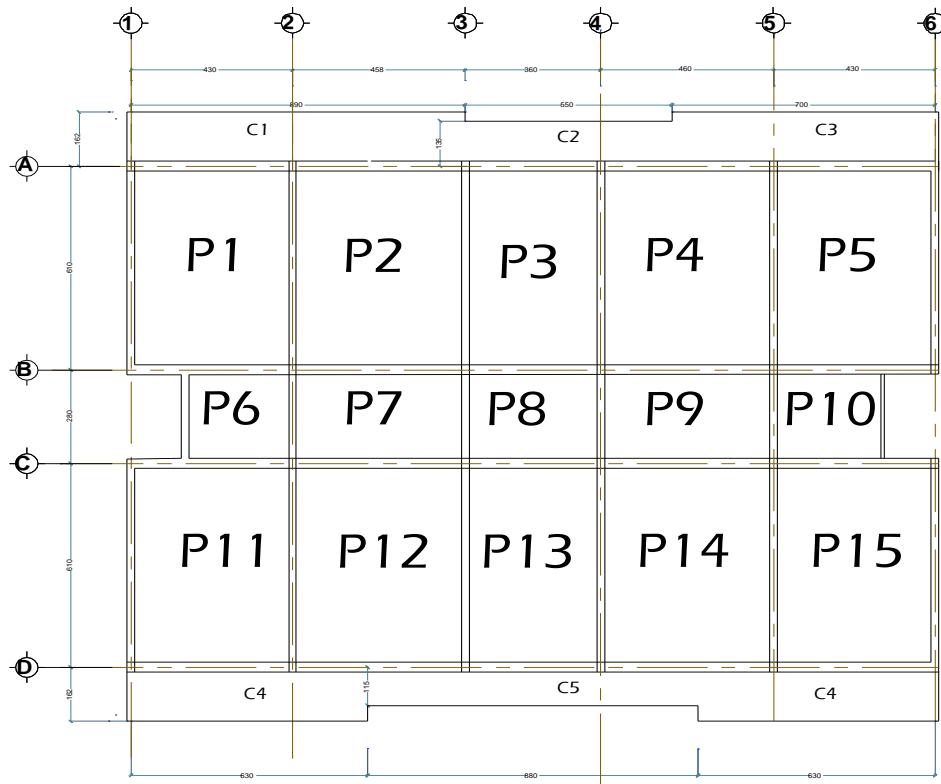


Figure 3.6 floor slab layout for Roof floor

3.3.1 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 whererelevant) 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

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

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)

- g. 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)
- h. Minimum strength class = C20/25, for exposure class XC1 [Appendix B,]
- i. Minimum concrete cover for bond/durability is 12 mm

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

$C_{min, b} = 12 \text{ mm}$, assumed diameter of bar (Appendix B, Table B4). $C_{min, dur} = 10 \text{ mm}$,

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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

j. Nominal cover

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

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.

k. Minimum cover Design for Fire = 20mm

l. Governing concrete cover = 22mm

3.3.2 Depth determination

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

$$\frac{l}{d} = N * K * F1 * F2 * F3 \text{ (Yohannes, 2021)}$$

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

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

$$F2 = 1, F3 = 1$$

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

Sample calculation for panel 1

$$\frac{l_x}{d} = N * K * F1 * F2 * F3 = 17.71 * 1.3 * 1.25 * 1 * 1 = 28.78$$

$$\frac{l_x}{d} = 28.78, \quad d = 149.43\text{mm}$$

The other depth determination calculation for the rest panels found in [Appendix D]

3.3.3 Determination of loading

When we determine the design load, we use the load combination $P_d = 1.35G_k + 1.5Q_k$

The imposed or live loads on the floor based on its specific use is derived from (ES EN1992-1-1, 2015) shown in the table below

Table 3.6 Load determination for roof slab

	Material	Unit weight	Dimension	Load KN/m ²
Slab	Reinforced concrete	25	0.2100	5.25
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		-0.811	KN/m ²	
DEAD LOAD G_k		6.63	KN/m ²	
COMB 1	$P_d = 1.35 G_k + 1.5 Q_k$	9.22	KN/m ²	
COMB 2	$P_d = 1.35 G_k + 1.5 WL$	7.734	KN/m ²	
COMB 3	$P_d = 1.35 G_k + 1.5 Q_k + 0.9WL$	8.49	KN/m ²	
COMB 4	$P_d = 1.35 G_k + 1.5 Q_k - 0.9WL$	9.95	KN/m ²	

3.3.4 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.

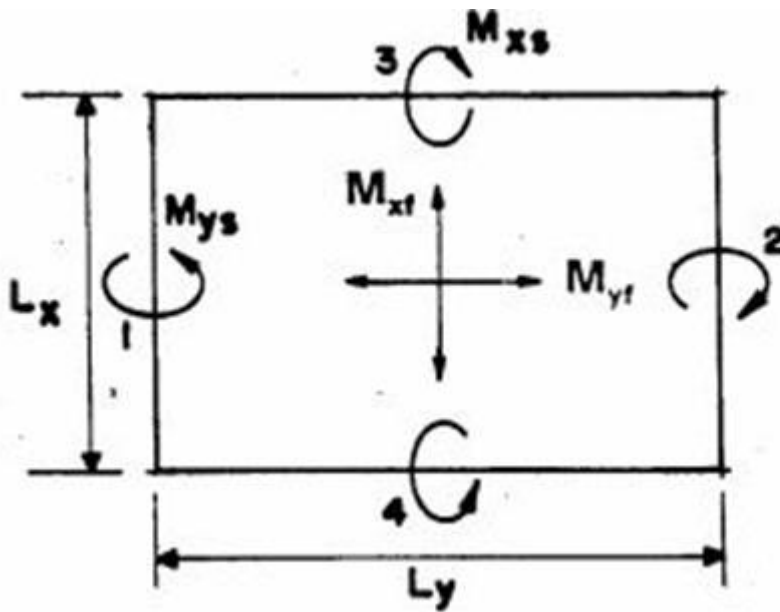


Figure 3.7 Moment direction on panels

Sample calculation

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

$$M_i = 0.0689 * 9.95 * 4.3^2 = 12.68$$

$$M_i = 0.0517 * 9.95 * 4.3^2 = 9.51$$

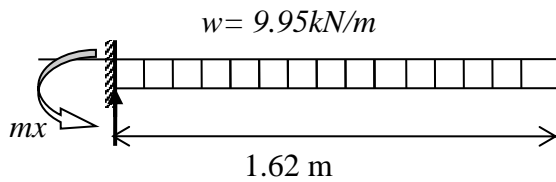
$$M_i = 0.039 * 9.95 * 4.3^2 = 7.17$$

$$M_i = 0.030 * 9.95 * 4.3^2 = 5.52$$

The moment analysis calculation for the rest panels found in [Appendix D-2]

Moment analysis for cantilever slabs

C1

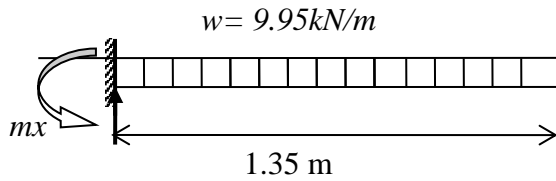


$$M_{xs} = \frac{wL^2}{2} = 9.95 * 1.62^2 / 2 = 13.06 \text{ KNM}$$

$$R_A = wL = 9.95 * 1.62 = 16.12 \text{ KN}$$

$M_{xs} =$

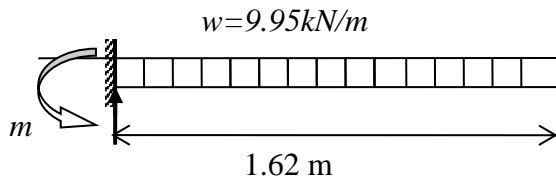
C2



$$M_{xs} = \frac{wL^2}{2} = 9.95 * 1.35^2 / 2 = 9.02 \text{ KNm}$$

$$R_A = wL = 9.95 * 1.35 = 13.43 \text{ KN}$$

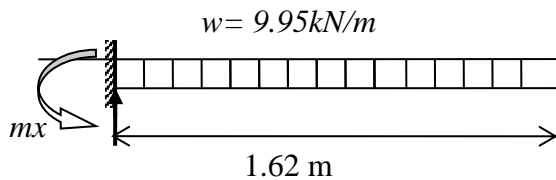
C3



$$M_{xs} = \frac{wL^2}{2} = 9.95 * 1.62^2 / 2 = 13.06 \text{ KN}$$

$$R_A = wL = 9.95 * 1.62 = 16.12 \text{ KN}$$

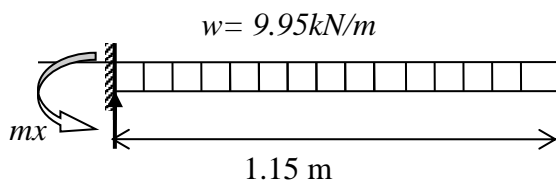
C4



$$M_{xs} = \frac{wL^2}{2} = 9.95 * 1.62^2 / 2 = 13.06 \text{ KNm}$$

$$R_A = wL = 9.95 * 1.62 = 16.12 \text{ KN}$$

C5



$$M_{xs} = \frac{wL^2}{2} = 9.95 * 1.15^2 / 2 = 6.56 \text{ KNM}$$

$$R_A = wL = 9.95 * 1.15 = 11.44 \text{ KN}$$

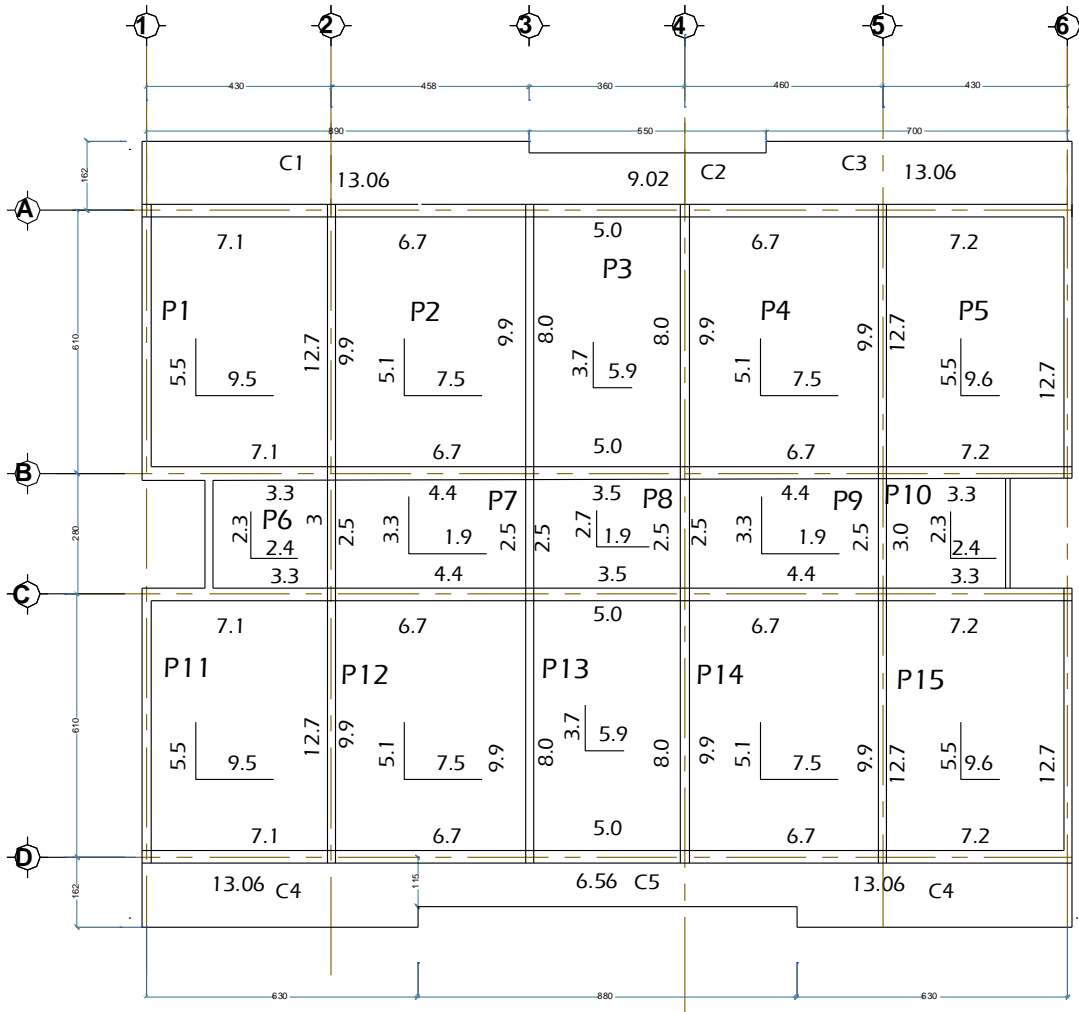


Figure 3.8 Unadjusted Moment for typical roof floor

Support moment adjustment

Two methods of moment adjustment are used

There are two cases (on EBCS-2015)

Two methods of moment adjustments are used.

Methods (1) if $\Delta N < 20\%$ (M large) we use average method.

Methods (2) if $\Delta N > 20\%$ (M large) we use moment distribution method based on relative stiffness.

- If M_s is decreased, then field moments are increased to allow equilibrium compatibility.
- If \bar{M}_s is increased, no field moment adjustment.

- ✓ 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.

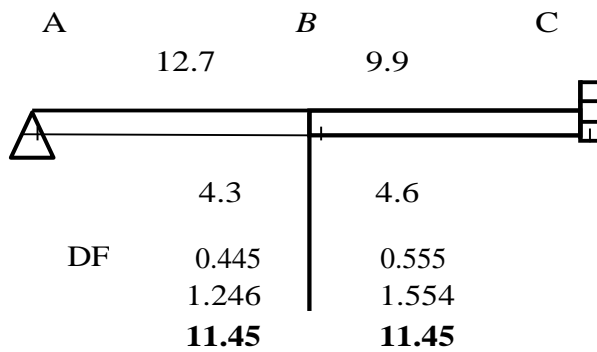
- ✓ Sample calculation

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

$$\Delta M_s = 12.7 - 9.9 = 2.8$$

$$20\% M_{large} = 0.2 * 12.3 = 2.54$$

Moment distribution method required



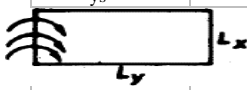
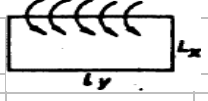
$$K_{AB} = \frac{I}{L}, \frac{3I}{4 * 4.3} = 0.174 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.23256}{0.23256 + 0.2174} = 0.445 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.555$$

The other panels support moment adjustment is summarized in [Appendix D-3]

–

Field moment adjustment (panel 1)

M _{xf}	10.42	M _{xs}	12.97	M _{xs,adje}	11.45
M _{yf}	5.93	M _{ys}	7.1	M _{ys, adje}	5.9
ΔM_{xs}	1.52	M _{xs} - M _{xs,adj}			
ΔM_{ys}	1.2	M _{ys} - M _{ys,adj}			
					
CX	0.31314	CX	0.402279		
CY	0.107023	CY	0.327093		
ΔM_{xf}	0.987232	$\Delta M_{xf} = C_x \Delta M_{xs} + C_x \Delta M_{ys}$			
M _{xf,adje}	10.38	M _{xf,adj} = ΔM_{xf} + M _{xf}			
ΔM_{yf}	0.625609	$\Delta M_{yf} = C_y \Delta M_{xs} + C_y \Delta M_{ys}$			
M _{yf,adje}	6.04	M _{yf,adj} = ΔM_{yf} + M _{yf}			

The other panels support moment adjustment is summarized in [Appendix D-4]

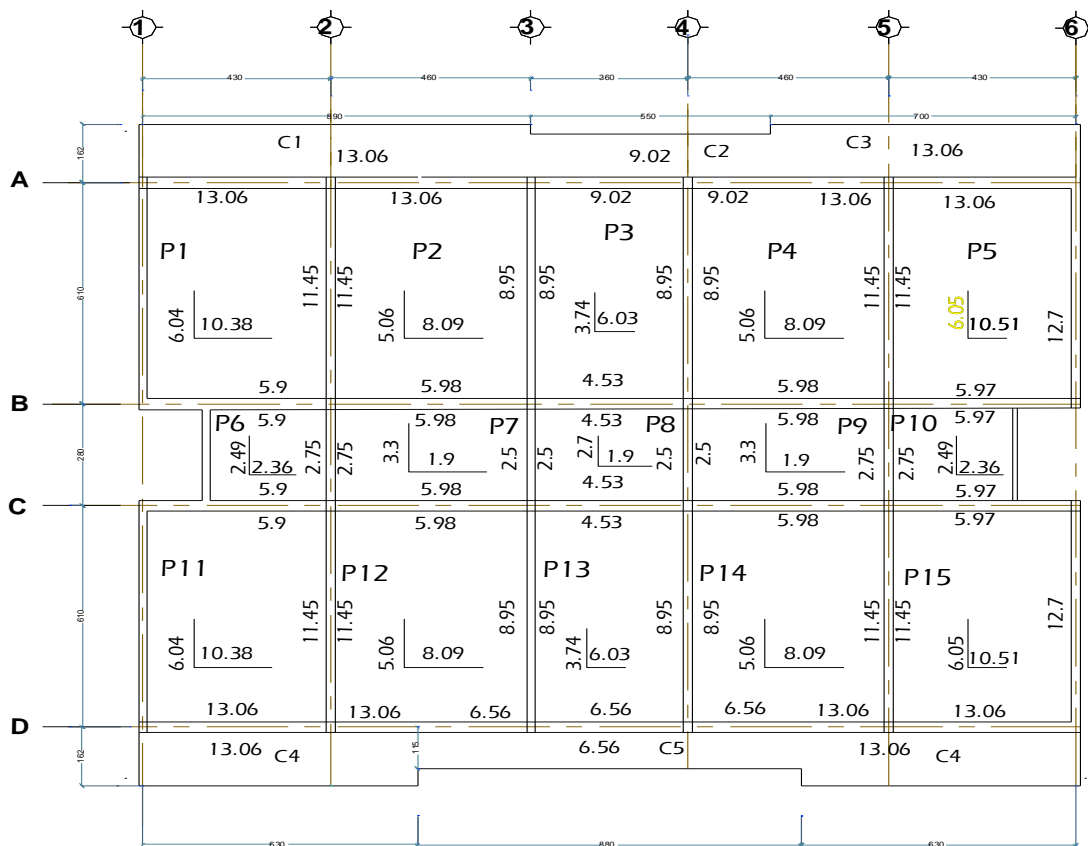


Figure 3.9 Adjusted moment for roof floor

Load transfer to beam

Two way slab shear

The shear force for each panel calculated by coefficient method

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

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

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

Sample calculation for P-1

$$\beta_{vxc} = 0.493 \quad \beta_{vxd} = 0.34$$

$$\beta_{vyc} = 0.36 \quad \beta_{vyd} = 0$$

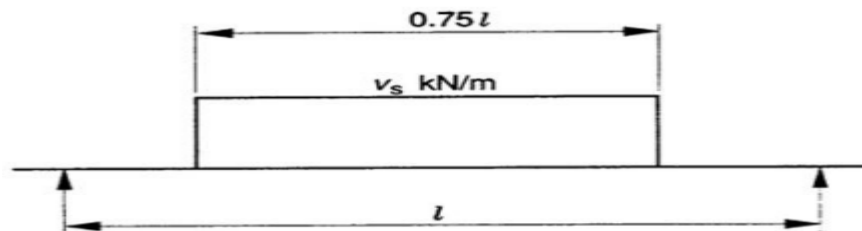
$$V_{xc} = \beta_{vxc} P d L_x = 0.493 * 9.95 * 4.3 = 21.1$$

$$V_{xd} = \beta_{vxd} P d L_x = 0.34 * 9.95 * 4.3 = 14.55$$

$$V_{yc} = \beta_{vyc} P d L_y = 0.36 * 9.95 * 4.3 = 15.4$$

$$V_{yd} = \beta_{vyd} P d L$$

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]

3.3.5 Check for shear strength

$$V_{Rdc} = [0.12K(100\rho_1 f_{ck})^{(1/3)}] b d \geq V_{min}$$

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

$$d = 185$$

$$K = 2.04 \quad \text{not} < 2, \text{ so } k = 2$$

$$\rho_1 = \frac{A_s l}{b d} \leq 0.02$$

$$VR_{dc} = [0.12 * 2(100 * 0.0015 * 20)^{(1/3)}] = 0.346$$

Taking minimum reinforcement ϕ 12 @ 430

$$\rho_1 = 0.0015$$

$$V_{min} = [0.035 (K)^{(3/2)} (f_{ck})^{(1/2)}] = 0.443$$

$$VR_{dc} = 0.346 \leq V_{min} = 0.443 \text{ Then } VR_{dc} = 0.443$$

Maximum shear force transfer to beam is $V_i = 21.1 \text{ KN/m}$

$$V_i = 21.1 \text{ KN/m} = 0.0211 \text{ KN/mm}$$

$0.443 > 0.0211$, the slab thickness is sufficient to support the design load (ES EN 1992-1-1:2015, -)

3.3.6 Reinforcement Calculation for roof floor slab

$$\mu_{sd} = \frac{msd}{f_{cd} * b * d} \quad A_{st} = \frac{msd}{z * f_y * d}$$

$$A_{st \min} = \max \left\{ \begin{array}{l} 0.26 \left(\frac{f_{ctm}}{f_{yk}} \right) bd \\ 0.0013bd \end{array} \right. \quad S = \frac{bas}{As} \quad S_{\max} = \max \left(\frac{3D}{400mm} \right) \quad (\text{Yohannes, 2021})$$

$$A_{st \min} = \max \left\{ \begin{array}{l} 0.26 \left(\frac{2.2}{400} \right) * 1000 * 185 = 264.55 \\ 0.0013 * 1000 * 185 = 240.5 \end{array} \right.$$

$$A_{St \text{ min}} = 264.55 \text{mm}^2$$

$$A_{St \text{ max}} = 0.04bd = 0.04 * 1000 * 185 = 7400 \text{mm}^2$$

$$A_{st} = \frac{msd}{Z * f_{yd}}, \quad F_{yd} = \frac{F_{yk}}{1.5} = \frac{400}{1.5} = 347.83, \quad Z = 0.9d = 0.9 * 185 = 166.6 \text{mm}$$

Panel 1 support moment 11.45KN-m

$$A_{st} = \frac{11.45 * 1000 * 1000}{166.6 * 347.83} = 197.56 \text{mm}^2 > 264.55 \text{mm}^2 \dots\dots\dots \text{OK!}$$

$$S_{\text{max}} = \max\left\langle \frac{3D}{400 \text{mm}} \right\rangle = \frac{3 * 210}{400 \text{mm}} = 630 \text{mm} \quad \text{Then } S_{\text{max}} = 630 \text{mm}$$

Let the ϕ of reinforcement be 12mm

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

$$S = \frac{bas}{A_{st}} = \frac{1000 * 78.5}{197.56} = 397.35 \text{mm}$$

397.35 mm \approx 390mm used < 630mm **OK!**

Reinforcement calculation for other panels found in [Appendix D]

3.3.7 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 $4 \varnothing$ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds $4 \varnothing$ 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

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

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

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

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

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

$\eta_1 = 1$ for good quality of bond

$\eta_1 = 0.7$ for other cases

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

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

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

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

$f_{ctk,0.05} = 1.5$ for C20/25 mpa, from ESEN 1992

Sample calculation for $\emptyset 10$ reinforcement bar

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

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

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

$\eta_1 = 0.7$ for other cases

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

$$\sigma_{sd} = \frac{400}{1.15} = 347.83$$

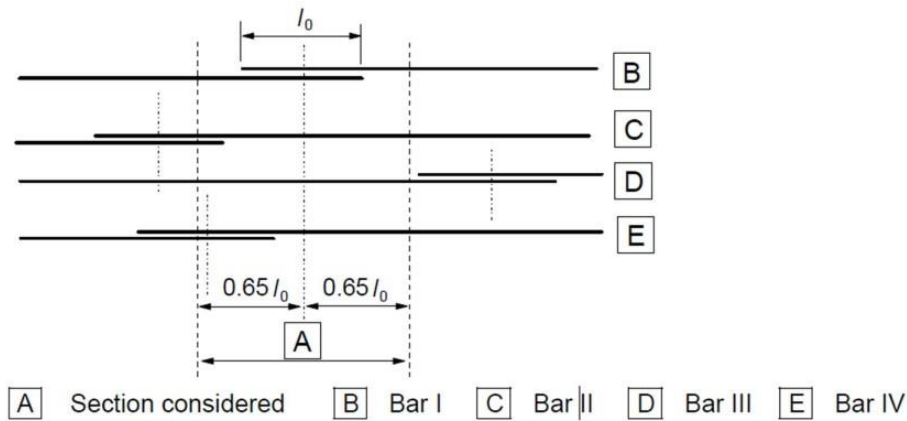
$$l_{b,rqd} = \frac{\emptyset * \sigma_{sd}}{4 * f_{bd}} = \frac{10 * 347.83}{4 * 1.91} = 455.3\text{mm}$$

$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq 10, \text{min}$

$\alpha_1, \alpha_2 \text{ \& } \alpha_3 = 1, \alpha_4 = 0.7$, 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 $0.65 l_0$ 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 $\rho_1 > 50\%$,

$$l_{0, \min} \geq \max \begin{cases} 0.3\alpha_6 l_{b, \text{reqd}} = 0.3 * 1.5 * 453.3 = 204.89\text{mm} \\ 15\phi = 15 * 10 = 150 \\ 200\text{mm} \end{cases}$$

$$l_{0, \min} = 204.89\text{mm}$$

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

$$l_0 = 1*1*1*1.5*455.3\text{mm} = 682.95\text{mm} \geq 204.89\text{mm}$$

$$\text{lap length, } l_0 = \mathbf{685\text{mm}}$$

3.3.8 Anchorage length

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

For anchorage in tension

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

$$l_{b,min} \geq \max \begin{cases} 0.3 * 455.3 = 136.59\text{mm} \\ 10 * 10 = 100\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 136.59\text{mm}$$

For anchorage in Compression

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

$$l_{b,min} \geq \max \begin{cases} 0.6 * 455.3 = 273.18\text{mm} \\ 10 * 10 = 100\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 273.28$$

3.3.9 Check depth of flexure

For flexure

$$N = [11 + 1.5 \sqrt{f_{co}} \frac{+ 3.2 \sqrt{f_{ck}} (\rho - 1)^{3/2}}{\rho}] \text{ If } \rho \leq \rho_o$$

$$N = 11 + 1.5 \sqrt{f_{ck}} \rho_o / (\rho - \rho_o) + 1/12 * \sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_o}}$$

If $\rho > \rho_o$ (ES EN 1992-1-1, 2015) (7.4.2)

$$\text{Ast. pro} = \frac{bd}{S_{pro}} \quad \text{Ast. pro} = \frac{1000 * 150}{400} = 375$$

$$\rho = \frac{\text{Ast. pro}}{bd} = \frac{375}{1000 * 150} = 0.0025$$

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

$$N = [11 + 1.5 \sqrt{20 * 0.00447} \frac{+ 3.2 \sqrt{20} (0.00447 - 1)^{3/2}}{0.0025}]$$

N=33.00

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

$$AS_{cal} = \frac{M_{sd}}{Z * f_{yd}}$$

$$M_{sd} = 28.32 \text{ KN-M}$$

$$Z = 0.9d = 0.9 * 150 = 135$$

$$f_{yd} = \frac{F_{yk}}{1.5} = \frac{400}{1.5} = 347.83$$

$$AS_{cal} = \frac{28.32 * 1000 * 1000}{135 * 347.84} = 603.1 \text{ mm}^2$$

$$F1 = \frac{500}{F_{yk} * \frac{AS_{cal}}{AS_{pro}}} = \frac{500}{400 * \frac{603.1}{375}} = 0.777$$

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

$$\frac{4300}{d} = 33 * 1.3 * 0.777 * 1 * 1$$

$$d = 129$$

D used = 129 + 25 + 10/2 = 159 mm < D used 190 mm then OK!

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).

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
 - Heavy dead load due to inclined length of slab acting over horizontal span and due to additional weight of steps,
 - 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

1.1. Concrete cover design

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

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

- g. 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, -)
- h. Minimum strength class = C20/25, for exposure class XC1 (Appendix B, Table B1).
- i. Minimum concrete cover for bond/durability = 12 mm

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

$C_{min,b} = 12$ mm, assumed diameter of bar (Appendix B, Table B4).

$C_{min,dur} = 10$ 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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

- j. Nominal cover = 22mm

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

$\Delta c_{dev} = 10$ mm, the value of Δc_{dev} for use in a Country may be found in its National Annex. The recommended value is 10 mm.(ES EN 1992-1-1:2015, -)(4.4.1.3,1)

- k. Minimum cover Design for Fire = 20mm

- l. Governing concrete cover = 22mm
-

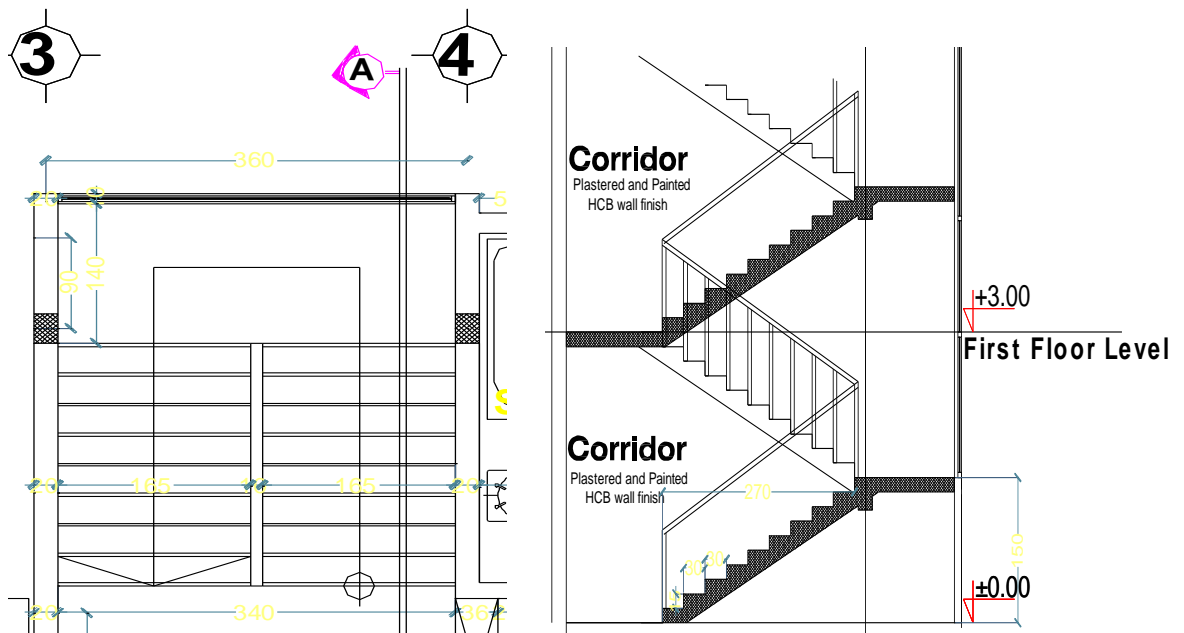


Figure 4.1 stair case plan and section view

Geometric data

- Riser height = 15cm
- Tread width 30cm
- Number of risers = 10
- Number of treads = 10

$$\theta = \tan^{-1} \left(\frac{1.5}{3} \right) = 26.56^\circ = 27^\circ$$

1.2. 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

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

.....(7.16a)

$$\frac{l}{d} = K \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_o}{\rho - \rho'} + \frac{1}{12} \sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_o}} \right] \text{ if } \rho > \rho_o$$

.....(7.16b)

where:

l/d is the limit span/depth

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

ρ_o 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_{ck} is in MPa units

Table 7.4N: 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

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

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

Concrete class C-25fck = 20MPa

fcd = 11.33MPa Steel grade S-400

fyk=347.82MPa

ρ_0 is the reference reinforcement ratio = $\sqrt{fck} * 10^{-3} = \sqrt{20} * 10^{-3} = 0.00447$
 $\rho = 0.5\% = 0.005$

Since $\rho > \rho_0$ use equation(7.16b)

$$\frac{l}{d} = K \left[11 + 1.5 \sqrt{fck} \frac{\rho_0}{\rho - \rho_0} + \frac{1}{12} * \sqrt{fck} \sqrt{\frac{\rho'}{\rho_0}} \right]$$

$\frac{l}{d} = 16.997$ Because we used S400 multiply the value $\frac{500}{dfyk} = 1.25$

$\frac{l}{d} = 16.997 * 1.25 = 21.25$

$L = lx = 2700mm$

$\frac{2700}{d} = 21.25$, $d = 127.06mm$

Drequired = drequired + minimum cover + diameter of $\frac{bar}{2} = 127.06 + 22 + \frac{12}{2} = 156.06mm$

Use D = **160mm**

$d = 160 - 15 - 12/2 =$ **139mm**

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.

1.3. Loading condition on stair.

Table 4-1 Material data for stair

Material	Thickness(m)	Unit weight (KN/m ³)
Concrete	0.16	25
Marble	0.03	27
Cement screed	0.02	23
Plastering	0.02	23

Load on stair

1- Dead Load

✓ **Soffit slab (inclined slab)**

$$\text{Self-weight of slab} = 0.16\text{m} * 1\text{m} * 25\text{KN/m}^3 / \cos 27^\circ = 4.49\text{KN/m}$$

$$\text{Plastering} = 0.02\text{m} * 1\text{m} * 23\text{KN/m}^3 / \cos 27^\circ = 0.52\text{KN/m}$$

$$\text{Total dead load} = 4.49\text{KN/m} + 0.52\text{KN/m} = \mathbf{5.01\text{KN/m}}$$

✓ **Thread**

$$\text{Self-weight of thread} = 0.5 * 0.3\text{m} * 1\text{m} * 25\text{KN/m}^3 = 3.75\text{KN/m}$$

$$\text{Floor finish} = 0.03\text{m} * 1\text{m} * 27\text{KN/m}^3 = 0.81\text{KN/m}$$

$$\text{Total dead load} = 3.75\text{KN/m} + 0.81\text{KN/m} = \mathbf{4.56\text{KN/m}}$$

✓ **Riser**

$$\text{D.L of Floor finish} = \frac{\text{Number of raseer } (R_{cs} * t_{cs} * \gamma_{sc})}{\text{Projected leneeth}}$$

$$\text{D.L of Floor finish} = \frac{10 * (0.02\text{m} * 23 * 1 * 0.15)}{(10 * 0.3)} = \mathbf{0.23\text{KN/m}}$$

$$\text{D.L of Cement screed} = \frac{\text{of raseer } (b_{cs} * t_{cs} * \gamma_{sc})}{\text{Projected leneeth}}$$

$$\text{D.L of Cement screed} = \frac{10 * (0.03 * 23 * 0.15)}{10 * 0.3} = 0.345\text{KN/m}$$

$$\text{Total dead load} = 0.575\text{KN/m}$$

$$\text{Total dead load of stair} = 5.01\text{KN/m} + 4.56\text{KN/m} + 0.575\text{KN/m} = 10.145\text{KN/m}$$

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

The live load for staircase given in codes is the loading on horizontal projection area.

Design Load, $P_d = 1.35 \text{ D.L} + 1.5 \text{ L.L}$

$$P_d = 1.35(10.145) + 1.5(3) = \mathbf{18.2 \text{ KN/m}}$$

Loading on landing

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

$$\begin{aligned} &= (0.03 \text{ m} * 27 \text{ KN/m}^3) + (0.02 * 23 \text{ KN/m}^3) + (0.16 \text{ m} * 25 \text{ KN/m}^3) + (0.02 \text{ m} * 23 \text{ KN/m}^3) \\ &= 5.96 \text{ KN/m} \end{aligned}$$

Therefore D.L of Landing = 5.96 KN/m

Total Dead Load and design load for the landing

$$\text{D.L} = 5.96 \text{ KN/m} \quad \text{L.L} = 3 \text{ KN/m}^2 * 1 \text{ m} = 3 \text{ KN/m}$$

$$\text{Design load, } P_d = 1.35(5.96) \text{ KN/m} + 1.5(3) \text{ KN/m} = 12.55 \text{ KN/m}$$

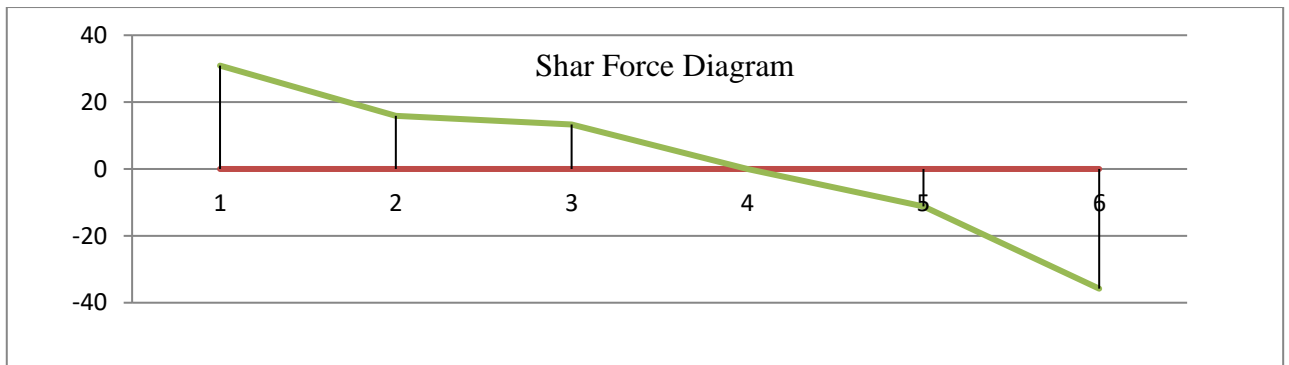
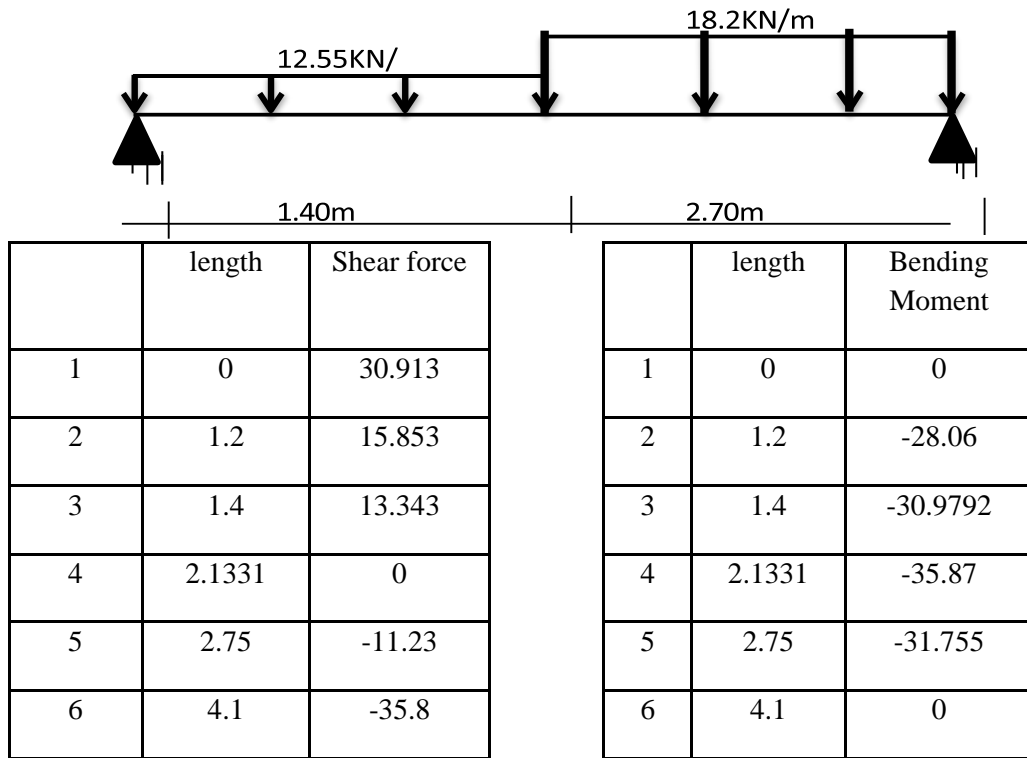


Figure 4.2 Shear force diagram

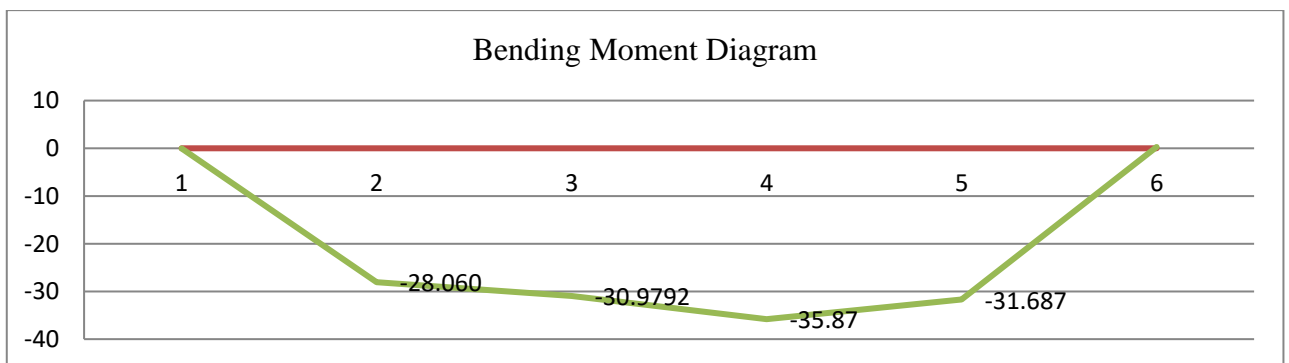


Figure 4.3 Bending moment diagram

4.1 Check depth of flexure

Maximum Moment is 35.87KN/m take $K_x = 0.448$

$$D_{min} = \sqrt{\frac{M_{sd}}{k f_{ck} * b}} = \sqrt{\frac{35.87}{0.22 * 25 * 1000}} = 80.75 < 160\text{mm}$$

There for, depth of 160mm satisfy flexural requirement for all floor.

4.2 Main Reinforcement Design

For the given; Material Data,

C-25, S-400

Effective depth, $d=160\text{mm}$

Width, $b=1000\text{mm}$

$F_{cd} = 11.33$

$F_{ctm} = 2.6\text{MPa}$

$F_{yd} = 347.83\text{Mpa}$

$D = 160\text{mm}$

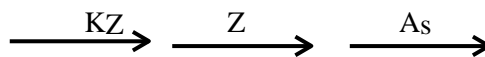
Effective depth, $d = D - 10/2 - 20 = 125\text{mm}$

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

Using $M_u = 35.87\text{KNm}$, calculate reinforcement,

$$\mu_{sd} = \frac{M}{f_{cd} b d^2}$$

Using $\mu_{sd} = 0.071$ read the



$$kZ=0.945$$

$$Z=Kz*d =0.945*160 = 151.2\text{mm}$$

$$A_s = \frac{Msd}{f_{yd}Z} = \frac{35.87 * 10^6}{347.83 * 151.2} = 682.05\text{mm}^2$$

To calculate spacing by selecting diameter of bar as

$$S = \frac{bas}{A_s} = \frac{1000 * 113.04}{682.05} = 165.73 = 160\text{mm}$$

where: - as=area of single bar as

As=calculated area of steel S=spacing

Use, $\phi 12$ c/c160mm

Compare the above result with minimum provision given by our code

$$A_{smin} = 0.26 * \frac{2.2 * 1000 * 139}{400} = 198.77\text{mm}^2$$

With $d = 160 - 15 - 6 = 139\text{mm}$, for $\phi 12$ main rebar

$$A_{smin} = 198.77\text{mm}^2 < A_{smin} = 682.05\text{mm}^2 \text{ OK!}$$

$$S_{max} \leq \begin{cases} 2D \\ 400 \\ S_{cal} \end{cases} = 2 * 160, 400\text{mm}, 200\text{mm}$$

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

Provide $\phi 12$ c/c200mm

✓ 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. $A_s \geq 20\% A_s$,

$$A_s \geq 0.2 * 682.05 \text{mm}^2 = 136.41 \text{mm}^2 < A_{smn} = 198.77 \text{mm}^2$$

Therefore, take the minimum reinforcement

$$d = 200 - 15 - 12 - 4 = 169 \text{mm, for } \varnothing 8 \text{ main transverse rebar}$$

$$A_{smin} = 0.26 * \frac{2.2 * 1000 * 169}{400} = 241.67 \text{mm}^2$$

To calculate spacing by selecting diameter of bar as

$$S = \frac{bas}{A_s} = \frac{1000 * 50.26}{241.67} = 207.97 = 200 \text{mm}$$

Provide $\varnothing 8 \text{c}/\text{c}200 \text{mm}$

✓ Check for shear

Shear Reinforcement

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

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

Where: F_{ck} is in MPa

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

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

$$\text{Take, } k=2 \quad (A_{s1}/b_w d) \leq 0.02 = (682.05/1000 * 139) = 0.04899$$

$$0.04899 < 0.02$$

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

b_{ww} -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} , - is [N]

Note: For the use of C_{RD} , V_{min} and k_1 refer the National Annex.

The recommended value is $\frac{0.18}{\gamma_c}$ that for V_{min} is given by expression (6.3N) and that for k_1 is 0.15 for C_{RD} ,

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

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

$$C_{RD} = \frac{0.18}{1.5} = 0.12$$

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

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

$$V_{Rd} = [0.12 * 2 * (100 * 0.00436 * 20)^{\frac{1}{3}} + 0.15 * 0] 1000 * 139 \leq (0.443 + 0.15 * 0) * 1000 * 128$$

= 68.66 kN/m > 56.7 kN/m... NOT OK

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

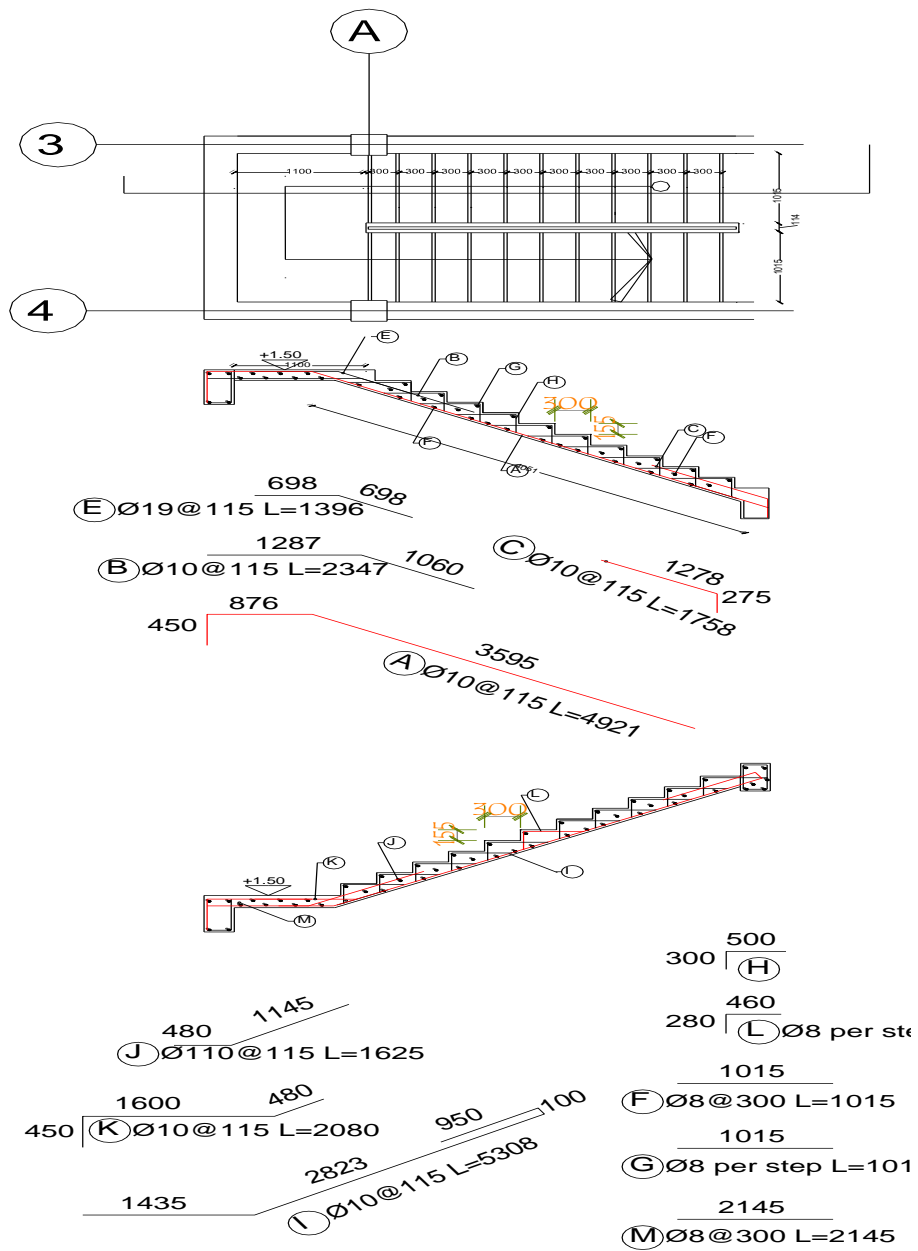


Figure 4.4 Stair case reinforcement detailing

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)

5.1 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-risebuildings

- 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 $L_u > 0.72m$, $\phi = H/L_u$

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

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

4Auto Wind Load Calculation for load pattern wind Y

EUROCODE1 2005 Auto Wind Load Calculation

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0; 90; 180 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 = TTB

Bottom Story = Ground Floor

Include Parapet = Yes, Parapet Height = 1.2

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)]	$I_v = \frac{k_1}{c_o(z) \ln\left(\frac{z}{z_0}\right)} \text{ for } z_{\text{min}} \leq z \leq z_{\text{max}}$ $= I_v(z_{\text{min}}) \text{ for } z < z_{\text{min}}$
Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5]	$k_r = 0.19 \left(\frac{z_0}{0.05}\right)^{0.25} \quad k_r = 0.215389$

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

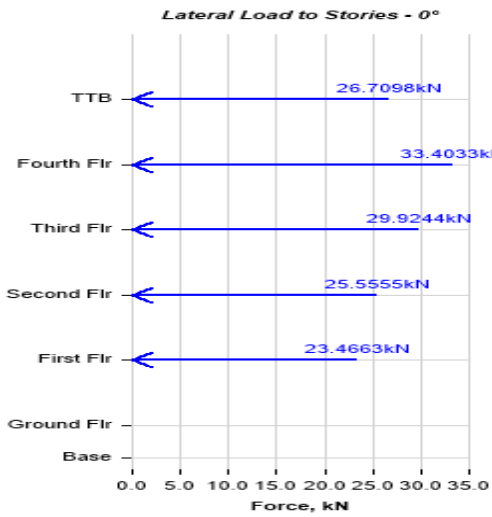
Peak Velocity Pressure, $\langle q_p \rangle(z)$ [EC 4.5(1) Eq. 4.8]

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

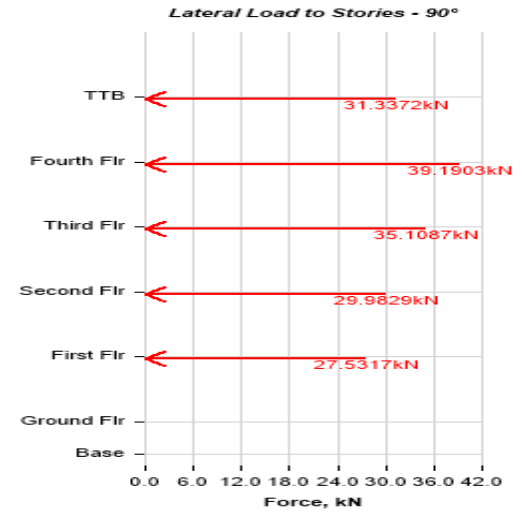
Wind Pressure, w [EC 5.2(1) Eq. 5.1]

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

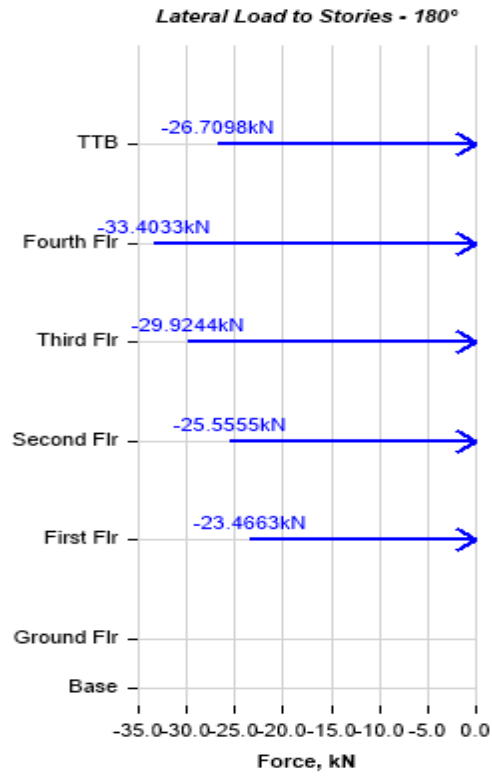
Applied Story Forces



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	26.7098	0
Fourth Floor	12	33.4033	0
Third Floor	9	29.9244	0
Second Floor	6	25.5555	0
First Floor	3	23.4663	0
Ground Floor	0	0	0
Base	-1.5	0	0



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	0	31.3372
Fourth Floor	12	0	39.1903
Third Floor	9	0	35.1087
Second Floor	6	0	29.9829
First Floor	3	0	27.5317
Ground Floor	0	0	0
Base	-1.5	0	0



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	-26.7098	0
Fourth Floor	12	-33.4033	0
Third Floor	9	-29.9244	0
Second Floor	6	-25.5555	0
First Floor	3	-23.4663	0
Ground Floor	0	0	0
Base	-1.5	0	0

EUROCODE1 2005 Auto Wind Load Calculation

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

Exposure Parameters

Exposure From = Diaphragms

Terrain Category = III

Wind Direction = 0; 90; 180 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 = TTB

Bottom Story = Ground Floor

Include Parapet = Yes, Parapet Height = 1.2

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)]

$$k_1 = 1$$

Orthography Factor, c_o [EC 4.3.3]

$$c_o = 1$$

Turbulence Intensity, I_v [EC 4.4(1)]

$$I_v = \frac{k_1}{c_o(z) \ln\left(\frac{z}{z_0}\right)} \text{ for } z_{\text{min}} \leq z \leq z_{\text{max}}$$

$$= I_v(z_{\text{min}}) \text{ for } z \leq z_{\text{min}}$$

Terrain Factor, k_r [EC 4.3.2(1) Eq. 4.5]

$$k_r = 0.19 \left(\frac{z_0}{0.05}\right)^{0.05} \quad k_r = 0.215389$$

Roughness Factor, $c_r < c_r(z)$ [EC 4.3.2(1) Eq. 4.4]

$$c_r(z) = k_r \ln\left(\frac{z}{z_0}\right) \text{ for } z_{\text{min}} \leq z \leq z_{\text{max}}$$

$$= c_r(z_{\text{min}}) \text{ for } z \leq z_{\text{min}}$$

Lateral Loading

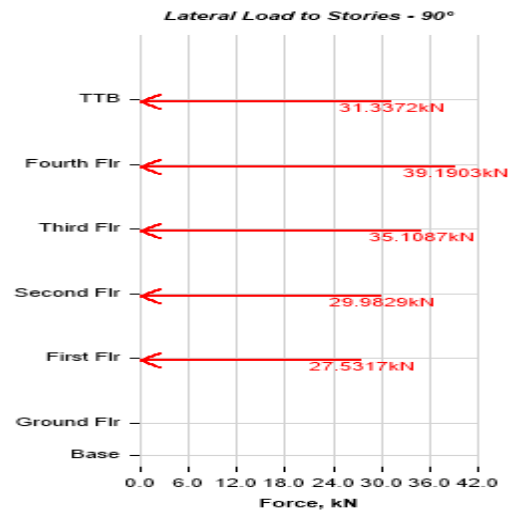
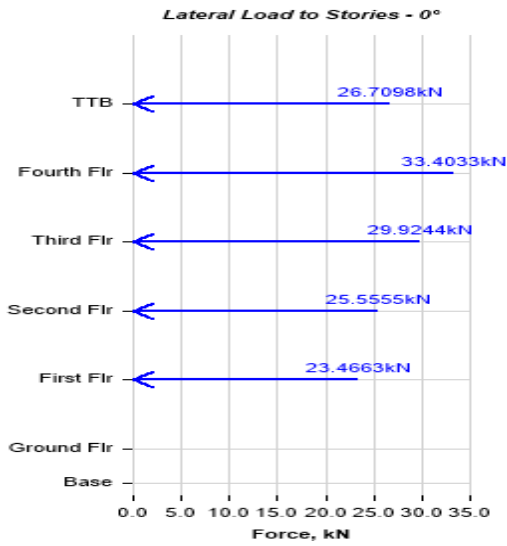
Peak Velocity Pressure, $< q_{p>(z)}$ [EC 4.5(1) Eq. 4.8]

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

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

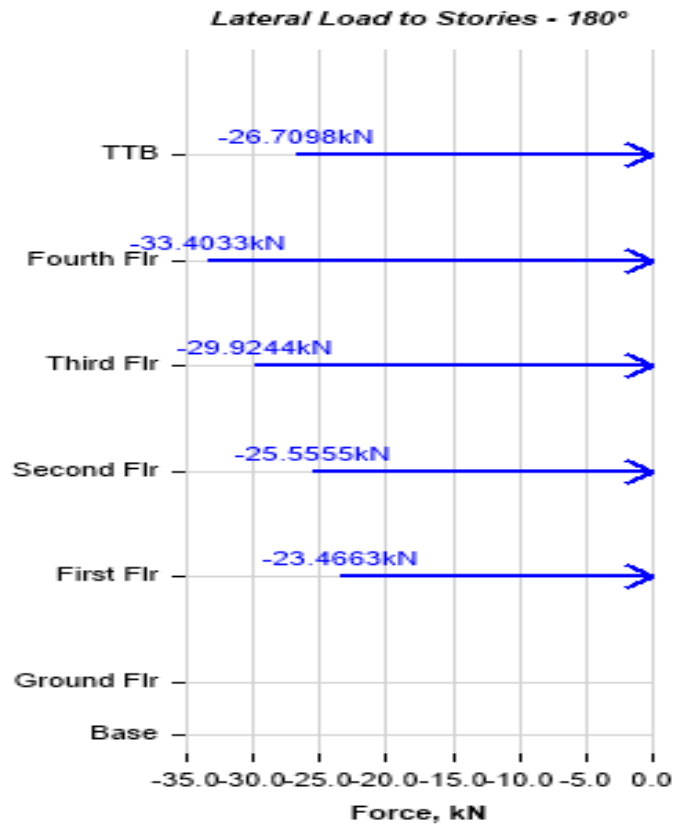
$$w = q_p(z)C_sC_d(C_{p,wind} + C_{p,lee})$$

Applied Story Forces



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	26.7098	0
Fourth Floor	12	33.4033	0
Third Floor	9	29.9244	0
Second Floor	6	25.5555	0
First Floor	3	23.4663	0
Ground Floor	0	0	0
Base	-1.5	0	0

Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	0	31.3372
Fourth Floor	12	0	39.1903
Third Floor	9	0	35.1087
Second Floor	6	0	29.9829
First Floor	3	0	27.5317
Ground Floor	0	0	0
Base	-1.5	0	0



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	-26.7098	0
Fourth Floor	12	-33.4033	0
Third Floor	9	-29.9244	0
Second Floor	6	-25.5555	0
First Floor	3	-23.4663	0
Ground Floor	0	0	0
Base	-1.5	0	0

5.2 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 loadresisting 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,

5.2.1 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 cohesion less 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 = 3.9, 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)

5.2.2 Base shear force

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

$$F_b = S_d(T_1)M \quad (4.5)$$

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 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 $C_t=0.075$, $H=16.5\text{m}$

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

$$T_1 = 0.075 \cdot 16.5^{0.75} = 0.614 \text{ sec}$$

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

$$T_1 = 0.614 \text{ sec} < 2 \text{ sec} \dots \text{Ok}$$

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 Σ models, horizontal forces F_i to all storey.

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

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

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

(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:

$$F_i = F_b \left(\frac{z_i m_i}{\sum_{j=1}^n z_j m_j} \right)$$

where

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

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:

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

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

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

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

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 behavior 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)

Story	Diaphragm	Mass X	Mass Y	XCM	YCM	Cumulative X	Cumulative Y	XCC M	YCC M
		kg	kg	m	m	kg	kg	m	m
TTB	D1	188156.85	188156.85	10.7	9.0255	188156.85	188156.85	10.7	9.0255
Fourth Floor	D1	264230.55	264230.55	10.6956	9.0628	452387.4	452387.4	10.6974	9.0473
Third Floor	D1	266796.83	266796.83	10.7	9.1233	719184.24	719184.24	10.6984	9.0755
Second Floor	D1	266796.83	266796.83	10.7	9.1233	985981.07	985981.07	10.6988	9.0884
First Floor	D1	266796.83	266796.83	10.7	9.1233	1252777.9	1252777.9	10.6991	9.0958
Ground Floor	D1	0	0	4.92	3.42	1252777.9	1252777.9	10.6991	9.0958

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

$$F_t = 0.07T_1F_b = 0.07 * 1 * 98639.3 = 6904.75 \text{ KN}$$

5.3 Auto Seismic Loading

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQXP 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	$H = 16.5 \text{ m}$

Factors and Coefficients

Country = Other

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)]	$q = 3.9$

Seismic Response

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

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

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

Equivalent Lateral Forces

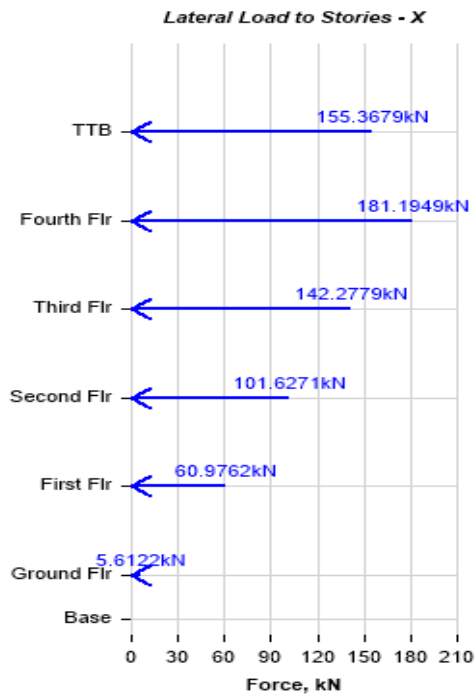
Seismic Base Shear Coefficient

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

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
X + Ecc. Y	0.987	13174.3383	647.0562

Applied Story Forces



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
TTB	15	155.36 79	0
Fourth Floor	12	181.19 49	0
Third Floor	9	142.27 79	0
Second Floor	6	101.62 71	0
First Floor	3	60.976 2	0
Ground Floor	0	5.6122	0
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQXN 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

$$H = 16.5 \text{ m}$$

Factors and Coefficients

Country = Other

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)]	$q = 3.9$

Seismic Response

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

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

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

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

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

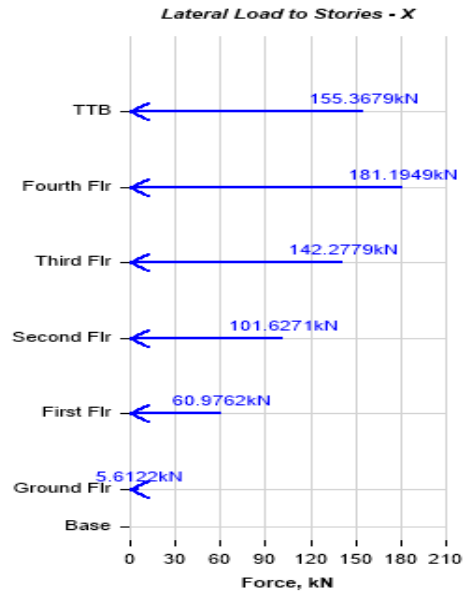
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{coeff} = S_d(T_1) \lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F_b (kN)
X - Ecc. Y	0.987	13174.3383	647.0562

Applied Story Forces



Story	Elevation m	X-Dir kN	Y-Dir kN
TTB	15	155.3679	0
Fourth Floor	12	181.1949	0
Third Floor	9	142.2779	0
Second Floor	6	101.6271	0
First Floor	3	60.9762	0
Ground Floor	0	5.6122	0
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQYP 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	$H = 16.5 m$

Factors and Coefficients

Country = Other

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)]	$q = 3.9$

Seismic Response

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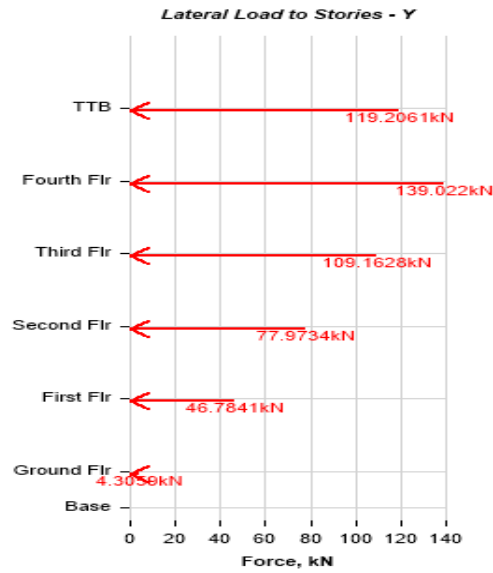
Equivalent Lateral Forces

Seismic Base Shear Coefficient	$V_{coeff} = S_d(T_1)\lambda$
--------------------------------	-------------------------------

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
Y + Ecc. X	1.286	13174.3383	496.4544

Applied Story Forces



Story	Elevation (m)	X-Dir (kN)	Y-Dir (kN)
TTB	15	0	119.2061
Fourth Floor	12	0	139.022
Third Floor	9	0	109.1628
Second Floor	6	0	77.9734
First Floor	3	0	46.7841
Ground Floor	0	0	4.3059
Base	-1.5	0	0

EUROCODE8 2004 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQYN 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 $H = 16.5 m$

Factors and Coefficients

Country = Other

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)] $q = 3.9$

Seismic Response

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

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

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

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

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

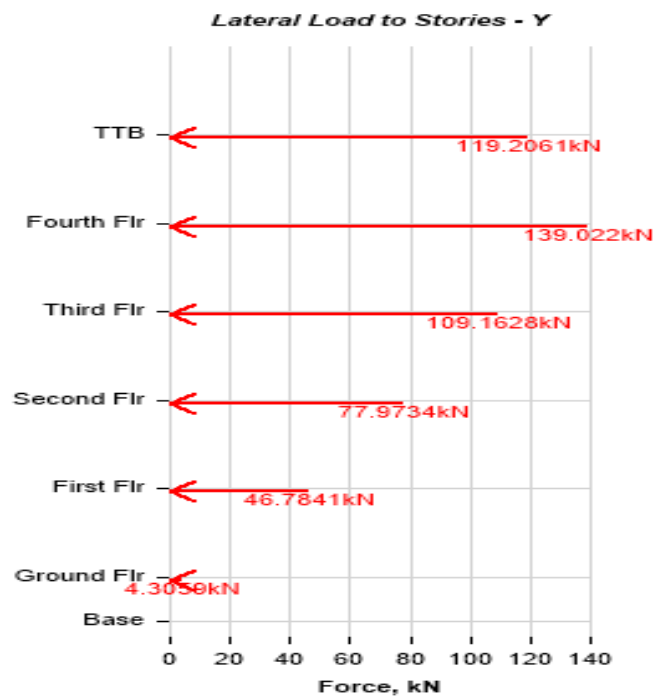
Equivalent Lateral Forces

Seismic Base Shear Coefficient $V_{\text{coeff}} = S_d(T_1)\lambda$

Calculated Base Shear

Direction	Period Used (sec)	W (kN)	F _b (kN)
Y - Ecc. X	1.286	13174.3383	496.4544

Applied Story Forces



Story	Elevation (m)	X-Dir (kN)	Y-Dir (kN)
TTB	15	0	119.2061
Fourth Floor	12	0	139.022
Third Floor	9	0	109.1628
Second Floor	6	0	77.9734
First Floor	3	0	46.7841
Ground Floor	0	0	4.3059
Base	-1.5	0	0

6 Frame analysis

Frame are often used in building and composed of beams and column that either pin or fixed connected. The loading on a frame cause bending of its member and if it has right joint connection. This structure is generally indeterminate strength of such frame is derived from the moment interconnection between the beam and the column at the rigid joint. (Kassimali 2011)

Frame can be classified in to two;

1. Non sway frame

- There is no horizontal movement
- When the frame is symmetric with geometric load
- Deflection is zero
- Bending does not cause the joint to have a linear displacement

2. Sway Frame

- Its lateral displacement and have horizontal moment
- Generally, frame will be sway
 - If the frame is not symmetric with geometric load
 - If we have inclined structure
 - Unsymmetrical shape of frame
 - Different end condition of the column
 - Non uniform section of the frame
 - Horizontal loading
 - Settlement of support
 - Combination of the above

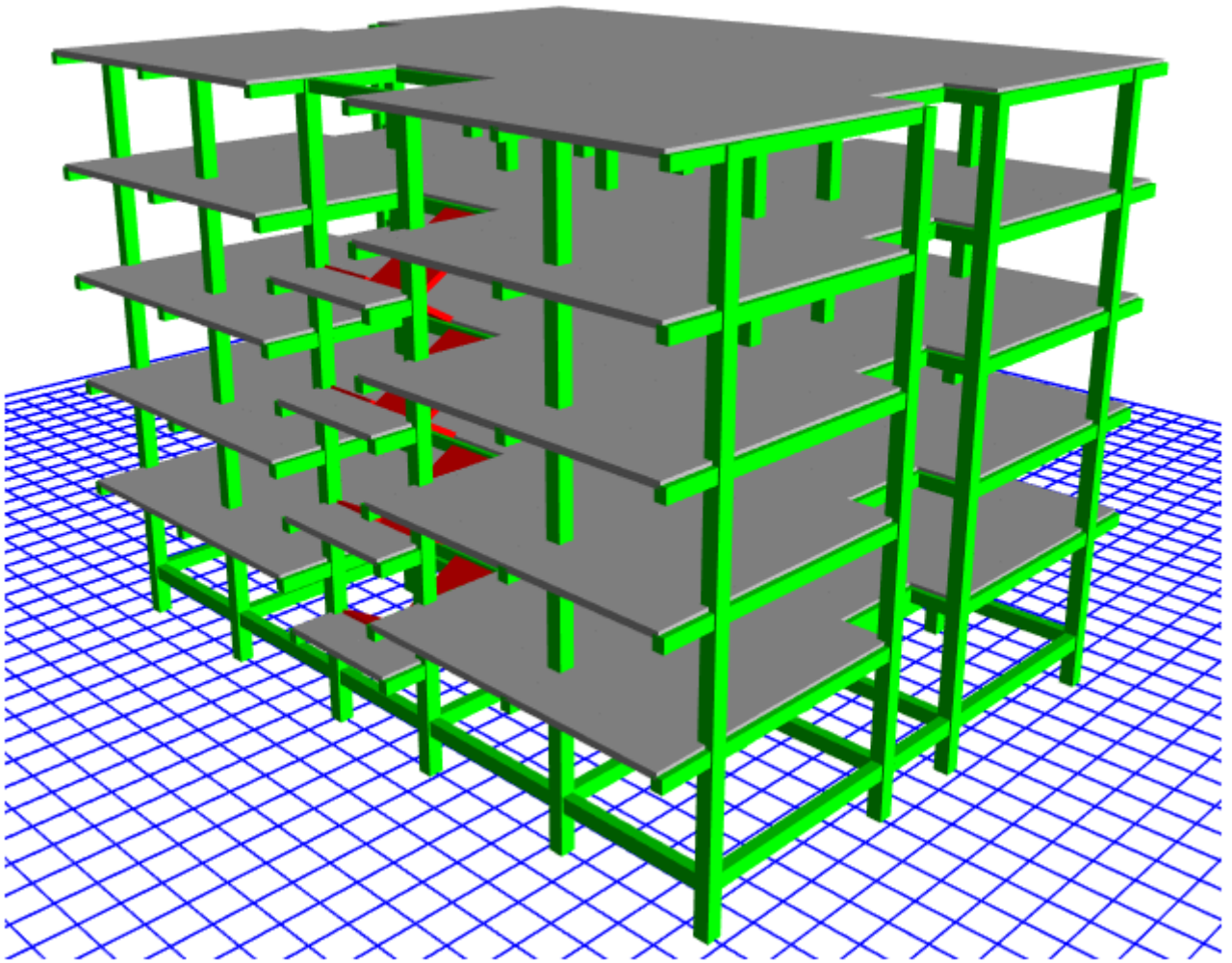


Figure 6.1 3D model of project

Frame is analyzed to get maximum shear and moments in the structure by using ETABs2017 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 Architectural drawing data.
3. Define Story Levels: - define story height and label them.

4. Define material Properties: - define material properties that we used in overall 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 for slab and beam

Modules of elasticity – 30 GPa

B. Concrete: -Grade – C30 for Column

Modules of elasticity – 31 GPa

Weight per unit volume- 25KN/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= $E/2(1+\nu)$.

C. Rebar

Grade – S400

Mass per unit volume = 7850Kg/m³ Modulus of

elasticity =200GPa=200000MPa

Coefficient of Thermal expansion=0.0000117

5. Define Section properties: - for initial we define column, beam and slabsections.
 - a. Beam
 - Grade Beam Cross section 300x400
 - Intermediate Beam Cross section 300x350
 - Top tie Beam Cross section 250x300
 - b. Column: - Cross section 400x400
 - c. Slab: - depth of 200mm
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 2 Load combination

Combinations	Factored load combinations					
	DL	Super DL	Partition L	LL		
Comb 1	DL	Super DL	Partition L	LL		
Comb 2	1.35DL	1.35 Sup DL	1.35 Part. L	1.5LL		
Comb 3	DL	1.35 Sup DL	1.35 Part. L	0.3LL	EQXP	0.3EQYN
Comb 4	DL	Super DL	Partition L	0.3LL	EQXP	-0.3EQYN
Comb 5	DL	Super DL	Partition L	0.3LL	-EQXP	0.3EQYN
Comb 6	DL	Super DL	Partition L	0.3LL	-EQXP	-0.3EQYN
Comb 7	DL	Super DL	Partition L	0.3LL	EQXP	0.3EQYP
Comb 8	DL	Super DL	Partition L	0.3LL	EQXP	-0.3EQYP
Comb 9	DL	Super DL	Partition L	0.3LL	-EQXP	0.3EQYP
Comb 10	DL	Super DL	Partition L	0.3LL	-EQXP	-0.3EQYP
Comb 11	DL	Super DL	Partition L	0.3LL	EQXN	0.3EQYN
Comb 12	DL	Super DL	Partition L	0.3LL	EQXN	-0.3EQYN
Comb 13	DL	Super DL	Partition L	0.3LL	-EQXN	0.3EQYN
Comb 14	DL	Super DL	Partition L	0.3LL	-EQXN	-0.3EQYN
Comb 15	DL	Super DL	Partition L	0.3LL	EQXN	0.3EQYP
Comb 16	DL	Super DL	Partition L	0.3LL	EQXN	-0.3EQYP
Comb 17	DL	Super DL	Partition L	0.3LL	-EQXN	0.3EQYP
Comb 18	DL	Super DL	Partition L	0.3LL	-EQXN	-0.3EQYP
Comb 19	DL	Super DL	Partition L	0.3LL	EQYP	0.3EQXN
Comb 20	DL	Super DL	Partition L	0.3LL	EQYP	-0.3EQXN
Comb 21	DL	Super DL	Partition L	0.3LL	-EQYP	0.3EQXN
Comb 22	DL	Super DL	Partition L	0.3LL	-EQYP	-0.3EQXN
Comb 23	DL	Super DL	Partition L	0.3LL	EQYP	0.3EQXP
Comb 24	DL	Super DL	Partition L	0.3LL	EQYP	-0.3EQXP
Comb 25	DL	Super DL	Partition L	0.3LL	-EQYP	0.3EQXP
Comb 26	DL	Super DL	Partition L	0.3LL	-EQYP	-0.3EQXP
Comb 27	DL	Super DL	Partition L	0.3LL	EQYN	0.3EQXN
Comb 28	DL	Super DL	Partition L	0.3LL	EQYN	-0.3EQXN
Comb 29	DL	Super DL	Partition L	0.3LL	-EQYN	0.3EQXN
Comb 30	DL	Super DL	Partition L	0.3LL	-EQYN	-0.3EQXN
Comb 31	DL	Super DL	Partition L	0.3LL	EQYN	0.3EQXP
Comb 32	DL	Super DL	Partition L	0.3LL	EQYN	-0.3EQXP
Comb 33	DL	Super DL	Partition L	0.3LL	-EQYN	0.3EQXP
Comb 34	DL	Super DL	Partition L	0.3LL	-EQYN	-0.3EQXP
Comb 35	1.35DL	1.35 Sup DL	1.35 Part. L	1.5LL	0.9WL	
Comb 36	1.35DL	1.35 Sup DL	1.35 Part. L	1.5LL	-0.9WL	
Comb 37	1.35DL	1.35 Sup DL	1.35 Part. L	1.5WL		
Comb 38	comb 3- comb 18					
Comb 39	comb 19- comb 34					
Comb 40	comb 1- comb 37					

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 27KN/m^3 and thickness of 20mm

Load of floor finish= $27 * 0.02 = 0.54 \text{ KN/m}^2$

Assume ceiling plastering and cement screed thickness=30mm and unit

weight of 23 KN/m^3

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

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

Super dead load= $0.54 + 0.69 + 0.69 = \underline{\underline{1.23\text{KN/m}^2}}$

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

Partition load= $0.15\text{m} * 2.8\text{m} * 14\text{KN/m}^3 = \underline{\underline{5.88 \text{ KN/m}}}$

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: - analyses the model to get moment and shear results.

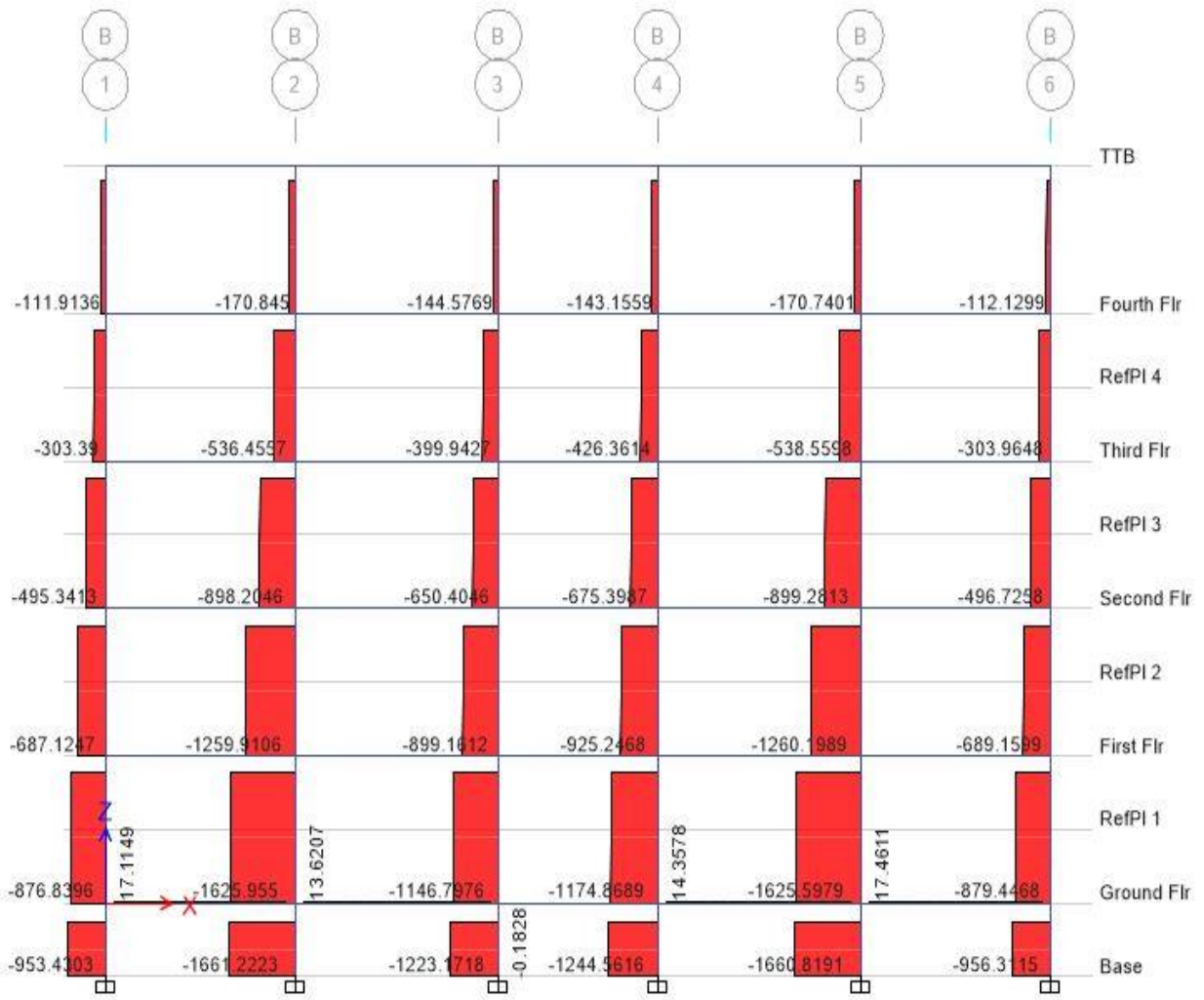


Figure 6.2 Axial load result sample generated from ETABS

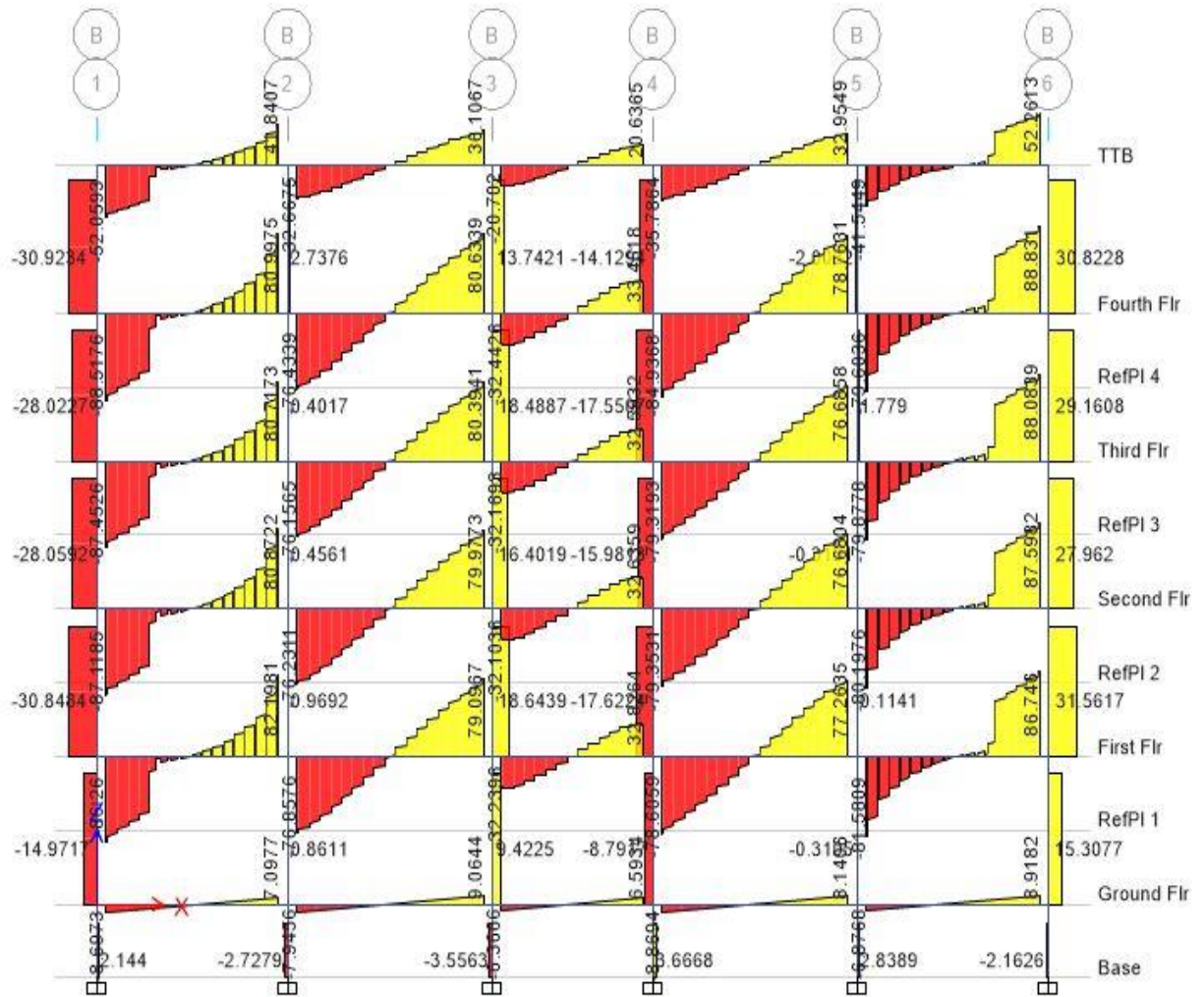


Figure 6.3 Shear force result sample generated from ETABS

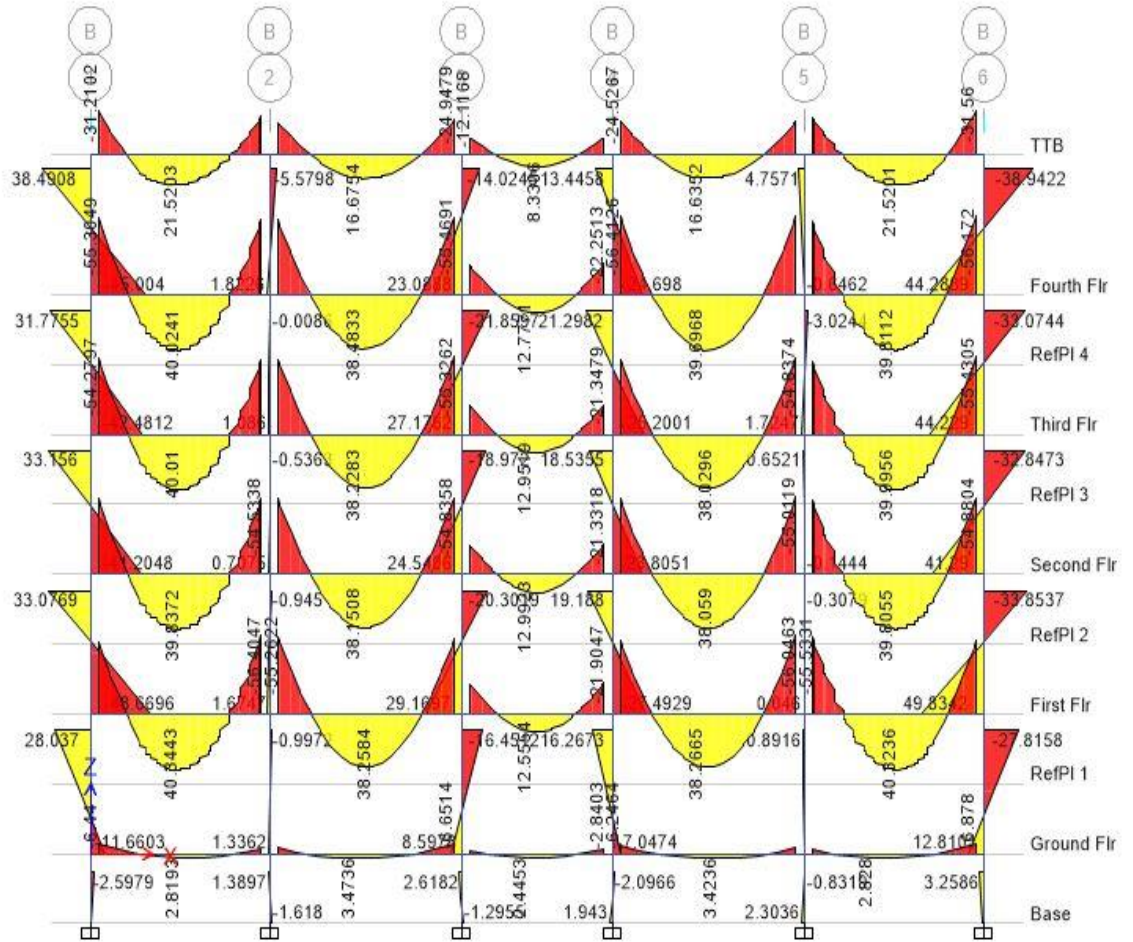


Figure 6.4 Moment result sample generated from ETABS

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, overreinforced 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

Beam Dimension

Grade Beam Cross section 300x400

Intermediate Beam Cross section 300x350

Top tie Beam Cross section 250x300

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, $f_{cd} = \alpha_{cc} * \frac{f_{ck}}{\gamma_c} = 0.85 * \frac{25}{1.5} = 11.33$

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.

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

- m. 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)
- n. Minimum strength class = C20/25, for exposure class XC1[Appendix B,]
- o. Minimum concrete cover for bond/durability is 12 mm

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10 \text{ mm} \end{array} \right\} = \max \left\{ \begin{array}{l} 20 \text{ mm} \\ 10 \text{ mm} \\ 10 \text{ mm} \end{array} \right\} = 20\text{mm}$$

$C_{min,b} = 20$ mm, assumed diameter of bar (Appendix B, Table B4). $C_{min,dur} = 10$ 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. The recommended minimum Structural Class is S1 (Appendix B, Table B5).

p. Nominal cover

$$c. C_{nom} = C_{min} + \delta c_{dev} = 20\text{mm} + 10\text{mm} = 30\text{mm}$$

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.

q. Minimum cover Design for Fire = 20mm

r. Governing concrete cover = 30mm

$\Delta C_{dur,st}$ -reduction of minimum cover for use of stainless steel

$\Delta C_{dur,add}$ -reduction of minimum cover for use of additional protection

But; the recommended value of $\Delta C_{dur,Y}$, $\Delta C_{dur,st}$, and $\Delta C_{dur,add}$ -is zero

➤ $C_{min,b}$ minimum cover due to bond requirement

7.2 Check depth for deflection

For flexure

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

$$N = K \left[11 + 1.5 \sqrt{\frac{f_{ck}}{\rho_o}} \rho_o / (\rho - \rho^{''}) + \frac{1}{12} * \sqrt{\frac{f_{ck}}{\rho_o}} \sqrt{\frac{\rho'}{\rho_o}} \right] \quad \text{If } \rho > \rho_o$$

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, $8*1.25=10$

- check the whether the beam single or double reinforced

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

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

Sample calculation 31.21 KNm at top tie beam axis A along 1

$$\text{Where } K = \frac{Msd}{fckbd^2} = \frac{31.21 \cdot 10^6}{20 \cdot 250 \cdot 300 \cdot 300} = 0.069, 0.069 < 0.167 \text{ single reinforcement}$$

- Calculate reinforcement

Let calculate for axis B top tie beam reinforcement on support location axis 6

$Msd = 31.21$

check the minimum and maximum reinforcement

$$Ast_{min} = \max \left\{ \begin{array}{l} 0.26 \left(\frac{f_{ctm}}{f_{yk}} \right) bd = 0.26 \left(\frac{2.2}{400} \right) 250 \cdot 300 = 107.25 \text{mm}^2 \\ 0.0013bd \quad \quad \quad 0.0013 \cdot 250 \cdot 300 = 97.5 \text{mm}^2 \end{array} \right.$$

$$Ast_{min} = 107.25 \text{mm}^2$$

$$Ast, \max = 0.04bd = 0.04 \cdot 250 \cdot 300 = 3000$$

$$Ast, \text{cal} = \frac{Msd}{zfyd} = \frac{31.21}{zfyd}$$

$$z = \frac{d}{2} \left(1 + \sqrt{1 - 3 \cdot 53k} \right)$$

$$z = \frac{300}{2} \left(1 + \sqrt{1 - 3 \cdot 53 \cdot 0.114} \right) = 280.46$$

$$Ast, \text{cal} = \frac{Msd}{zfyd} = \frac{31.21 \cdot 10^6}{280.46 \cdot 347.83} = 319.93 \text{mm}^2$$

Therefore $Ast, \min \leq Ast, \text{cal} \leq Ast, \max$, $A_{st} = 319.93 \text{mm}^2$

If we use $\phi 16$

$$A_{st} = 3.14 \cdot 16^2 / 4 = 200.96$$

no of bars= $A_{st}/a_{st}=319.93\text{mm}^2/200.96= 1.59$ USE 2 $\phi 16$

we should to give minimum reinforcement 2 $\phi 16$

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

Lap length

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

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

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

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

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

$\eta_1 = 1$ for good quality of bond

$\eta_1 = 0.7$ for other cases

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

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

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

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

$f_{ctk,0.05} = 1.5$ for C20/25 Mpa, from ESEN 1992

Sample calculation for $\varnothing 10$ reinforcement bar

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

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

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

$\eta_1 = 0.7$ for other cases

$$f_{bd} = 2.25 \cdot 1 \cdot 1 \cdot 0.85 = 1.91$$

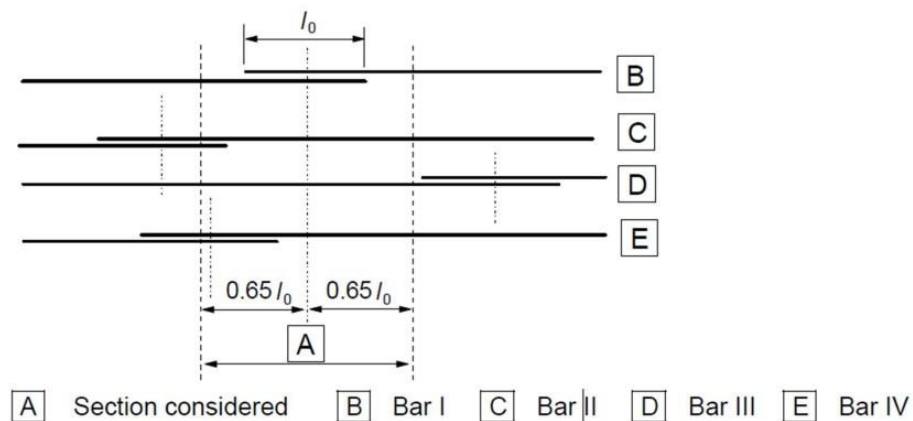
$$\sigma_{sd} = \frac{400}{1.15} = 347.83$$

$$l_{b,rqd} = \frac{\varnothing \cdot \sigma_{sd}}{4 \cdot f_{bd}} = \frac{16 \cdot 347.83}{4 \cdot 1.91} = 728.44\text{mm}$$

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

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

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

$$l_{o,min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} & 0.3 * 1.5 * 728.44 = 327.80\text{mm} \\ 15\phi & 15\phi = 15 * 16 = 240\text{mm} \\ 200\text{mm} & 200\text{mm} \end{cases}$$

$$l_{o,min} = 327.80\text{mm}$$

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

$$l_0 = 1 * 1 * 1 * 1.5 * 728.44 = 1092.66 \geq 327.8$$

$$\text{lap length } l_0 = 1100\text{mm}$$

7.2.1 Anchorage length

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

For anchorage in tension

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

$$l_{b,min} \geq \max \begin{cases} 0.3 * 728.44 = 218.53\text{mm} \\ 10 * 16 = 160\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,min} = 218.53\text{mm}$$

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_5$$

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

$$l_{bd} = 1 * 1 * 1 * 728.4 = 728.44 > 218.53\text{mm}$$

For anchorage in Compression

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

$$l_{b,\min} \geq \max \begin{cases} 0.6 * 728.44 = 437.06\text{mm} \\ 10 * 16 = 160\text{mm} \\ 100\text{mm} \end{cases}$$

$$l_{b,\min} = 437.06$$

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

$$l_{bd} = 1 * 1 * 1 * 728.44 = 728.44 > 437.06 \text{ mm Use } \mathbf{730\text{mm}}$$

7.2.2 Beam 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:

$$V_{Rd,c} = [0.12k(100\rho_1f_{ck})^{1/3}]$$

With minimum of

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

$$V_{Rd,c} = [0.12k(100\rho_1f_{ck})^{1/3}]$$

f_{ck} is in MPa

$$1 + \left(\frac{200}{d}\right)^2 < 2$$

$$\rho_1 = \frac{N_{ED}}{bwd} < 0.2f_{cd}[\text{MPa}]$$

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

b_w 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_{ED} may be ignored.

A_c is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is in newtons

The values of V_{\min} and k_1 for use in a country may be found in its National Annex. The recommended value 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

$$V_{rd,s} = \frac{A_{sw}}{s} Z f_{yw} d \cot\theta$$

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

Where

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

reinforcement s is the spacing of the stirrups

f_{yw} 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:

$$V = 0.6$$

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

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

$$\rho_w = \frac{A_{sw}}{(s \cdot b_w \cdot \sin\alpha)}$$

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 ρ_{wmin} for beams for use in a Country may be found in its National

Annex. The recommended value is $\rho_{wmin} = (0.08\sqrt{f_{ck}})/f_{yk}$

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 $S_{max} = 0.75(1 + \cot\alpha)$

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, V_{ed}

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:

$$\theta = 0.5 \sin^{-1} \left[\frac{2V_{ED}}{\alpha_c b_w Z v_{fcd}} \right]$$

Step 5: If $\cot \theta$ is less than 1, re-size element, otherwise go to step

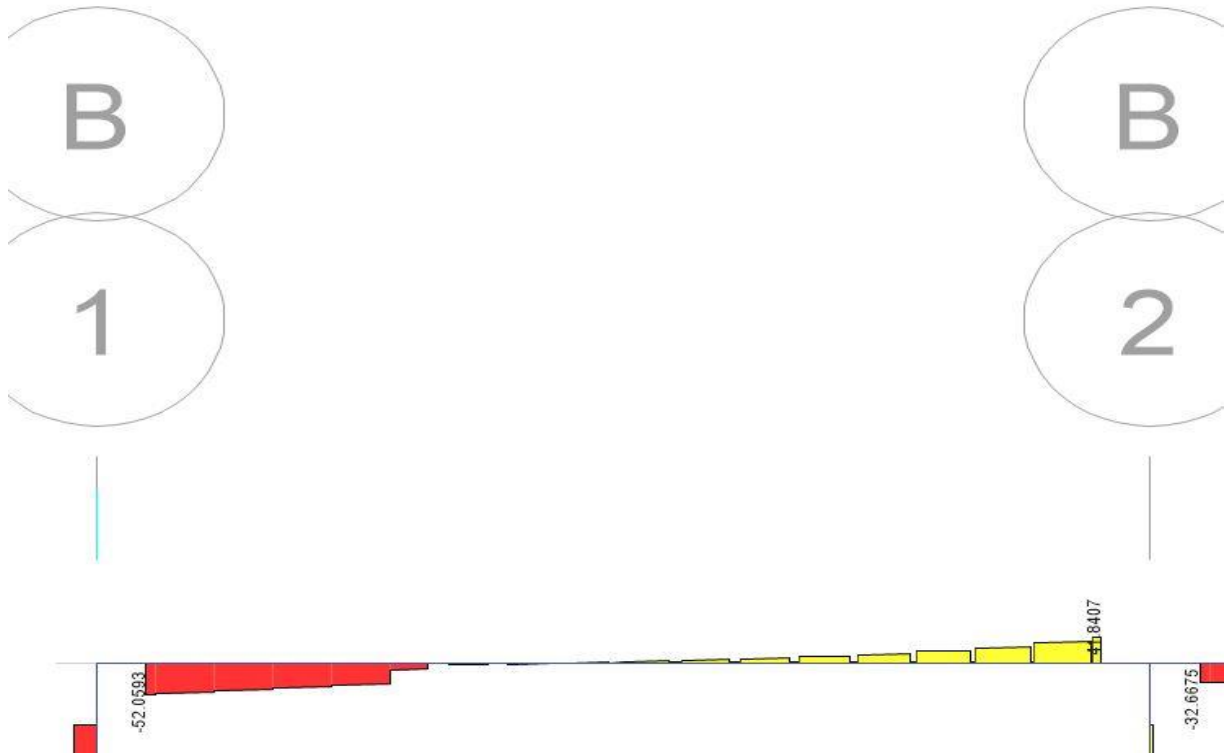
6 Step 6: Calculate $V_{Rd,c}$

Step 7: If $v_{rd,c} > v_{ed}$, Minimum shear reinforcement is sufficient

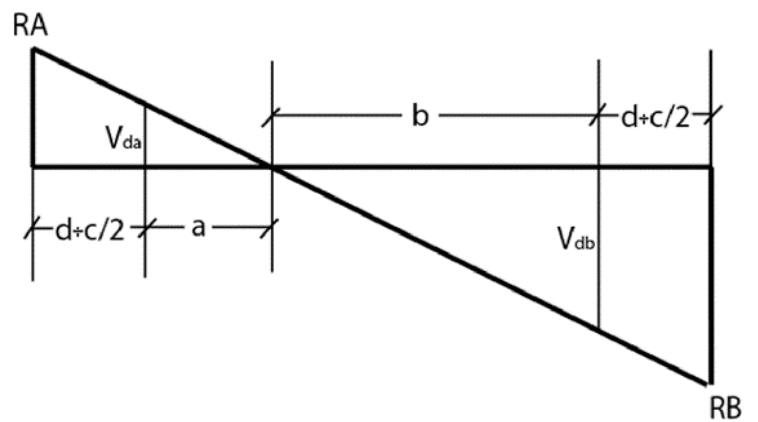
Step 8: If $v_{rd,c} < v_{ed}$, Calculate amount of shear reinforcement required Step

9: Check min shear reinforcement and maximum spacing

Sample calculation for shear design



RA=	52.06 KN
RB=	41.84 KN
L=	4.3 m
d=	300 mm
C=	300 mm
D+c/2	0.45 m
a=	1.58269m
b=	1.81731m
VE _{da} =	40.53488 KN
VE _{db} =	33.53589 KN
VE _d =	40.53488KN



Concrete Capacity check

$$K1 = 1 + \left(\frac{200}{d}\right)^{1/2} \leq 2$$

$$D = 300$$

$$B = 250$$

$$A_{sl} = 200.96$$

$$k = 1.816$$

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

Taking minimum reinforcement ϕ 8 @ 260

$$\rho_1 = 0.00268$$

$$V_{Rd,c} = \left[0.12k(100\rho_1f_{ck})^{\frac{1}{3}} \right]bd$$

$$v_{Rd,c} = 28.60$$

$v_{Rd,c} = V_{Rd,c} \leq V_{Ed}$, Shear use min spacing, 15*long. Bar dia.

Longitudinal bar diam. = 16 reinforcement is required

Minimum spacing = 240

Diagonal compression check

$$V_{Rd,max} = a_{cw} * b_w * ZV_1f_{cd} \left(\frac{1}{\cot\theta + \tan\theta} \right)$$

$$V_{Rd,c} = \left[0.12(100\rho_1f_{ck})^{\frac{1}{3}} \right]bd \geq V_{min}$$

$$v_1 = 0.6 \left(1 - \frac{f_{ck}}{250} \right)$$

$$V_1 = 0.552$$

$$z = 0.9d$$

$$Z = 270$$

$$\cot\theta = 2.5$$

$$\tan\theta = 0.4$$

$$V_{Rd,max} = 145.43\text{KN}$$

$V_{Rd,max} > V_{Ed}$, 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 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.

$$\rho_{wmin} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} = \frac{0.08\sqrt{20}}{400} = 0.00089$$

➤ The maximum longitudinal spacing of bent-up bars should not exceed $S_{b,max}$

$$S_{b,max} = 0.75d(1 + \cot\alpha) = 0.6 * 300(1 + \cot 90) = 180\text{mm}$$

$$S_{min} = \frac{A_{sw}}{\rho_w b_w \sin\alpha} = \frac{50.24}{0.00089 * 250 * 1} = 225.8$$

$$\text{Required shear reinforcement} = \frac{V_{ED}}{0.9d f_{yk} \cot\theta} = 0.00089$$

$$S_{calc} = \frac{A_{sw} 0.78d f_{yk} \cot\theta}{V_{ED}} = \frac{50.24 * 0.9 * 300 * 400}{40.54} = 133.84\text{mm}$$

Use $s = 130\text{mm}$

TTB-1 (300X250) AXIS-B (1Pc)

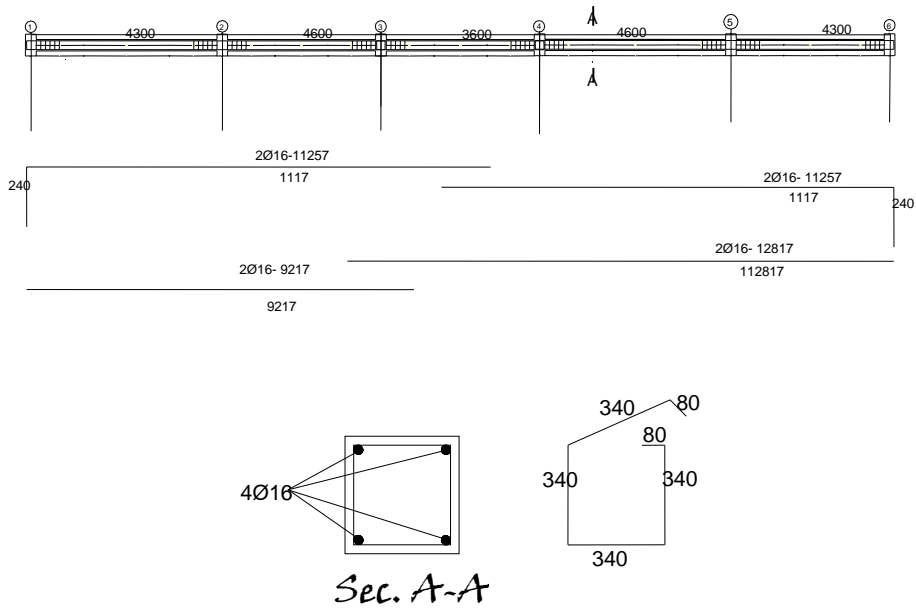


Figure 7.1 Beam Reinforcement Detailing

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 crushing. 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 into 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 on 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 contains bracing elements. These are vertical elements usually walls, which are so stiff relative to other vertical elements

That may be assumed to be attracting 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

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.

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

- a) Exposure class = XC2,
- b) Minimum strength class = C25/30, for exposure class XC2[Appendix B,]
- c) Minimum concrete cover for bond/durability is 12 mm

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10 \text{ mm} \end{array} \right\} = \max \left\{ \begin{array}{l} 20 \text{ mm} \\ 10 \text{ mm} \\ 10 \text{ mm} \end{array} \right\} = 20 \text{ mm}$$

$C_{min,b} = 20 \text{ mm}$, assumed diameter of bar (Appendix B, Table B4). $C_{min,dur} = 10 \text{ mm}$,

Note: 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. But based on the above table the exposure class is reducing by 1 and the structural class would be S3.

Therefore, the value of minimum cover required for durability of reinforcement steel is determined using (Appendix B, Table B5).

Depending on the above case and table 4.4N : values of minimum cover, $C_{min,dur}$ requirements with regard to durability for reinforcement steel in accordance with EN 1080

d. Nominal cover

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

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.

e. Minimum cover Design for Fire = 20mm

f. 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_0) for braced member From (*ES EN 1992-1-1*, 2015)

$$l_0 = 0.5l * \sqrt{\left(1 + \frac{K_1}{0.45+k_1}\right) * \left(1 + \frac{K_2}{0.45+k_2}\right)}$$

Where, K_1, K_2 -relative flexibilities of rotational restraint at both ends 1, 2-

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

$$K = \frac{\text{Column stiffness}}{\sum \text{Beam stiffness}}$$

$$K_i = \frac{\frac{EI}{(T)^c}}{\left(\frac{2*EI}{1}\right)_b}$$

8.1. 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:

$$A = \sqrt{(1 + 0.2ef)} \text{ (if } ef \text{ is not known, } A = 0.7 \text{ may be used)}$$

$$B = 1 / (1 + 0.2 ef) \text{ (if } ef \text{ is not known, } A = 0.7 \text{ may be used)}$$

$$B = dg \sqrt{1 + 2\omega} \text{ (if } \omega \text{ is not known, } B = 1.1 \text{ may be used)}$$

$$C = 1.7 - r_m \text{ (if } r_m \text{ is not known } C = 0.7 \text{ may be used)}$$

ef effective creep ratio; see 5.8.4:

$$ef = A_s f_{yd} / (A_c f_{cd}); \text{ mechanical reinforcement ratio;}$$

A_s is the total area of longitudinal reinforcement

$$n = N_{Ed} / (A_c f_{cd}); \text{ relative normal force}$$

$$r_m = M_{01} / M_{02}; \text{ moment ratio}$$

$$M_{01}, M_{02} \text{ are the first order end moments, } |M_{02}| \geq |M_{01}|$$

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

$$e_i = \begin{cases} \frac{i_o}{400} \\ \frac{h}{30} \\ 20\text{mm} \end{cases}$$

In the both direction

8.2. Calculate reinforcement

$$A_{s, tot} = \frac{\omega A_c * f_{cd}}{f_{yd}}$$

, is taken design chart by using, V_{sd} , $\mu_{sd,x}$, $\mu_{sd,y}$ from (ES EN 1992-1-1, 2015)

$$A_{s, tot} = \frac{N_{sd}}{A_c f_{cd}}$$

$$\mu_{sd,x} = \frac{M_{EDx}}{f_{cd} * A_c * h}$$

$$\mu_{sd,y} = \frac{M_{EDy}}{f_{cd} * A_c * h}$$

If the column is slender column

$$M_{ED} = \max \{M_{02}; M_{0e} + M_2; M_{01} + 0.5M_2; N_{ED}\}$$

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

$$A_{s, min} = \max \left\{ \begin{array}{l} 1 + \frac{N_{ED}}{f_{yd}} \\ 0.002 * A_c \end{array} \right.$$

$$A_{s,max} = 0.04A_c$$

For shear reinforcement

$$Spacong = \min \left\{ \begin{array}{l} 20 * \phi_{long} \\ \min \text{ lesser dimension of column} \\ 400mm \end{array} \right.$$

Sample calculation

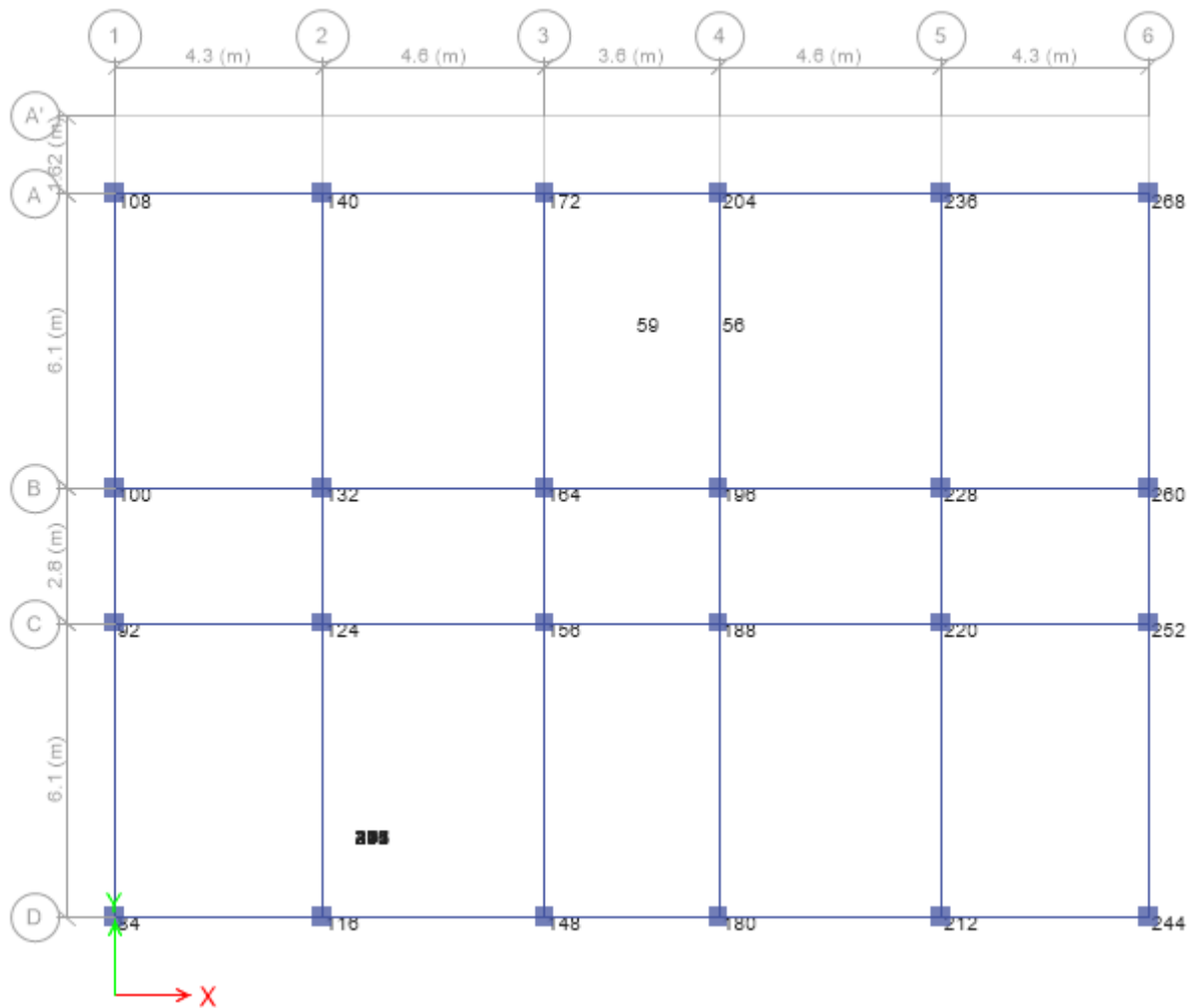
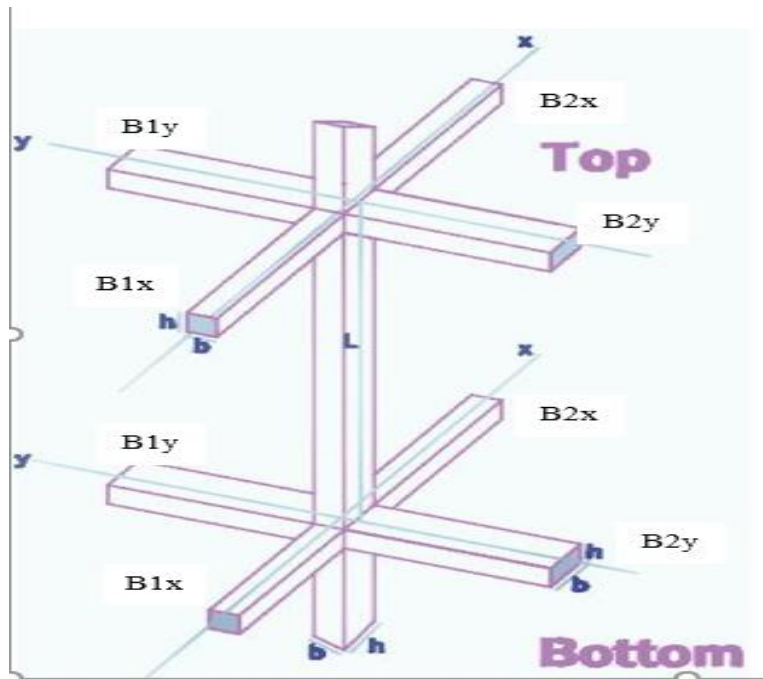


Figure 8.1 column label layout

Effective length determination

Let calculate for Column at axis 5 along B

Given Data



Element	Depth 'h' (Y-dxn) (mm)	width 'b' (X-dxn)(mm)	length 'L' (mm)
column	400	400	2800
Top beam			
B1 (x-x) (cantilever)	300	250	4600
B2 (x-x) (pin)	300	250	3600
B1 (y-y) (cantilever)	300	250	2800
B2 (y-y) (pin)	300	250	6100
Bott beam			
B1 (x-x) (continous)	350	300	4600
B2 (x-x) (pin)	350	300	3600
B1 (y-y) (continous)	350	300	2800
B2 (y-y) (continous)	350	300	6100

$$l_{bxl} = 5630\text{mm}$$

$$l_{bxR} = 4180\text{mm}$$

$$l_{byl} = 4930$$

$$l_{bxR} = 4170$$

$$l_c = 2800$$

Beam = 250 X 300

Column = 400 x 400

$$l_o = 0.5l * \sqrt{\left(1 + \frac{k_1}{0.45+k_1}\right) \cdot \left(1 + \frac{k_2}{0.45+k_2}\right)}$$

$$K_{1x} = \frac{\frac{400 * 400^3}{12}}{2800} = \frac{380952.38}{\left(\frac{250 * 300^3}{12}\right) \left(\frac{250 * 300^3}{12}\right)} = \frac{380952.38}{79928.952 + 107655.5024} = 1.44$$

Same for $K_{2x}=1.44$

$$K_{1y} = \frac{\frac{400 * 400^3}{12}}{2800} = 1.36$$

$$\left(\frac{250 * 300^3}{12}\right) \left(\frac{250 * 300^3}{6100}\right)$$

Same for $K_{2y}=1.36$

$$l_{ox} = 0.5 * 2800 * \sqrt{\left(1 + \frac{1.44}{0.45+1.44}\right) \cdot \left(1 + \frac{1.44}{0.45+1.44}\right)} = 2465.87$$

$$l_{oy} = 0.5 * 2800 * \sqrt{\left(1 + \frac{1.36}{0.45+1.36}\right) \cdot \left(1 + \frac{1.36}{0.45+1.36}\right)} = 2452.73$$

Check slenderness limit

$$\text{Slenderness ratio } i = \sqrt{\frac{I_c}{A}}$$

$$i = \sqrt{\frac{\frac{400*400^3}{12}}{400*400}} = 115.47$$

$$\lambda_x = \frac{l_{ox}}{i} = \frac{2465.87}{115.47} = 24.41$$

$$\lambda_y = \frac{l_{oy}}{i} = \frac{2452.73}{115.47} = 24.28$$

The minimum limiting value of slenderness is

$$\lambda_{lim} = \frac{20 * ABC}{\sqrt{n}}$$

\square_{ef} is not known

$$A=0.7 \quad B=1.1$$

$$C= 1.7 - r_m$$

$$r_m = \frac{M_{01}}{M_{02}}$$

$$M_{01} = \min \{|M_{top}|, |M_{bottom}|\} + e_i * NED$$

$$M_{02} = \{|M_{top}|, |M_{bottom}|\} + e_i * NED$$

Accidental eccentricity

$$\frac{144.46}{400} = 5.75$$

$$e_{ix} = \max \left\{ \begin{array}{l} \frac{i_o}{400} \quad \frac{2465.07}{400} = 6.36 \\ \frac{h}{30} \quad \frac{400}{30} = 13.33 \\ 20mm \quad 20mm \end{array} \right. \quad e_{ix} = 20mm$$

$$e_{iy} = \max \left\{ \begin{array}{l} \frac{i_o}{400} \quad \frac{2452.73}{400} = 6.33 \\ \frac{h}{30} \quad \frac{400}{30} = 13.33 \\ 20mm \quad 20mm \end{array} \right. \quad e_{iy} = 20mm$$

$$NED \text{ (axial force)} = 353.16KN$$

$$M_{01x} = \min \{|-27.77|, |41.87|\} + 0.02 * 144.46 = 44.87$$

$$M_{02x} = \{|-27.77|, |41.87|\} + 0.02 * 144.46 = 51.51$$

$$M_{01y} = \min \{|33.74|, |-36.89|\} + 0.02 * 144.46 = 14.94$$

$$M_{02y} = \{|33.74|, |-36.89|\} + 0.02 * 144.46 = 26.02$$

$$n = \frac{NED}{A_{cfcd}} = \frac{144.46}{400 * 400 * 14.17} = 0.084$$

$$\lambda_{xlim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{44.87}{51.51})}{\sqrt{0.064}} = 50.55$$

$$\lambda_{ylim} = \frac{20 * 0.7 * 1.1 * (1.7 - \frac{14.94}{26.02})}{\sqrt{0.064}} = 68.67$$

24.41 < 50.55, column is not slender

24.28 < 68.67, column is not slender

Calculate reinforcement

$$A_{s, tot} = \frac{\omega * A_c * f_{cd}}{f_{yd}}$$

, is taken design chart by using, V_{sd} , $\mu_{sd,x}$,
 $\mu_{sd,y}$ from (ES EN 1992-1-1, 2015)

$$V_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{144.46 * 1000}{400 * 400 * 14.17} = 0.06$$

$$\mu_{sd,x} = \frac{MEDX}{A_c * f_{cd} * h} = \frac{51.50 * 1000000}{400 * 400 * 400 * 14.17} = 0.06$$

$$\mu_{sd,y} = \frac{MEDY}{A_c * f_{cd} * h} = \frac{26.02 * 1000000}{400 * 400 * 400 * 14.17} = 0.03$$

ω , is taken from biaxial chart on [appendix F] for this sample $\omega = 0.12$

Minimum and maximum area of reinforcement

$$A_{s, min} = \max \left\{ \begin{array}{l} \left(0.1 * \frac{NED}{f_{yd}} \right) \\ 0.002 * A_c \end{array} \right.$$

$$A_{s, min} = \max \left\{ \begin{array}{l} \left(0.1 * \frac{144.46}{\frac{400}{1.15}} \right) * 1000 = 101.53 \\ 0.002 * 400 * 400 = 320 \end{array} \right.$$

$$A_{s, min} = 320$$

$$A_s, \max = 0.04A_c = 0.04 * 400 * 400 = 6400$$

$$A_s, \text{tot} = \frac{\omega * A_c * \frac{f_{cd}}{1.5}}{f_{yd}} = \frac{0.12 * 400 * 400 * \frac{25}{1.5}}{347.83} = 781.62$$

Let us use ϕ 16 bar

$$\text{No of bar} = \frac{A_{st}}{a_{st}} = \frac{781.62}{\frac{3.14 * 20 * 20}{4}} = 2.49$$

Use 4 ϕ 20 bar

For shear reinforcement

Take ϕ 8mm

$$S_{pacong} = \min \begin{cases} 20 * \phi_{long} = 20 * 20 = 400 \\ \min \text{ lesser dimension of column} = 400 \\ 400mm \end{cases}$$

Use ϕ 8 c/c 400

Lap length

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

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

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

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

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

$\eta_1 = 1$ for good quality of bond

$\eta_1 = 0.7$ for other cases

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

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

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

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

$f_{ctk, 0.05} = 1.5$ for C20/25 Mpa, from ESEN 1992

Sample calculation for $\phi 20$ reinforcement bar

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

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

$\eta_1 = 1$ our slab D is <250mm for good quality of bond

$\eta_1 = 0.7$ for other cases

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

$$\sigma_{sd} = \frac{400}{1.15} = 347.83$$

$$l_{b,rqd} = \frac{\phi * \sigma_{sd}}{4 * f_{bd}} = \frac{20 * 347.83}{4 * 1.91} = 910.55\text{mm}$$

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

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.

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

$$l_{o,min} \geq \max \begin{cases} 0.3\alpha_6 l_{b,rqd} & 0.3 * 1.5 * 910.55 = 409.75\text{mm} \\ 15\phi & 15\phi = 15 * 20 = 300\text{mm} \\ 200\text{mm} & 200\text{mm} \end{cases}$$

$$l_{o,min} = 409.75\text{mm}$$

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

$$l_0 = 1 * 0.7 * 0.7 * 0.7 * 1.4 * 409.75 = 186.76 = 200\text{mm}$$

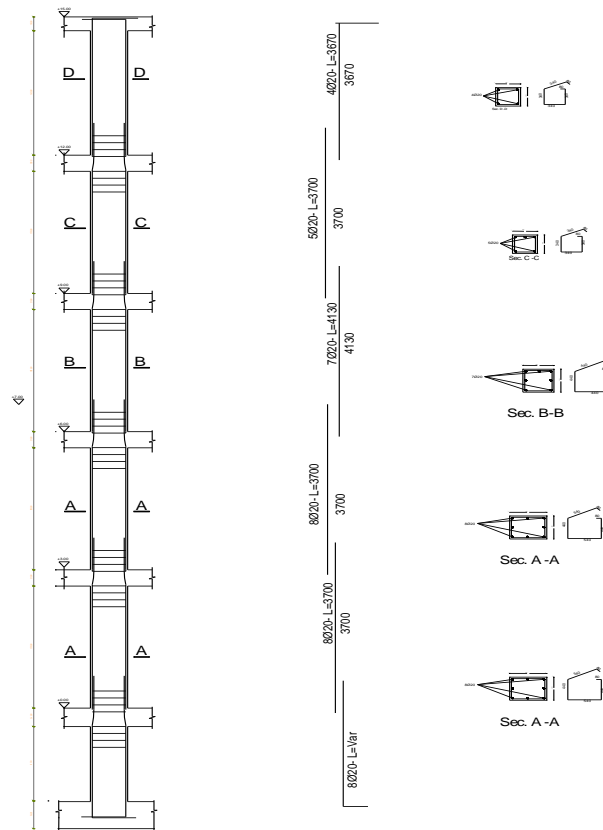


Figure 8.2 Column reinforcement detailing

9 Foundation design

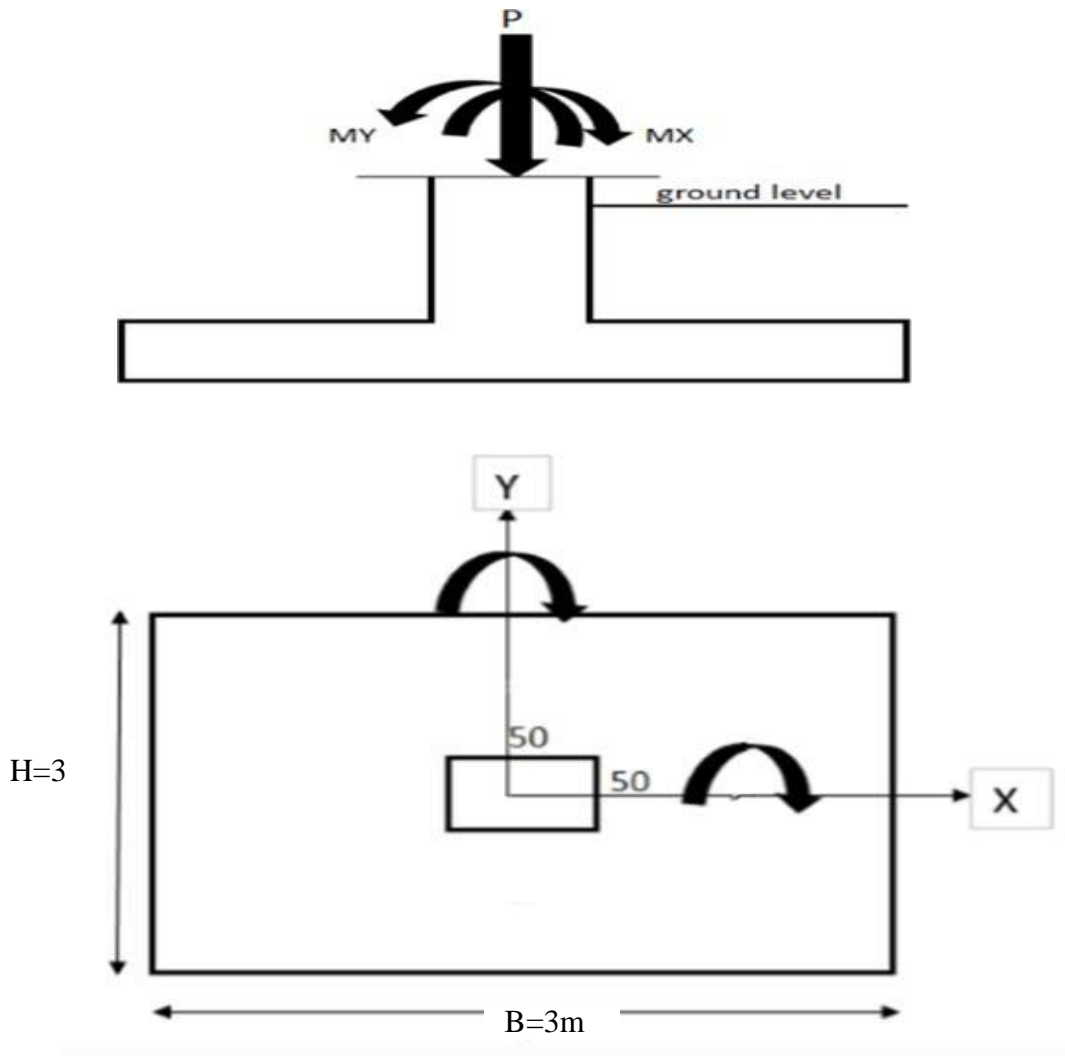
The foundation is the part of an engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and its self-weight.

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



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.

Cover Design

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

Step 1: Concrete cover design

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

- A. Exposure class = XC2, wet, rarely dry, concrete surfaces subjected to long term water contact, Many foundations. [Appendix B]
- B. Minimum strength class = C25/30, for exposure class XC2 [AppendixB]
- C. Minimum concrete cover for bond/durability

$$C_{min} = \max \left\{ \begin{array}{l} C_{min, B} \\ C_{min, Dur} \\ 10 \text{ mm} \end{array} \right\} = \max \left\{ \begin{array}{l} 24 \text{ mm} \\ 15 \text{ mm} \\ 10 \text{ mm} \end{array} \right\} = 24 \text{ mm}$$

$C_{min, b} = 24 \text{ mm}$, assumed diameter of bar [Appendix

B] $C_{min, dur} = 15 \text{ mm}$,

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 $C_{min} = 24 \text{ mm}$

A. Nominal cover

$$C_{nom} = C_{min} + \delta c_{dev} = 24 \text{ mm} + 10 \text{ mm} = 34 \text{ mm}$$

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

- e) Minimum cover Design for Fire = 20mm
- f) Governing concrete cover = 34mm \approx 40mm

9.1 Design of Footing Pad

The footing for a single column may be made square in plan, but where there is a large moment acting about one axis it may be more economical to have a rectangular base.

The safe bearing pressure value is a serviceability value as it is used to control settlement of the foundation.

Design Information The stress distribution (contact pressure distribution) is assumed to be linear & assuming square footing and from the allowable bearing capacity of the soil,

For Footing on axis 2 along D

- Depth of foundation = 1.5m
- Allowable bearing capacity = 280kPa

Consider a SQUARE footing pad under column on axis

From frame analysis using ETABS we have the following foundation column axial forces

Story	Joint label	load case /combo	Fx	Fy	Fz	Mx	My	Mz
Base	117	Comb1 - Service	1.05	0.07	1401.61	-8.05	0.64	-0.03
Base	117	Comb2 - Ultimate	1.44	-0.075	1919.37	-11.01	0.89	-0.04

Axial load, P = 1401.61 kN... .. For the serviceability limit state

Axial load, P = 1919.37 kN... .. For the ultimate limit state

9.2 Area proportioning

For the serviceability limit state

$$P/A \leq \sigma_{\text{allowable}}$$

$$A \geq \frac{P}{\sigma}$$

$$A = 1401.61/280 = 5.00\text{m}^2$$

Provide a 3m X 3 m square footing. → A = 9m²

$$F_s = \frac{(Fz + My + Mz) \text{ of Ultimate}}{(Fz + My + Mz) \text{ of Serviceability}}$$

$$F_s = \frac{(1919.37 + 0.89 + (-0.04))}{(1401.61 + 0.64 + (-0.03))} = 1.369$$

$$Q_{ullo} = F_s \cdot q_{all} = 1.369 \cdot 280 = 383.32$$

9.3 Eccentricity by Serviceability

$$e_x = My/P = 0.64/1401.61 = 0.00046m$$

$$e_y = Mx/P = 8.05/1401.61 = 0.0057m$$

Contact pressure ρ_{max}

$$\rho = \frac{P}{A} * \left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{B}\right)$$

$$\begin{aligned} \rho_{max} &= \frac{1401.61}{9} * \left(1 + \frac{6(0.00046)}{2} + \frac{6(0.0057)}{2}\right) \\ &= 160.54 \end{aligned}$$

$$\rho_{min} = \frac{1401.61}{9} * \left(1 - \frac{6(0.00046)}{2} - \frac{6(0.0057)}{2}\right) = 152.86$$

9.4 Eccentricity by Ultimate Limit state

$$e_x = My/P = 0.89/1919.37 = 0.00046m$$

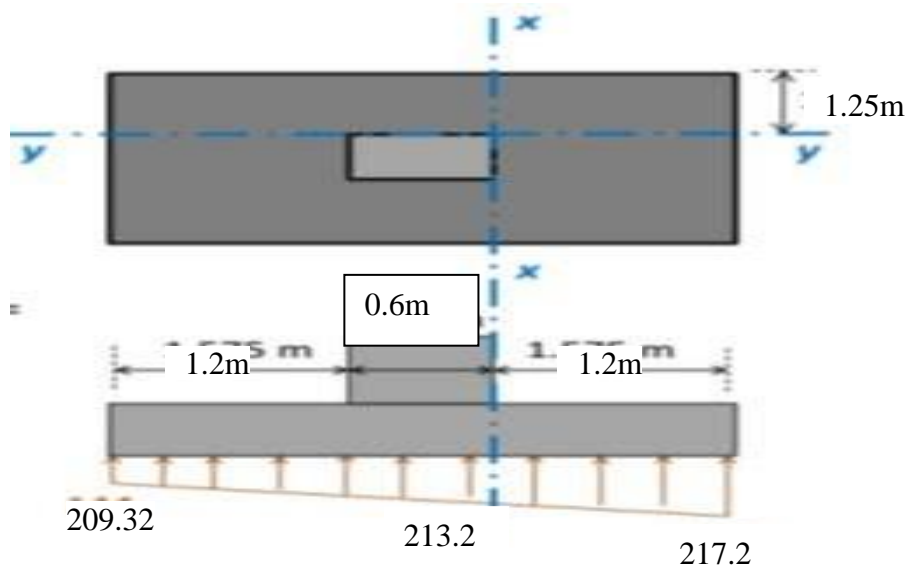
$$e_y = Mx/P = 811.01/1919.31 = 0.0057m$$

Contact pressure ρ_{max}

$$\rho = \frac{P}{A} * \left(1 \pm \frac{6e_y}{B} \pm \frac{6e_x}{B}\right)$$

$$\rho_{max} = \frac{1919.37}{9} * \left(1 + \frac{6(0.00046)}{2} + \frac{6(0.0057)}{2}\right) = 217.2$$

$$\rho_{\min} = \frac{1919.37}{9} * \left(1 - \frac{6(0.00046)}{2} - \frac{6(0.0057)}{2} \right) = 209.32$$



$$M_{xx} = (213.26 * \frac{1.2^2}{2}) + (217.2 - 209.32) * (\frac{1.2}{2}) * (1.2 * \frac{2}{3}) = 157.33 \text{KNm}$$

$$M_{yy} = \frac{(217.2 + 209.32)}{2} * \frac{(1.2 * 1.2)}{2} = 153.54 \text{KNm}$$

Effective depth

$$dx = h - c - 0.5\phi_{\text{bar}} \quad \text{use C-30 and}$$

$$= 600 - 40 - 0.5 * 24$$

$$= 548 \text{mm}$$

$$dy = h - c - 1.5\phi_{\text{bar}} = 600 - 40 - 1.5 * 24$$

$$= 524 \text{mm}$$

9.4.1 Main reinforcement -longitudinal bar

$$K = \frac{M_{xx}}{f_c b d^2} \leq K_{bal}$$

$$K = \frac{157.33}{14.17 * 3 * 600 * 600} = 0.0000103 \leq 0.167$$

Compressive reinforcement is not required

Obtain lever arm Z

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53K}] \leq 0.95d = 0.82d \leq 0.95d$$

$$= 0.82 * 0.60 \leq 0.95 * 0.60$$

$$= 0.57$$

$$A_{s, req} = \frac{(M_{xx})}{0.87 f_{yk} Z} = \frac{157.33 * 10^6}{0.87 * 347.83 * 0.57} = 912.11 \text{ mm}^2$$

$$A_{s, min} = \frac{0.26 * f_{ctm} * b d}{f_{yk}} = \frac{0.26 * 2.56 * 3000 * 600}{400} = 2995.2 \text{ mm}^2$$

$$A_{s, max} = 0.04 A_c = 0.04 * 3000 * 600 = 72000$$

Since $A_s < A_{s, min}$ use $A_{s, min} = 2995.2 \text{ mm}^2$

Number of bar = $2995.2 / (3.14 * 24^2 / 4) = 7 \text{ } \Phi 24$

Main reinforcement- transverse bar-transverse bar

$$K = \frac{M_{yy}}{f_{cb}bd^2} \leq K_{bal}$$

$$K = \frac{153.54}{14.17 \cdot 3 \cdot 600 \cdot 600} = 0.0000221 \leq 0.167$$

Compression reinforcement is not required

Obtain lever arm Z

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53K}] \leq 0.95d = 0.82d \leq 0.95d$$

$$Z = \frac{d}{2} [1 + \sqrt{1 - 3.53K}] \leq 0.95d = 0.82d \leq 0.95d$$

$$= 0.82 \cdot 0.60 \leq 0.95 \cdot 0.60$$

$$= 0.57$$

$$A_{s, req} = \frac{(M_{yy})}{0.87 f_{yd} Z} = \frac{153.54 \cdot 10^6}{0.87 \cdot 347.83 \cdot 0.57} = 569.04 \text{ mm}^2$$

$$A_{s, min} = \frac{0.26 \cdot f_{ctm} \cdot bd}{f_{yk}} = \frac{0.26 \cdot 2.56 \cdot 3000 \cdot 600}{400} = 2995.2 \text{ mm}^2$$

$$A_{s, max} = 0.04 A_c = 0.04 \cdot 2000 \cdot 600 = 72000$$

Since $A_s < A_{s, min}$ use $A_{s, min} = 2995.2 \text{ mm}^2$

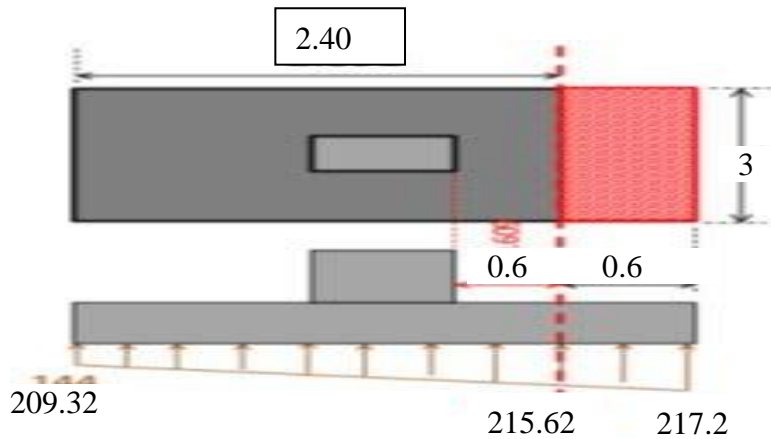
Number of bar = $2995.2 / (3.14 \cdot 24^2)$

Use 7Ø24 for transverse bar

- i. Vertical shear (wide beam shear)

Critical shear at 1.0d from face of column

$$A_{s, req} = 209.32 + \left(\frac{2.4}{3}\right) * 7.88 = 215.62$$



Design shear force, $V_{Ed} = 209.32 * 0.6 * 3 = 388.12 \text{ kN}$

Note bar extend beyond critical section at $= 600 - 40 = 560 > (1bd + d) = 36\phi + d$ use $\phi 24$

$$= 36 * 24 + 600 = 1464 \text{ mm}, A_{sl} = 0 \text{ mm}^2$$

$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{548}} = 1.604 < 2$$

$$\rho_l = \frac{A_{sl}}{bd} = 0$$

$$V_{rdc} = 0$$

$$V_{min} = \left[0.035 * K^{\frac{3}{2}} * \sqrt{f_{ck}} \right] bd$$

$$V_{min} = \left[0.035 * 1.604^{\frac{3}{2}} * \sqrt{25} \right] 3000 * 600 = 639.91$$

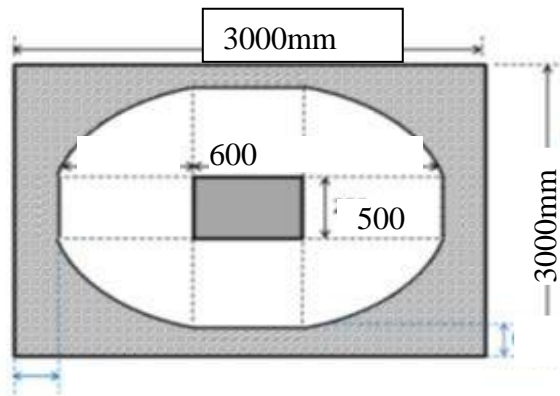
$V_{ED}(286.22) < V_{min}(639.91 \text{ kN}) \dots \dots \dots \text{OK}$

i. Punching shear

Critical shear at $2d$ from face of column

$$\text{Average } d = \frac{D + dy}{2} = \frac{600 + 524}{2} = 562$$

$$2d = 2 * 562 = 1124 \text{mm}$$



Control perimeter

$$U_1 = 2(600 + 500) + (2\pi * 1124) = 9258.72 \text{mm}$$

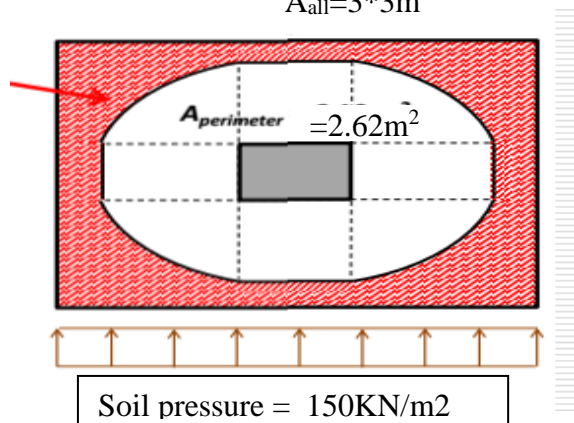
Area within perimeter

$$A = 0.6 * 0.5 + 2(0.6 * 1.124) + (2 * 0.6 * 0.1.124) + (\pi * 0.1.124^2)$$

$$= 6.96 \text{m}^2$$

Average punching shear force at control perimeter

$$A_{\text{all}} = 3 * 3 \text{m}$$



$$V_{Ed} = 150((3 \cdot 3) - 6.96) = 306 \text{ kN}$$

Punching shear stress

$$v_{ED} = \frac{\beta V_{Ed}}{u d}$$

Where

$$\beta = 1 + K \left(\frac{M_{ED}}{V_{Ed} d} \right) + \left(\frac{u_1}{w_1} \right)$$

$$W_1 = 0.5 C_1^2 + C_1 C_2 + 4 C_2 d = 16 d^2 + 2 \Pi d C_1$$

$$= 0.5 \cdot 600^2 + 600 \cdot 500 + 4 \cdot 500 \cdot 562 = 16 \cdot 562^2 + 2 \Pi \cdot 562 \cdot 600$$

$$= 9.1 \cdot 10^6 \text{ mm}^2$$

$$\beta = 1 + 0.6 \left(\frac{11.01 \cdot 10^6}{286.22} \right) + \left(\frac{9258.72}{9.1 \cdot 10^6} \right) = 1.23$$

$$v_{ED} = \frac{1.23 \cdot 388.12 \cdot 1000}{9258.7 \cdot 600} = 0.063 \text{ N/mm}^2$$

Punching Shear resistance

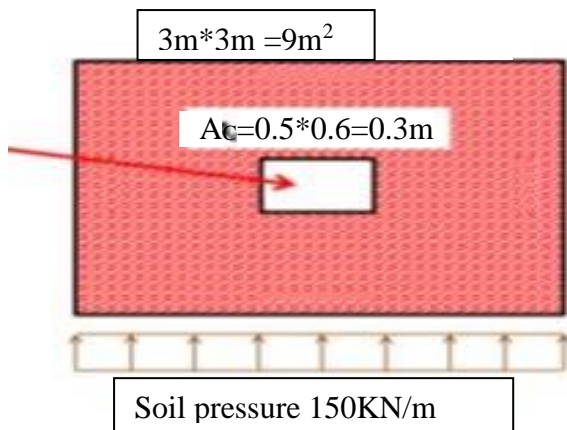
$$K = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{600}} = 1.577 < 2 \dots\dots\dots \text{ok}$$

$$V_{min} = \left[0.035 \cdot K^{\frac{3}{2}} \cdot \sqrt{f_{ck}} \right] b d$$

$$V_{min} = \left[0.035 \cdot 1.577^{\frac{3}{2}} \cdot \sqrt{25} \right] = 0.347 \text{ Nmm}^2 > V_{ED} (0.063)$$

ii. Maximum punching shear at column perimeter

$$V_{Ed,} = 1919.37 \text{ kN}$$



Column perimeter (V_0)

$$V_0 = 2(500 + 600) = 2200 \text{ mm}$$

Punching shear stress

Punching shear stress

$$v_{ED} = \frac{\beta V_{Ed}}{u d}$$

Where

$$\beta = 1 + K \left(\frac{M_{ED}}{V_{Ed}} \right) + \left(\frac{u_1}{w_1} \right)$$

$$W_1 = 0.5 C_1^2 + C_1 C_2 + 4 C_2 d = 16 d^2 + 2 \Pi d C_1$$

$$= 0.5 * 600^2 + 600 * 500 + 4 * 500 * 562 = 16 * 562^2 + 2 \Pi * 562 * 600$$

$$= 9.1 * 10^6 \text{ mm}^2$$

$$\beta = 1 + 0.6 \left(\frac{11.01 * 10^6}{1919.37} \right) + \left(\frac{9258.72}{9.1 * 10^6} \right) = 1.0$$

$$vED = \frac{1.0 \cdot 1919.37 \cdot 1000}{9258.7 \cdot 600} = 0.3455 \text{ N/mm}^2$$

Maximum Shear resistance

$$V_{min} = 0.5 \left[0.6 * \left(1 - \frac{f_{ck}}{500} \right) \right] \frac{f_{ck}}{1.5}$$

$$V_{min} = 0.5 \left[0.6 * \left(1 - \frac{25}{500} \right) \right] \frac{25}{1.5} = 4.75 > V_{ED} \text{ -----OK!}$$

ISOLATED FOOTING

Footing Label	PSC	Dimension [mm]			Bar Schedule
		L	W	D	① and ②
F1	24	3000	3000	600	

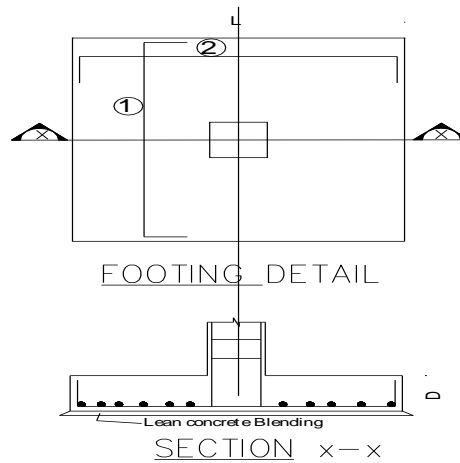


Figure 9.1 footing reinforcement layout

10 Reference

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11 Appendix

11.1 Appendix A (Architectural design review) Typical 1st- 4th floor

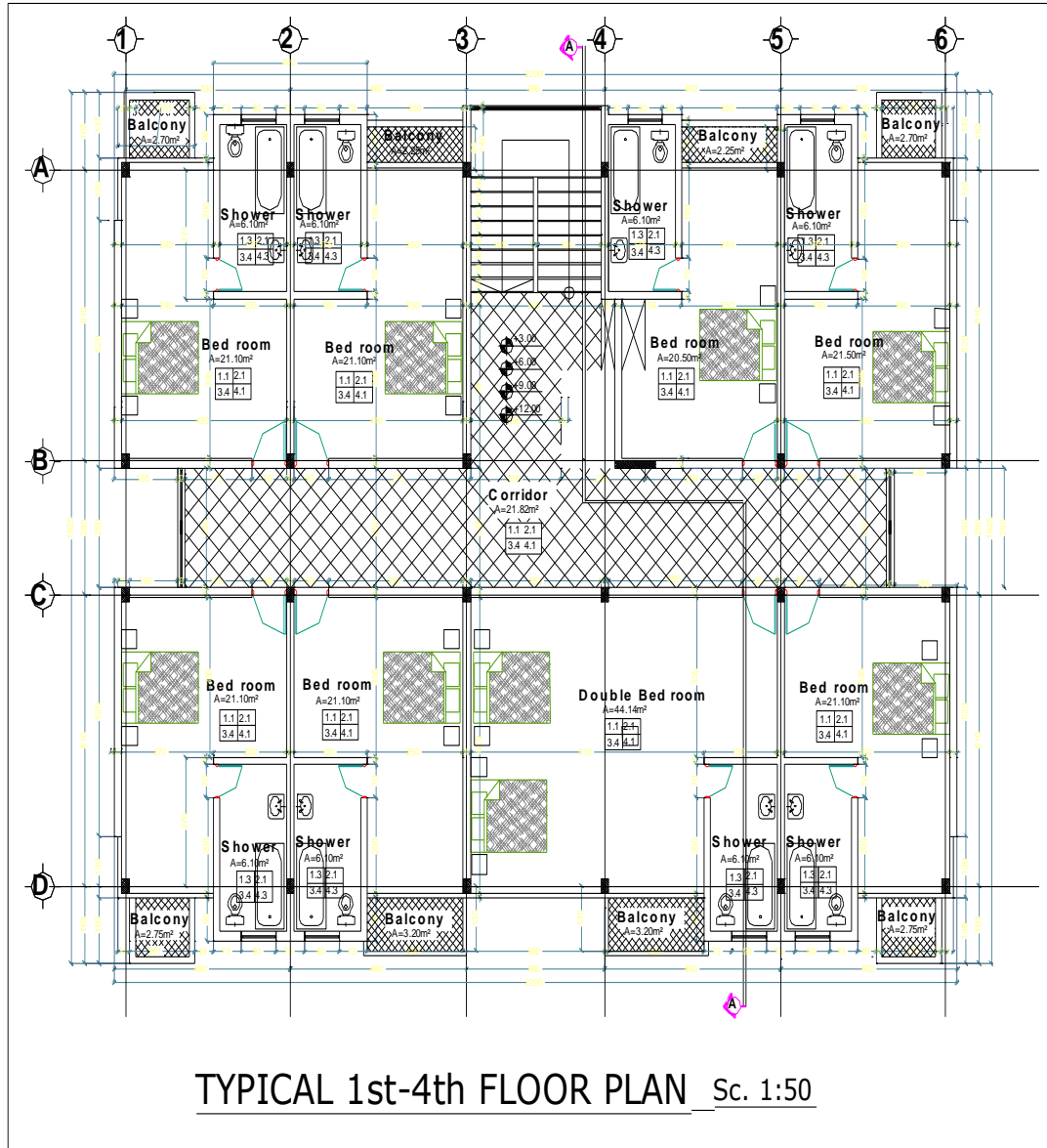


Table 11.1 minimum cover $C_{min,b}$ requirements with regard to bond

11.2 Appendix B (Acquired tables from ESEN)

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							
Criterion	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 11.2 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 11.3 Bending moment coefficient for rectangular panel

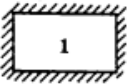
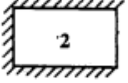
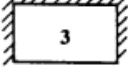
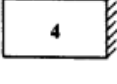
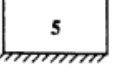
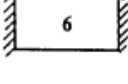
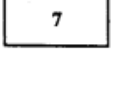
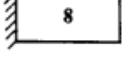
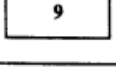
Support Condition	Coeff.	values of L_y/L_x								coefficients, α_x and α_y , for all values of L_y/L_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
	α_x	0.032	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
	α_y	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	
	α_x	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.039
	α_y	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	
	α_x	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.039
	α_y	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	
	α_x	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.047
	α_y	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	
	α_x	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	-
	α_y	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	
	α_x	-	-	-	-	-	-	-	-	0.045
	α_y	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	
	α_x	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	-
	α_y	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	
	α_x	-	-	-	-	-	-	-	-	0.058
	α_y	0.044	0.054	0.063	0.071	0.078	0.084	0.096	0.105	
	α_x	-	-	-	-	-	-	-	-	0.056
	α_y	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	

Table 10-7 factors for adjusting span moments M_x and M_y

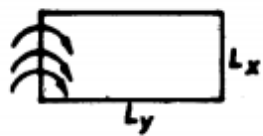
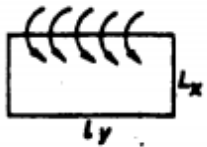
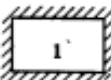
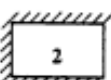
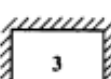
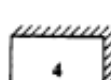
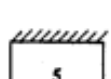
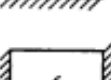
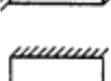
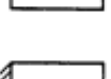
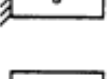
L_y/L_x				
	c_x	c_y	c_x	c_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481	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 Discontinuous	0.36 -	0.39 -	0.42 -	0.44 -	0.45 -	0.47 -	0.50 -	0.52 -	0.36 0.24
	Continuous Discontinuous	0.36 0.24	0.40 0.27	0.44 0.29	0.47 0.31	0.49 0.32	0.51 0.34	0.55 0.36	0.59 0.38	0.36 -
	Continuous Discontinuous	0.40 0.26	0.44 0.29	0.47 0.31	0.50 0.33	0.52 0.34	0.54 0.35	0.57 0.38	0.60 0.40	0.40 0.26
	Continuous Discontinuous	0.40 -	0.43 -	0.45 -	0.47 -	0.48 -	0.49 -	0.52 -	0.54 -	- 0.26
	Continuous Discontinuous	- 0.26	- 0.30	- 0.33	- 0.36	- 0.38	- 0.40	- 0.44	- 0.47	0.40 -
	Continuous Discontinuous	0.45 0.30	0.48 0.32	0.51 0.34	0.53 0.35	0.55 0.36	0.57 0.37	0.60 0.39	0.63 0.41	- 0.30
	Continuous Discontinuous	- 0.30	- 0.33	- 0.36	- 0.38	- 0.40	- 0.42	- 0.45	- 0.48	0.45 0.30
	Discontinuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

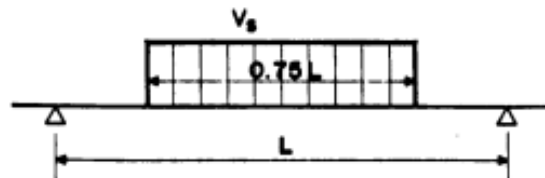


Table 10-9 α_1 α_2 α_3 α_4 α_5 coefficients

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	-

11.3 Appendix C (Analysis and design of slab)

11.3.1 Appendix C-1, Design load calculation for Typical 1st- 4th floor

P-1 & P-11

					Material	Unit weight	Dimension	Load
								KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.076	1.057
4.7	0.15	2.81	6.1	4.3				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.010	0.232
4.7	0.02	2.81						
Partition wall Volume (l*t*h)				1.98				
Partition wall plastering Volume (l*t*h)				0.2641				
Area of slab (l*w)				26.23				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					7.959	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					13.745	KN/m ²		

P-2 & P-12

Panel -2 and Panel 12					Material	Unit weight	Dimension	Load
								KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.076	1.057
4.7	0.15	2.81	6.1	4.3				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.010	0.232
4.7	0.02	2.81						
Partition wall Volume (l*t*h)				1.98105				
Partition wall plastering Volume (l*t*h)				0.26414				
` Area of slab (l*w)				26.23				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					7.959	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					13.745	KN/m ²		

P-3

Panel -3					Material	Unit weight	Dimension	Load
								KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.000	0.000
0	0.15	2.81	3.6	3.37				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.000	0.000
0	0.02	2.81						
Partition wall Volume (l*t*h)				0				
Partition wall plastering Volume (l*t*h)				0				
` Area of slab (l*w)				12.132				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					6.670	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					12.005	KN/m ²		

P-4& P-14

Panel -4 and Panel 14					Material	Unit weight	Dimension	Load KN/m ²
Slab								
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab		HCB	14	0.071	0.988
l	t	h	l	w				
4.7	0.15	2.81	6.1	4.6				
Dimension of hcb wall plastering					Concrete	23	0.009	0.217
l	t	h						
4.7	0.02	2.81						
Partition wall Volume (l*t*h)				1.98105				
Partition wall plastering Volume (l*t*h)				0.26414				
Area of slab (l*w)				28.06				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					7.875	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					13.631	KN/m ²		

P-5& P-15

Panel -5 and Panel 15					Material	Unit weight	Dimension	Load KN/m ²
Slab								
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab		HCB	14	0.076	1.059
l	t	h	l	w				
4.7	0.15	2.815	6.1	4.3				
Dimension of hcb wall plastering					Concrete	23	0.010	0.232
l	t	h						
4.7	0.02	2.815						
Partition wall Volume (l*t*h)				1.984575				
Partition wall plastering Volume (l*t*h)				0.26461				
Area of slab (l*w)				26.23				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					7.961	KN/m ²		

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DESIGN LOAD Pd = 1.35 Gk + 1.5 Qk	13.748	KN/m ²	
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P-6& P-10

Panel -6 and Panel -10					Material	Unit weight	Dimension	Load KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.191	2.667
2.8	0.2	2.81	2.95	2.8				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.019	0.438
2.8	0.02	2.81						
Partition wall Volume (l*t*h)				1.5736				
Partition wall plastering Volume (l*t*h)				0.15736				
Area of slab (l*w)				8.26				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					9.775	KN/m ²		
DESIGN LOAD Pd = 1.35 Gk + 1.5 Qk					16.197	KN/m ²		

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P- 7, 8 & 9

Panel 7 , Panel 8 and Panel 9					Material	Unit weight	Dimension	Load KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.000	0.000
0	0.15	2.81	6.1	4.6				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.000	0.000
0	0.02	2.81						
Partition wall Volume (l*t*h)				0				
Partition wall plastering Volume (l*t*h)				0				
` Area of slab (l*w)				28.06				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					6.670	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					12.005	KN/m ²		

P-13

					Material	Unit weight	Dimension	Load KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.000	0.000
0	0.15	2.815	6.1	4.3				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.000	0.000
0	0.02	2.815						
Partition wall Volume (l*t*h)				0				
Partition wall plastering Volume (l*t*h)				0				
` Area of slab (l*w)				26.23				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					6.670	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					12.005	KN/m ²		

Cantilever-1

					Material	Unit	Dimension	Load
						weight		KN/m ²
Slab					Reinforced concrete	25	0.19	4.75
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.299	4.183
4.84	0.2	1	2	1.62				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.030	0.687
4.84	0.02	1						
Partition wall Volume (l*t*h)				0.968				
Partition wall plastering Volume (l*t*h)				0.0968				
Area of slab (l*w)				3.24				
LIVE LOAD Q _k					4	KN/m ²		
DEAD LOAD G _k					11.540	KN/m ²		
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					<u>21.579</u>	KN/m ²		

Cantilever-2

					Material	Unit weight	Dimension	Load	
								KN/m ²	
Slab					Reinforced concrete	25	0.19	4.75	
cement screed					concrete	23	0.03	0.69	
ceiling plaster					concrete	23	0.03	0.69	
Floor finishing					Marble tiling	27	0.02	0.54	
								6.67	
Dimension of hcb wall			Dimension of slab						
l	t	h	l	w	HCB	14	0.721	10.092	
5.9	0.2	2.81	4	1.15					
Dimension of hcb wall plastering									
l	t	h			Concrete	23	0.072	1.658	
5.9	0.02	2.81							
Partition wall Volume (l*t*h)				3.3158					
Partition wall plastering Volume (l*t*h)				0.33158					
Area of slab (l*w)				4.6					
LIVE LOAD Q _k					2	KN/m ²			
DEAD LOAD G _k					18.419	KN/m ²			
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					27.866	KN/m ²			

Cantilever-3(BALCONY)

					Material	Unit weight	Dimension	Load	
								KN/m ²	
Slab					Reinforced concrete	25	0.19	4.75	
cement screed					concrete	23	0.03	0.69	
ceiling plaster					concrete	23	0.03	0.69	
Floor finishing					Marble tiling	27	0.02	0.54	
								6.67	
Dimension of hcb wall			Dimension of slab						
l	t	h	l	w	HCB	14	0.174	2.435	
2.5	0.2	1	2.5	1.15					
Dimension of hcb wall plastering									
l	t	h			Concrete	23	0.017	0.400	
2.5	0.02	1							
Partition wall Volume (l*t*h)				0.5					
Partition wall plastering Volume (l*t*h)				0.05					
Area of slab (l*w)				2.875					
LIVE LOAD Q _k					4	KN/m ²			
DEAD LOAD G _k					9.505	KN/m ²			
DESIGN LOAD Pd = 1.35 G _k + 1.5 Q _k					18.831	KN/m ²			

Cantilever-4

					Material	Unit weight	Dimension	Load KN/m ²
Slab								
cement screed					concrete	23	0.03	0.69
ceiling plaster					concrete	23	0.03	0.69
Floor finishing					Marble tiling	27	0.02	0.54
								6.67
Dimension of hcb wall			Dimension of slab					
l	t	h	l	w	HCB	14	0.762	10.673
3.9	0.2	2.81	2.5	1.15				
Dimension of hcb wall plastering								
l	t	h			Concrete	23	0.027	0.624
3.9	0.02	1						
Partition wall Volume (l*t*h)				2.1918				
Partition wall plastering Volume (l*t*h)				0.078				
Area of slab (l*w)				2.875				
LIVE LOAD Q _k					2	KN/m ²		
DEAD LOAD G _k					17.967	KN/m ²		
DESIGN LOAD P _d = 1.35 G _k + 1.5 Q _k					27.256	KN/m ²		

11.3.2 Appendix C-2 analysis of individual panel moment for typical 1st-4th floor

Panel	Support condition	Design load (Pd)	Effective length $\ell_x(m)$	Span ratio (ℓ_y/ℓ_x)	Moment coefficient			
					Location	Value	Location	Value
P1	TYPE 3	13.745	4.3	1.419	α_{xs}	0.0689	M_{xs}	17.52
					α_{xf}	0.0517	M_{xf}	13.15
					α_{ys}	0.039	M_{ys}	9.91
					α_{yf}	0.03	M_{yf}	7.63
P2	TYPE 1	13.748	4.6	1.326	α_{xs}	0.047	M_{xs}	13.69
					α_{xf}	0.0355	M_{xf}	10.33
					α_{ys}	0.032	M_{ys}	9.31
					α_{yf}	0.024	M_{yf}	6.98
P3	TYPE 3	12.005	3.37	1.068	α_{xs}	0.0458	M_{xs}	6.248
					α_{xf}	0.0341	M_{xf}	4.648
					α_{ys}	0.039	M_{ys}	5.317
					α_{yf}	0.03	M_{yf}	4.09
P4	TYPE 1	13.674	4.6	1.326	α_{xs}	0.047	M_{xs}	13.61
					α_{xf}	0.0355	M_{xf}	10.28
					α_{ys}	0.032	M_{ys}	9.26
					α_{yf}	0.024	M_{yf}	6.94
P5	TYPE 3	13.748	4.3	1.419	α_{xs}	0.069	M_{xs}	17.52
					α_{xf}	0.052	M_{xf}	13.15
					α_{ys}	0.039	M_{ys}	9.91
					α_{yf}	0.03	M_{yf}	7.63
P6	TYPE 2	16.197	2.8	1.05	α_{xs}	0.254	M_{xs}	32.23
					α_{xf}	0.031	M_{xf}	3.95
					α_{ys}	0.039	M_{ys}	4.95
					α_{yf}	0.029	M_{yf}	3.68
P7	TYPE 1	12.005	2.8	1.643	α_{xs}	0.056	M_{xs}	5.31
					α_{xf}	0.042	M_{xf}	3.98
					α_{ys}	0.032	M_{ys}	3.01
					α_{yf}	0.024	M_{yf}	2.26
P8	TYPE 1	12.005	2.8	1.286	α_{xs}	0.045	M_{xs}	4.28
					α_{xf}	0.035	M_{xf}	3.25
					α_{ys}	0.032	M_{ys}	3.01
					α_{yf}	0.024	M_{yf}	2.26
P9	TYPE 1	12.005	2.8	1.643	α_{xs}	0.056	M_{xs}	5.31
					α_{xf}	0.042	M_{xf}	3.98
					α_{ys}	0.032	M_{ys}	3.01
					α_{yf}	0.024	M_{yf}	2.26

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P10	TYPE 2	16.197	2.8	1.05	α_{xs}	0.254	M_{xs}	32.23
					α_{xf}	0.031	M_{xf}	3.95
					α_{ys}	0.039	M_{ys}	4.95
					α_{yf}	0.029	M_{yf}	3.68
P11	TYPE 3	13.745	4.3	1.419	α_{xs}	0.0689	M_{xs}	17.52
					α_{xf}	0.0517	M_{xf}	13.15
					α_{ys}	0.039	M_{ys}	9.91
					α_{yf}	0.03	M_{yf}	7.63
P12	TYPE 1	13.748	4.6	1.326	α_{xs}	0.047	M_{xs}	13.69
					α_{xf}	0.0355	M_{xf}	10.33
					α_{ys}	0.032	M_{ys}	9.31
					α_{yf}	0.024	M_{yf}	6.98
P13	TYPE 2	12.005	3.6	1.694	α_{xs}	0.062	M_{xs}	9.63
					α_{xf}	0.046	M_{xf}	7.17
					α_{ys}	0.039	M_{ys}	6.07
					α_{yf}	0.029	M_{yf}	4.51
P14	TYPE 1	13.674	4.6	1.326	α_{xs}	0.047	M_{xs}	13.61
					α_{xf}	0.0355	M_{xf}	10.28
					α_{ys}	0.032	M_{ys}	9.26
					α_{yf}	0.024	M_{yf}	6.94
P15	TYPE 3	13.748	4.3	1.419	α_{xs}	0.069	M_{xs}	17.52
					α_{xf}	0.052	M_{xf}	13.15
					α_{ys}	0.039	M_{ys}	9.91
					α_{yf}	0.03	M_{yf}	7.63

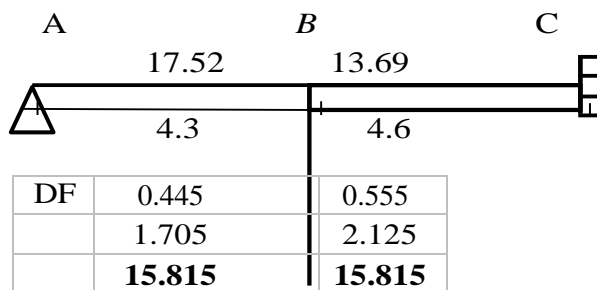
11.3.3 Appendix C-3 support moment adjustment for typical 1-4th floor

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

$$\Delta M_s = 17.52 - 13.69 = 3.83$$

$$20\% M_{large} = 0.2 * 17.52 = 3.50$$

Moment distribution method required



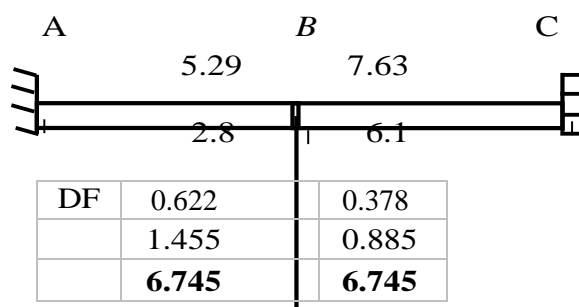
$$K_{AB} = \frac{I}{L}, \frac{3I}{4*4.3} = 0.174 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.23256}{0.23256 + 0.2174} = 0.445 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.555$$

Panel P-6 and P-11 –on axis C adjustment is required

$$\Delta M_s = 7.63 - 5.29 = 2.34$$

$$20\% M_{large} = 0.2 * 7.63 = 1.526$$



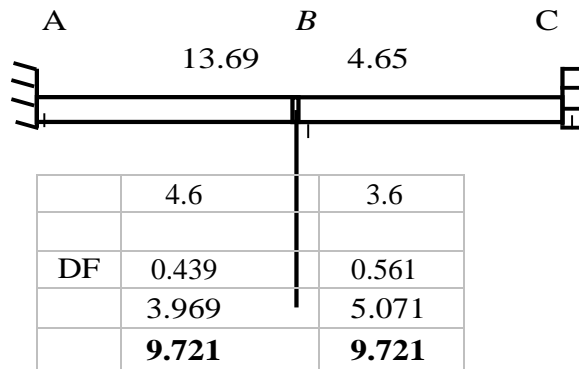
$$K_{AB} = \frac{I}{L}, \frac{I}{2.8} = 0.357 \quad K_{BC} = \frac{I}{L}, \frac{I}{6.1} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.445 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.555$$

Panel P-2 and P-3 –on axis 3 adjustment is required

$$\Delta M_s = 13.69 - 5.32 = 8.37$$

$$20\% M_{large} = 0.2 * 13.69 = 2.74$$



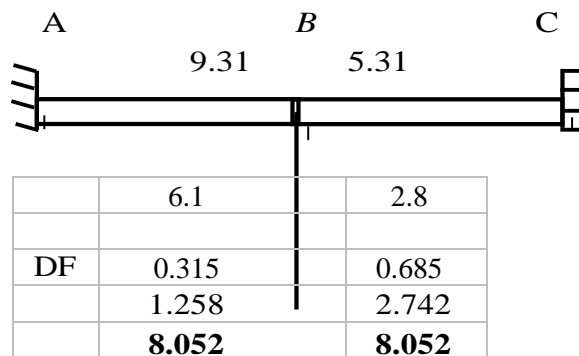
$$K_{AB} = \frac{I}{L}, \frac{I}{4.6} = 0.217 \quad K_{BC} = \frac{I}{L}, \frac{I}{3.6} = 0.278$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.439 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.561$$

Panel P-2 and P-7 –on axis B adjustment is required

$$\Delta M_s = 9.31 - 5.31 = 4$$

$$20\% M_{large} = 0.2 * 9.31 = 1.86$$



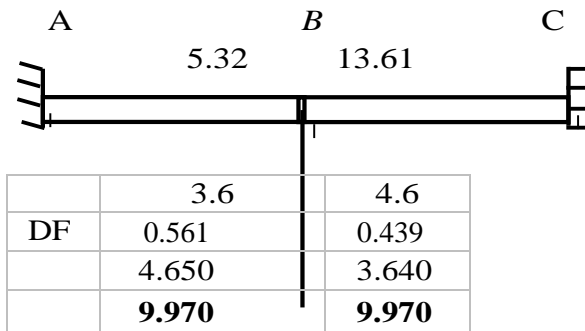
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.174}{0.174 + 0.2174} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-3 and P-4—on axis 4 adjustments is required

$$\Delta M_s = 13.61 - 5.32 = 8.29$$

$$20\% M_{large} = 0.2 * 13.61 = 2.72$$



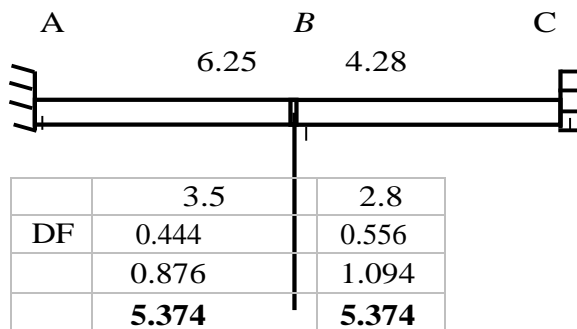
$$K_{AB} = \frac{I}{L}, \frac{I}{3.6} = 0.278 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.278}{0.278 + 0.2174} = 0.561 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.439$$

Panel P-3 and P-8—on axis B adjustment is required

$$\Delta M_s = 6.25 - 4.28 = 1.97$$

$$20\% M_{large} = 0.2 * 6.25 = 1.25$$



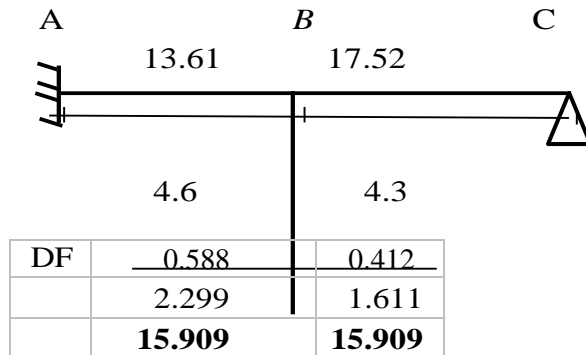
$$K_{AB} = \frac{I}{L}, \frac{I}{3.5} = 0.286 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.444 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.556$$

Panel P-4 and P-5—on axis 5 adjustment is required

$$\Delta M_s = 17.52 - 13.61 = 3.91$$

$$20\% M_{large} = 0.2 * 17.52 = 3.50$$



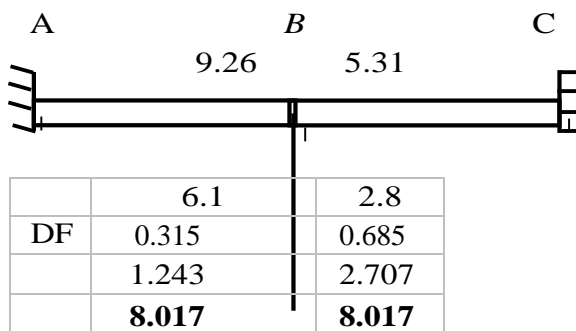
$$K_{AB} = \frac{I}{L}, \frac{I}{4.6} = 0.233 \quad K_{BC} = \frac{I}{L}, \frac{3I}{4L} = 0.163$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.588 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.412$$

Panel P-4 and P-9—on axis B adjustment is required

$$\Delta M_s = 9.26 - 5.31 = 3.95$$

$$20\% M_{large} = 0.2 * 9.26 = 1.85$$



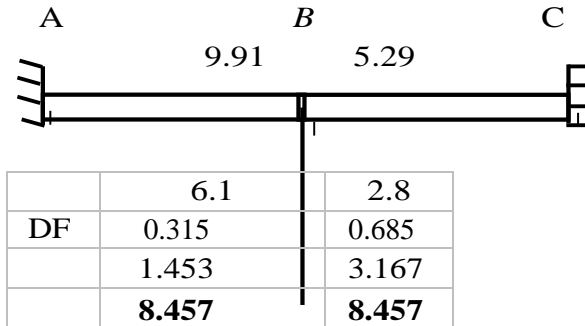
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-5 and P-10 – on axis B adjustment is required

$$\Delta M_s = 9.91 - 5.29 = 4.62$$

$$20\% M_{large} = 0.2 * 9.91 = 1.98$$



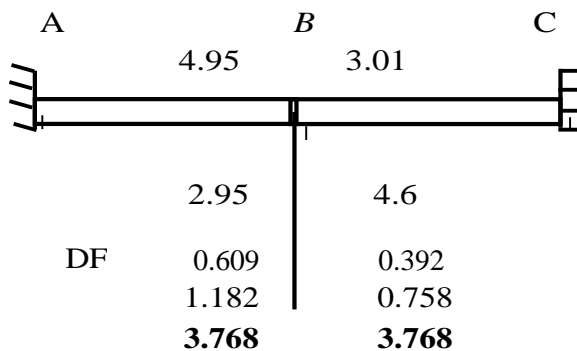
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-6 and P-7 – on axis 2 adjustment is required

$$\Delta M_s = 4.95 - 3.01 = 1.94$$

$$20\% M_{large} = 0.2 * 4.95 = 0.99$$



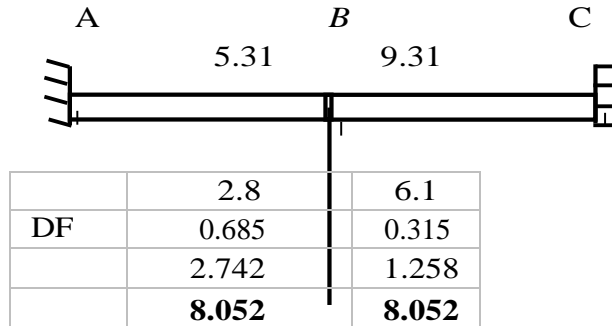
$$K_{AB} = \frac{I}{L}, \frac{I}{2.95} = 0.39 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.609 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.392$$

Panel P-7 and P-12 –on axis C adjustment is required

$$\Delta M_s = 9.31 - 5.31 = 4$$

$$20\% M_{large} = 0.2 * 4.95 = 1.862$$



$$K_{AB} = \frac{I}{L}, \frac{I}{2.8} = 0.357 \quad K_{BC} = \frac{I}{L}, \frac{I}{6.1} = 0.164$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.685 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.315$$

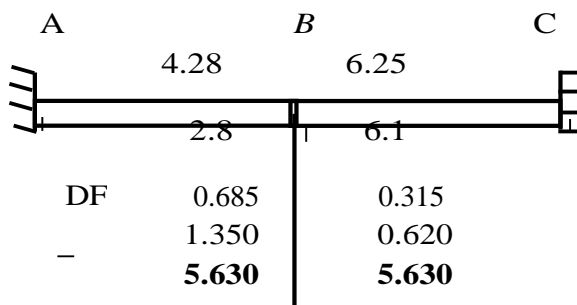
Panel P-7 and P-8 –on axis 3 does not required moment adjustment since they have equal magnitude of moment

Panel P-8 and P-9 –on axis 4 also does not required moment adjustment since they have equal magnitude of moment

Panel P-8 and P-13 –on axis 3 adjustment is required

$$\Delta M_s = 6.25 - 4.28 = 1.97$$

$$20\% M_{large} = 0.2 * 6.25 = 1.25$$



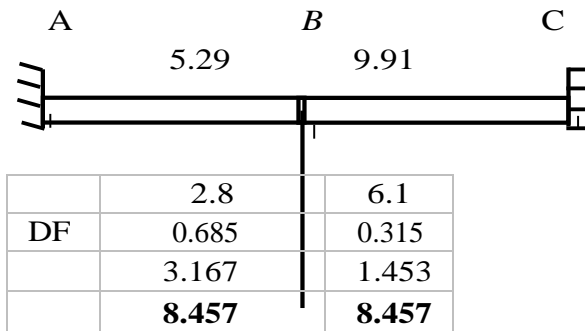
$$K_{AB} = \frac{I}{L}, \frac{I}{2.8} = 0.357 \quad K_{BC} = \frac{I}{L}, \frac{I}{6.1} = 0.164$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.685 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.315$$

Panel P-9 and P-14 –on axis 3 adjustment is required

$$\Delta M_s = 9.91 - 5.29 = 4.62$$

$$20\% M_{large} = 0.2 * 9.26 = 1.982$$



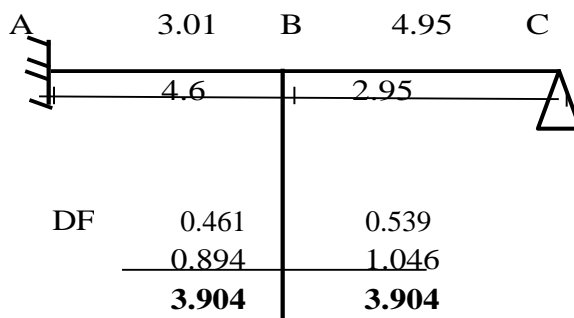
$$K_{AB} = \frac{I}{L}, \frac{I}{2.8} = 0.357 \quad K_{BC} = \frac{I}{L}, \frac{I}{6.1} = 0.164$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.685 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.315$$

Panel P-9 and P-10 –on axis 5 adjustment is required

$$\Delta M_s = 4.95 - 3.01 = 1.94$$

$$20\% M_{large} = 0.2 * 4.95 = 0$$



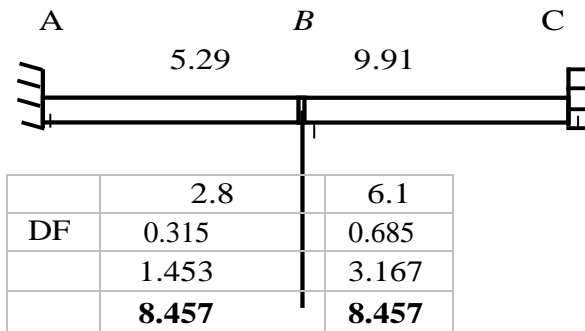
$$K_{AB} = \frac{I}{L}, \frac{I}{4.6} = 0.217 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.95} = 0.254$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.462 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.539$$

Panel P-10 and P-15 – on axis B adjustment is required

$$\Delta M_s = 9.91 - 5.29 = 4.62$$

$$20\% M_{large} = 0.2 * 9.91 = 1.98$$



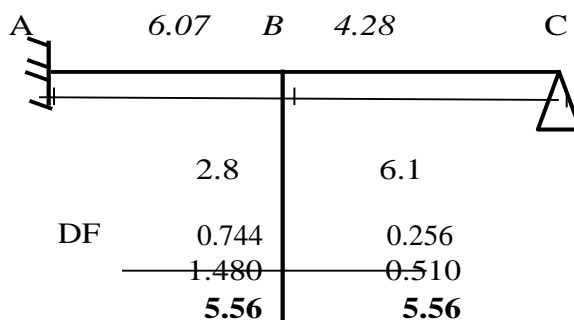
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-8 and P-13 – on axis C adjustment is required

$$\Delta M_s = 6.07 - 4.28 = 1.79$$

$$20\% M_{large} = 0.2 * 6.07 = 1.21$$



$$K_{AB} = \frac{I}{L}, \frac{I}{2.8} = 0.357 \quad K_{BC} = \frac{I}{L}, \frac{3}{4} * \frac{I}{6.1} = 0.123$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.744 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.256$$

Panel P-13 and P-14 –on axis 4 adjustment is required

$$\Delta M_s = 13.61 - 9.63 = 3.98$$

$$20\% M_{large} = 0.2 * 13.61 = 2.72$$

	3.6	4.6
DF	0.561	0.439
	2.233	1.747
	11.863	11.863

$$K_{AB} = \frac{I}{L}, \frac{I}{3.6} = 0.278 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

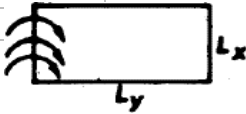
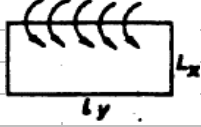
$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.561 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.439$$

No more adjustment is required between the cantilever slab and the adjacent panel since the cantilever create large amount of moment relative to the panel, we take it as its own.

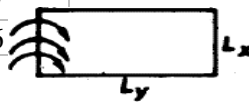
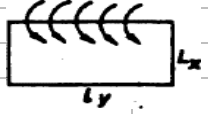
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11.3.4 Appendix C-4 field moment adjustment for typical 1-4 floor

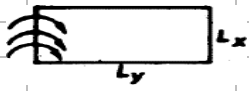
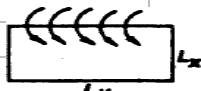
P-1& P-11

Mxf	13.15	Mxs	17.52	Mxs,adje	15.85
Myf	9.91	Mys	7.63	Mys, adje	8.46
ΔM_{xs}	1.67				
ΔM_{ys}	0.74				
					
CX	0.31314	CX	0.402279		
CY	0.107023	CY	0.327093		
ΔM_{xf}	0.903529				
Mxf,adje	14.05				
ΔM_{yf}	0.625443				
Myf,adje	10.54				

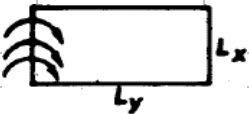
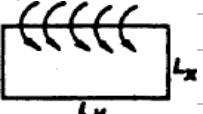
P- 2

Mxf	10.33	Mxs	13.69	Mxs,adje	10.02
Myf	6.98	Mys	9.31	Mys, adje	8.05
ΔM_{xs}	3.67				
ΔM_{ys}	1.26				
					
CX	0.322391	CX	0.379522		
CY	0.128478	CY	0.345043		
ΔM_{xf}	1.799058				
Mxf,adje	12.13				
ΔM_{yf}	1.428192				
Myf,adje	8.41				

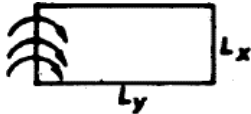
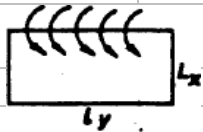
P-3

Mxf	4.65	Mxs	6.25	Mxs,adje	5.37
Myf	4.09	Mys	5.32	Mys, adje	10.02
ΔM_{xs}	0.88				
ΔM_{ys}	-4.7				
					
	CX	0.398545			
	CY	0.341818		CX	0.253727
				CY	0.384636
ΔM_{xf}		-1.87316			
Mxf,adje		2.78		ΔM_{xf}	0.22328
				Mxf,adje	4.87
ΔM_{yf}		-1.60655			
Myf,adje		2.48		ΔM_{yf}	0.33848
CX		0.398545		Myf,adje	4.43

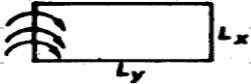
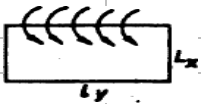
P-4

Mxf	10.28	Mxs	13.61	Mxs,adje	9.97
Myf	6.94	Mys	9.26	Mys, adje	8.02
ΔM_{xs}	3.64				
ΔM_{ys}	1.24				
					
		0.322391		CX	0.379522
	CY	0.128478		CY	0.345043
ΔM_{xf}		1.781224		ΔM_{xf}	1.381459
Mxf,adje		12.06		Mxf,adje	11.661
ΔM_{yf}		1.415271		ΔM_{yf}	0.427854

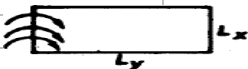
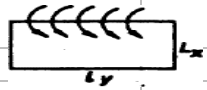
P-5&P15

Mxf	13.15	Mxs	17.52	Mxs,adje	15.91
Myf	7.63	Mys	9.91	Mys, adje	8.46
ΔM_{xs}	1.61				
ΔM_{ys}	1.45				
					
					
	CX	0.31314		CX	0.402279
	CY	0.107023		CY	0.327093
	ΔM_{xf}	1.101722			
	Mxf,adje	14.25			
	ΔM_{yf}	0.681803			
	Myf,adje	8.31			

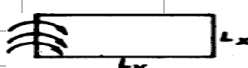
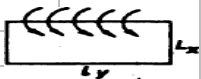
P-6&P-10

Mxf	3.95	Mxs	5.29	Mxs,adje	6.89		
Myf	3.68	Mys	4.95	Mys, adje	3.77		
ΔM_{xs}	-1.6						
ΔM_{ys}	1.18						
							
					CX	0.298214	
	CX	0.367143			CY	0.376786	
	CY	0.247857					
					ΔM_{xf}	0.766071	
	ΔM_{xf}	0.433229			Mxf,adje	4.72	
	Mxf,adje	4.38					
					ΔM_{yf}	-0.60286	
	ΔM_{yf}	0.292471			Myf,adje	3.927857	
	Myf,adje	3.97					

P-12

Mxf	10.33	Mxs	13.69	Mxs,adje	11.88
Myf	6.98	Mys	9.31	Mys, adje	8.05
ΔM_{xs}	1.81				
ΔM_{ys}	1.26				
					
	CX	0.322391		CX	0.379522
	CY	0.128478		CY	0.345043
	ΔM_{xf}	1.093147		ΔM_{xf}	0.686934
	Mxf,adje	11.42		Mxf,adje	11.02

P-6&P10

Mxf	10.28	Mxs	13.61	Mxs,adje	11.86
Myf	6.94	Mys	9.26	Mys, adje	8.46
ΔM_{xs}	1.75				
ΔM_{ys}	0.8				
					
	CX	0.322391		CX	0.379522
	CY	0.128478		CY	0.345043
	ΔM_{xf}	0.922076		ΔM_{xf}	0.664163
	Mxf,adje	11.2		Mxf,adje	10.94
	ΔM_{yf}	0.706609		ΔM_{yf}	0.276035
	Myf,adje	7.65		Myf,adje	7.07

11.3.5 Appendix C 2 shear force calculation for typical 1st-4th floor

Panel	Support condition	Design load (Pd)	Effective length $\ell_x(m)$	Span ratio (ℓ_y/ℓ_x)	Moment coefficient		Shear force(V)	
					Location	Value	Location	Value
P1	TYPE 3	13.745	4.3	1.419	$B_{vx}(cont)$	0.493	$V_x(cont)$	29.1
					$B_{vx}(disc)$	0.34	$V_x(disc)$	20.1
					$B_{vy}(cont)$	0.36	$V_y(cont)$	21.3
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P2	TYPE 1	13.748	4.6	1.326	$B_{vx}(cont)$	0.415	$V_x(cont)$	26.2
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.33	$V_y(cont)$	20.9
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P3	TYPE 3	12.005	3.37	1.068	$B_{vx}(cont)$	0.387	$V_x(cont)$	15.7
					$B_{vx}(disc)$	0.26	$V_x(disc)$	10.5
					$B_{vy}(cont)$	0.36	$V_y(cont)$	14.6
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P4	TYPE 1	13.674	4.6	1.326	$B_{vx}(cont)$	0.41	$V_x(cont)$	25.8
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.33	$V_y(cont)$	20.8
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P5	TYPE 3	13.748	4.3	1.419	$B_{vx}(cont)$	0.494	$V_x(cont)$	29.2
					$B_{vx}(disc)$	0.324	$V_x(disc)$	19.2
					$B_{vy}(cont)$	0.36	$V_y(cont)$	21.3
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P6	TYPE 2	16.197	2.8	1.05	$B_{vx}(cont)$	0.376	$V_x(cont)$	17.1
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.36	$V_y(cont)$	16.3

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					$B_{vy}(\text{disc})$	0.24	$V_y(\text{disc})$	10.9
P7	TYPE 1	12.005	2.8	1.643	$B_{vx}(\text{cont})$	0.467	$V_x(\text{cont})$	15.7
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	11.1
						0	$V_y(\text{disc})$	0.0
P8	TYPE 1	12.005	2.8	1.286	$B_{vx}(\text{cont})$	0.407	$V_x(\text{cont})$	13.7
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	11.1
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.0
P9	TYPE 1	12.005	2.8	1.643	$B_{vx}(\text{cont})$	0.467	$V_x(\text{cont})$	15.7
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	11.1
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.0
P10	TYPE 2	16.197	2.8	1.05	$B_{vx}(\text{cont})$	0.376	$V_x(\text{cont})$	17.1
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	16.3
					$B_{vy}(\text{disc})$	0.24	$V_y(\text{disc})$	10.9
P11	TYPE 3	13.745	4.3	1.419	$B_{vx}(\text{cont})$	0.493	$V_x(\text{cont})$	29.1
					$B_{vx}(\text{disc})$	0.34	$V_x(\text{disc})$	20.1
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	21.28
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00
P12	TYPE 1	13.748	4.6	1.326	$B_{vx}(\text{cont})$	0.415	$V_x(\text{cont})$	26.24
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.00
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	20.87
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00
P13	TYPE 2	12.005	3.6	1.694	$B_{vx}(\text{cont})$	0.493	$V_x(\text{cont})$	21.31

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					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.00
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	15.56
					$B_{vy}(\text{disc})$	0.24	$V_y(\text{disc})$	10.37
P14	TYPE 1	13.674	4.6	1.326	$B_{vx}(\text{cont})$	0.41	$V_x(\text{cont})$	25.79
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.00
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	20.76
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00
P15	TYPE 3	13.748	4.3	1.419	$B_{vx}(\text{cont})$	0.494	$V_x(\text{cont})$	29.20
					$B_{vx}(\text{disc})$	0.324	$V_x(\text{disc})$	19.15
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	21.28
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00

11.3.6 Appendix D depth determination for individual panel for roof slab

Panel	ly (mm)	Lx (mm)	ly/lx	Type	structural system	k	N	lx/d	d
P-1	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.43
P-2	6100	4600	1.33	two way	continuous	1.5	17.71	33.20	138.54
P-3	3600	3370	1.07	two way	continuous	1.5	17.71	33.20	101.50
P-4	6100	4600	1.33	two way	continuous	1.5	17.71	33.20	138.54
P-5	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.43
P-6	2950	2800	1.05	two way	end span	1.3	17.71	28.78	97.30
P-7	4600	2800	1.64	two way	continuous	1.5	17.71	33.20	84.33
P-8	3600	2800	1.29	two way	continuous	1.5	17.71	33.20	84.33
P-9	4600	2800	1.64	two way	continuous	1.5	17.71	33.20	84.33
P-10	2950	2800	1.05	two way	end span	1.3	17.71	28.78	97.30
P-11	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.43
P-12	6100	4600	1.33	two way	continuous	1.5	17.71	33.20	138.54
P-13	6100	3600	1.69	two way	end span	1.3	17.71	28.78	125.10
P-14	6100	4600	1.33	two way	continuous	1.5	17.71	33.20	138.54
P15	6100	4300	1.42	two way	end span	1.3	17.71	28.78	149.43
C1	8900	1620	5.49	one way	Cantilever	0.4	17.71	8.85	182.97
C2	5500	1350	4.07	one way	Cantilever	0.4	17.71	8.85	152.47
C3	7000	1620	4.32	one way	Cantilever	0.4	17.71	8.85	182.97
C4	6300	1620	3.89	one way	Cantilever	0.4	17.71	8.85	182.97
C5	8800	1150	7.65	one way	Cantilever	0.4	17.71	8.85	129.88
Governing panel depth(mm)									182.97
clear cover(mm)									22
Assuming reinforcements ϕ									10
D(mm)=d+c+(ϕ /2)									209.96
Use D(mm)									210mm

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11.3.7 Appendix D- 2 analysis of individual panel moment for roof floor

Panel	Support condition	Design load (Pd)	Effective length ℓ_x (m)	Span ratio (ℓ_y/ℓ_x)	Moment coefficient			
					Location	Value	Location	Value
P1	TYPE 3	9.95	4.3	1.419	α_{xs}	0.0689	M_{xs}	12.7
					α_{xf}	0.0517	M_{xf}	9.5
					α_{ys}	0.039	M_{ys}	7.2
					α_{yf}	0.03	M_{yf}	5.5
P2	TYPE 1	9.95	4.6	1.326	α_{xs}	0.047	M_{xs}	9.9
					α_{xf}	0.0355	M_{xf}	7.5
					α_{ys}	0.032	M_{ys}	6.7
					α_{yf}	0.024	M_{yf}	5.1
P3	TYPE 2	9.95	3.6	1.694	α_{xs}	0.0458	M_{xs}	8.0
					α_{xf}	0.0341	M_{xf}	5.9
					α_{ys}	0.039	M_{ys}	5.0
					α_{yf}	0.03	M_{yf}	3.7
P4	TYPE 1	9.95	4.6	1.326	α_{xs}	0.047	M_{xs}	9.9
					α_{xf}	0.0355	M_{xf}	86.7
					α_{ys}	0.032	M_{ys}	10.9
					α_{yf}	0.024	M_{yf}	10.9
P5	TYPE 3	9.95	4.3	1.419	α_{xs}	0.069	M_{xs}	12.7
					α_{xf}	0.052	M_{xf}	9.6
					α_{ys}	0.039	M_{ys}	7.2
					α_{yf}	0.03	M_{yf}	5.5
P6	TYPE 2	9.95	2.8	1.05	α_{xs}	0.254	M_{xs}	3.3
					α_{xf}	0.031	M_{xf}	2.4
					α_{ys}	0.039	M_{ys}	3.0
					α_{yf}	0.029	M_{yf}	2.3
P7	TYPE 1	9.95	2.8	1.643	α_{xs}	0.056	M_{xs}	4.4
					α_{xf}	0.042	M_{xf}	3.3
					α_{ys}	0.032	M_{ys}	2.5
					α_{yf}	0.024	M_{yf}	1.9
P8	TYPE 1	9.95	2.8	1.286	α_{xs}	0.045	M_{xs}	3.5
					α_{xf}	0.035	M_{xf}	2.7
					α_{ys}	0.032	M_{ys}	2.5
					α_{yf}	0.024	M_{yf}	1.9
P9	TYPE 1	9.95	2.8	1.643	α_{xs}	0.056	M_{xs}	4.4
					α_{xf}	0.042	M_{xf}	3.3
					α_{ys}	0.032	M_{ys}	2.5
					α_{yf}	0.024	M_{yf}	1.9

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P10	TYPE 2	9.95	2.8	1.05	α_{xs}	0.254	M_{xs}	19.8
					α_{xf}	0.031	M_{xf}	2.4
					α_{ys}	0.039	M_{ys}	3.0
					α_{yf}	0.029	M_{yf}	2.3
P11	TYPE 3	9.95	4.3	1.419	α_{xs}	0.0689	M_{xs}	12.7
					α_{xf}	0.0517	M_{xf}	9.5
					α_{ys}	0.039	M_{ys}	7.18
					α_{yf}	0.03	M_{yf}	5.52
P12	TYPE 1	9.95	4.6	1.326	α_{xs}	0.047	M_{xs}	9.90
					α_{xf}	0.0355	M_{xf}	7.47
					α_{ys}	0.032	M_{ys}	6.74
					α_{yf}	0.024	M_{yf}	5.05
P13	TYPE 2	9.95	3.6	1.694	α_{xs}	0.062	M_{xs}	8.00
					α_{xf}	0.046	M_{xf}	5.93
					α_{ys}	0.039	M_{ys}	5.03
					α_{yf}	0.029	M_{yf}	3.74
P14	TYPE 1	9.95	4.6	1.326	α_{xs}	0.047	M_{xs}	9.90
					α_{xf}	0.0355	M_{xf}	7.47
					α_{ys}	0.032	M_{ys}	6.74
					α_{yf}	0.024	M_{yf}	5.05
P15	TYPE 3	9.95	4.3	1.419	α_{xs}	0.069	M_{xs}	12.69
					α_{xf}	0.052	M_{xf}	9.57
					α_{ys}	0.039	M_{ys}	7.18
					α_{yf}	0.03	M_{yf}	5.52

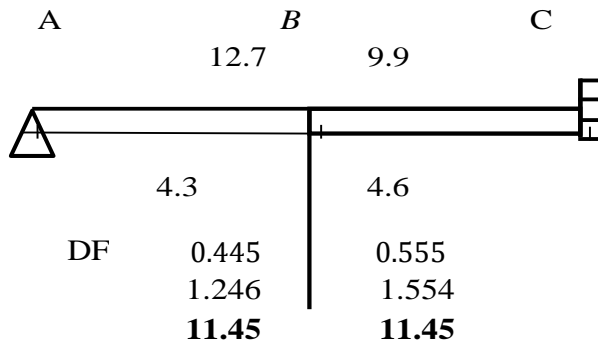
-

11.3.8 Appendix C-2 support moment adjustment for typical roof floor

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

$$\Delta M_s = 12.7 - 9.9 = 2.8$$

$$20\% M_{large} = 0.2 * 12.7 = 2.54$$



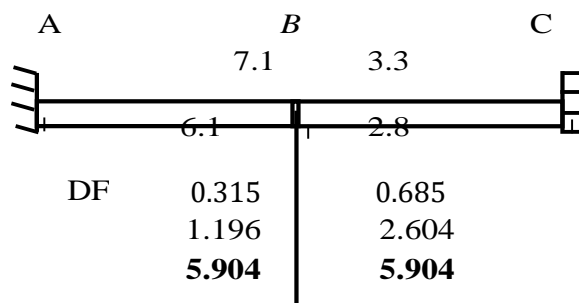
$$K_{AB} = \frac{I}{L}, \frac{3I}{4*4.3} = 0.174 \quad K_{BC} = \frac{I}{L}, \frac{I}{4.6} = 0.217$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.174}{0.174 + 0.2174} = 0.445 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.555$$

Panel P-1 and P-6 –on axis B adjustment is required

$$\Delta M_s = 7.1 - 3.3 = 3.8$$

$$20\% M_{large} = 0.2 * 7.1 = 1.42$$



$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

-

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.164}{0.164 + 0.357} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-2 and P-3 –on axis 3 adjustment is required

$$\Delta M_s = 9.9 - 8 = 1.9$$

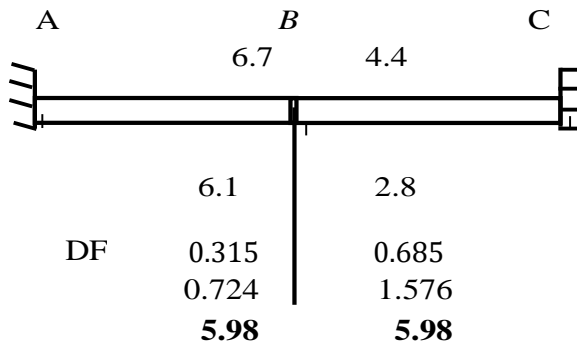
$$20\% M_{large} = 0.2 * 9.9 = 1.98 \quad \text{Use average Method}$$

$$M_{adj.} = (9.9 + 8) / 2 = \mathbf{8.95}$$

Panel P-2 and P-7 –on axis B adjustment is required

$$\Delta M_s = 6.7 - 4.4 = 2.3$$

$$20\% M_{large} = 0.2 * 6.7 = 1.34$$



$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}}, \frac{0.174}{0.174 + 0.2174} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-3 and P-4 on axis 4 adjustment is required

$$\Delta M_s = 9.9 - 8 = 1.9$$

$$20\% M_{large} = 0.2 * 9.9 = 1.98$$

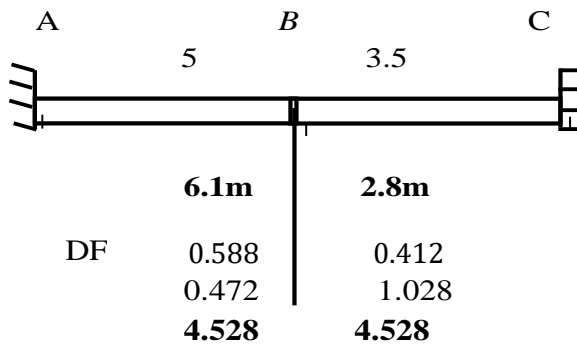
$$M_{adj.} = (9.9 + 8) / 2 = 8.95$$

–

Panel P-3 and P-8 –on axis B adjustment is required

$$\Delta M_s = 5 - 3.5 = 1.5$$

$$20\% M_{large} = 0.2 * 5 = 1$$



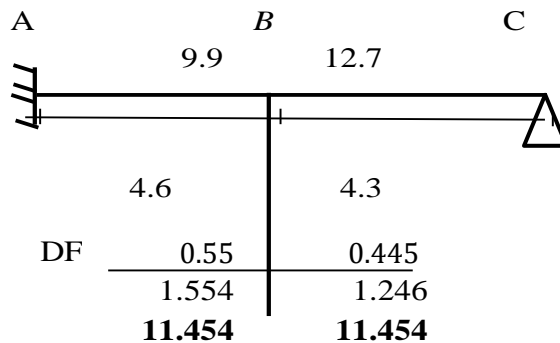
$$K_{AB} = \frac{I}{L}, \frac{I}{3.5} = 0.286 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.444 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.556$$

Panel P-4 and P-5—on axis 5 adjustments is required

$$\Delta M_s = 12.7 - 9.9 = 2.8$$

$$20\% M_{large} = 0.2 * 12.7 = 2.54$$



$$K_{AB} = \frac{I}{L}, \frac{I}{4.6} = 0.233 \quad K_{BC} = \frac{I}{L}, \frac{3I}{4L} = 0.163$$

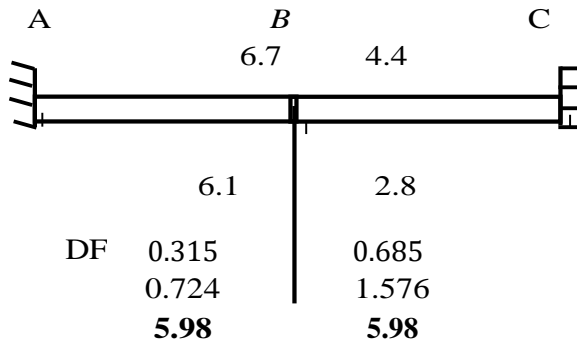
—

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.588 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.412$$

Panel P-4 and P-9—on axis B adjustment is required

$$\Delta M_s = 6.7 - 4.4 = 2.3$$

$$20\% M_{large} = 0.2 * 6.7 = 1.34$$



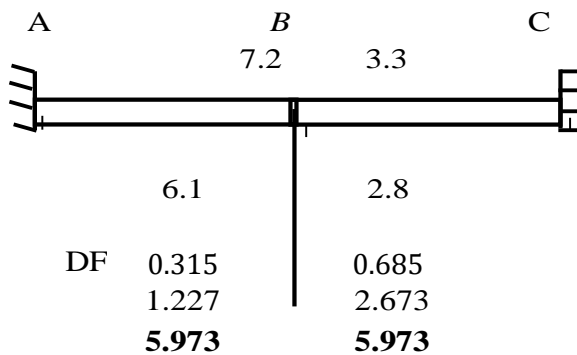
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

Panel P-5 and P-10—on axis B adjustment is required

$$\Delta M_s = 7.2 - 3.3 = 3.99$$

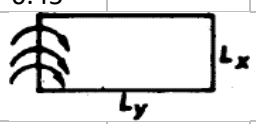
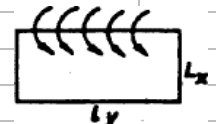
$$20\% M_{large} = 0.2 * 7.2 = 1.44$$



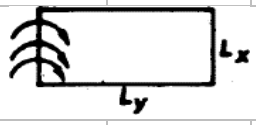
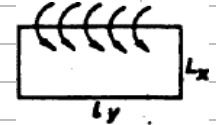
$$K_{AB} = \frac{I}{L}, \frac{I}{6.1} = 0.164 \quad K_{BC} = \frac{I}{L}, \frac{I}{2.8} = 0.357$$

$$DF = \frac{K_{AB}}{K_{AB} + K_{BC}} = 0.315 \quad DF = \frac{K_{BC}}{K_{BC} + K_{AB}} = 0.685$$

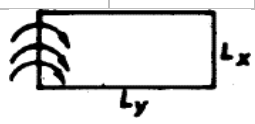
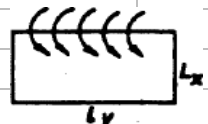
11.3.9 Appendix D-3 field moment adjustment for typical 1-4 floor
P3 & P13

Mxf	5.9	Mxs	8	Mxs,adje	5.98		
Myf	3.7	Mys	5	Mys, adje	2.5		
ΔM_{xs}	-0.95						
ΔM_{ys}	0.45						
							
	CX	0.285556				CX	0.350714
	CY	0.0745				CY	0.221714
	ΔM_{xf}	0.134211					
	Mxf,adje	6.03					
	ΔM_{yf}	0.035015					
	Myf,adje	3.74					

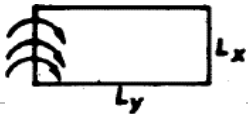
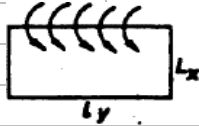
P1 & P 11

Mxf	10.42	Mxs	7.97	Mxs,adje	11.45		
Myf	5.93	Mys	7.1	Mys, adje	5.9		
ΔM_{xs}	1.52						
ΔM_{ys}	1.2						
							
	CX	0.31314				CX	0.350714
	CY	0.107023				CY	0.251714
	ΔM_{xf}	0.987234					
	Mxf,adje	10.38					
	ΔM_{yf}	0.625609					
	Myf,adje	6.01					

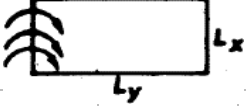
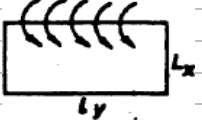
P5& P 15

Mxf	9.6	Mxs	12.7	Mxs,adje	11.45		
Myf	5.5	Mys	7.2	Mys, adje	5.9		
ΔM_{xs}	1.25						
ΔM_{ys}	1.3						
							
	CX	0.31314				CX	0.150714
	CY	0.107023				CY	0.211714
	ΔM_{xf}	0.9099					
	Mxf,adje	10.51					
	ΔM_{yf}	0.547997					
	Myf,adje	16.05					

P2, P12, P4 & P 14

Mxf	7.5	Mxs	9.9	Mxs,adje	5.98		
Myf	5.1	Mys	6.7	Mys, adje	2.5		
ΔM_{xs}	0.095						
ΔM_{ys}	0.72						
							
	CX	0.322391				CX	0.480714
	CY	0.038674				CY	0.381714
	ΔM_{xf}	0.592667					
	Mxf,adje	8.09					
	ΔM_{yf}	0.355637					
	Myf,adje	5.46					

P7 & P 9

Mxf	3.3	Mxs	4.4	Mxs,adje	5.98		
Myf	1.9	Mys	2.5	Mys, adje	2.5		
ΔM_{xs}	-1.58						
ΔM_{ys}	0						
							
	CX	0.290714				CX	0.450714
	CY	0.079143				CY	0.281714
ΔM_{xf}	0						
Mxf,adje	3.3						
ΔM_{yf}	0						
Myf,adje	1.9						

11.3.9.1.1 Appendix D-4 shear force calculation for typical roof floor

Panel	Support condition	Design load (Pd)	Effective length $\ell_x(m)$	Span ratio (ℓ_y/ℓ_x)	Moment coefficient		beta	
					Location	Value	Location	Value
P1	TYPE 3	9.95	4.3	1.419	$B_{vx}(cont)$	0.493	$V_x(cont)$	21.1
					$B_{vx}(disc)$	0.34	$V_x(disc)$	14.5
					$B_{vy}(cont)$	0.36	$V_y(cont)$	15.4
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P2	TYPE 1	9.95	4.6	1.326	$B_{vx}(cont)$	0.415	$V_x(cont)$	19.0
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.33	$V_y(cont)$	15.1
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P3	TYPE 3	9.95	3.6	1.068	$B_{vx}(cont)$	0.387	$V_x(cont)$	13.9
					$B_{vx}(disc)$	0.26	$V_x(disc)$	9.3
					$B_{vy}(cont)$	0.36	$V_y(cont)$	12.9
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P4	TYPE 1	9.95	4.6	1.326	$B_{vx}(cont)$	0.41	$V_x(cont)$	18.8
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.33	$V_y(cont)$	15.1
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P5	TYPE 3	9.95	4.3	1.419	$B_{vx}(cont)$	0.494	$V_x(cont)$	21.1
					$B_{vx}(disc)$	0.324	$V_x(disc)$	13.9
					$B_{vy}(cont)$	0.36	$V_y(cont)$	15.4
					$B_{vy}(disc)$	0	$V_y(disc)$	0.0
P6	TYPE 2	9.95	2.8	1.05	$B_{vx}(cont)$	0.376	$V_x(cont)$	10.5
					$B_{vx}(disc)$	0	$V_x(disc)$	0.0
					$B_{vy}(cont)$	0.36	$V_y(cont)$	10.0

					$B_{vy}(\text{disc})$	0.24	$V_y(\text{disc})$	6.7
P7	TYPE 1	9.95	2.8	1.643	$B_{vx}(\text{cont})$	0.467	$V_x(\text{cont})$	13.0
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	9.2
						0	$V_y(\text{disc})$	0.0
P8	TYPE 1	9.95	2.8	1.286	$B_{vx}(\text{cont})$	0.407	$V_x(\text{cont})$	11.3
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	9.2
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.0
P9	TYPE 1	9.95	2.8	1.643	$B_{vx}(\text{cont})$	0.467	$V_x(\text{cont})$	13.0
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	9.2
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.0
P10	TYPE 2	9.95	2.8	1.05	$B_{vx}(\text{cont})$	0.376	$V_x(\text{cont})$	10.5
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.0
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	10.0
					$B_{vy}(\text{disc})$	0.24	$V_y(\text{disc})$	6.7
P11	TYPE 3	9.95	4.3	1.419	$B_{vx}(\text{cont})$	0.493	$V_x(\text{cont})$	21.1
					$B_{vx}(\text{disc})$	0.34	$V_x(\text{disc})$	14.5
					$B_{vy}(\text{cont})$	0.36	$V_y(\text{cont})$	15.40
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00
P12	TYPE 1	9.95	4.6	1.326	$B_{vx}(\text{cont})$	0.415	$V_x(\text{cont})$	18.99
					$B_{vx}(\text{disc})$	0	$V_x(\text{disc})$	0.00
					$B_{vy}(\text{cont})$	0.33	$V_y(\text{cont})$	15.10
					$B_{vy}(\text{disc})$	0	$V_y(\text{disc})$	0.00

P13	TYPE 2	9.95	3.6	1.694	$B_{vx}(cont)$	0.493	$V_x(cont)$	17.66
					$B_{vx}(disc)$	0	$V_x(disc)$	0.00
					$B_{vy}(cont)$	0.36	$V_y(cont)$	12.90
					$B_{vy}(disc)$	0.24	$V_y(disc)$	8.60
P14	TYPE 1	9.95	4.6	1.326	$B_{vx}(cont)$	0.41	$V_x(cont)$	18.77
					$B_{vx}(disc)$	0	$V_x(disc)$	0.00
					$B_{vy}(cont)$	0.33	$V_y(cont)$	15.10
					$B_{vy}(disc)$	0	$V_y(disc)$	0.00
P15	TYPE 3	9.95	4.3	1.419	$B_{vx}(cont)$	0.494	$V_x(cont)$	21.14
					$B_{vx}(disc)$	0.324	$V_x(disc)$	13.86
					$B_{vy}(cont)$	0.36	$V_y(cont)$	15.40
					$B_{vy}(disc)$	0	$V_y(disc)$	0.00

11.3.10 Appendix D-5 Reinforcement calculation for individual panel of roof slab

Table 11.4 Roof plan Reinforcement

Panel	Depth	Moment		Ast= Msd/Z*fy d	Ø	Spacing	Spacing used	Reinforcement		
		Location	Value							
P1	149.42	Mxs =	12.70	271.517	10	289.116	250	10	c/c	250
		Mxf =	10.38	221.917	10	353.735	350	10	c/c	350
		Mys =	5.90	126.138	10	622.334	600	10	c/c	600
		Myf =	6.04	129.131	10	607.909	600	10	c/c	600
P2	138.53	Mxs =	11.45	264.033	10	297.312	250	10	c/c	250
		Mxf =	8.09	186.552	10	420.793	400	10	c/c	400
		Mys =	5.98	137.897	10	569.267	550	10	c/c	550
		Myf =	5.06	116.682	10	672.770	630	10	c/c	630
P3	101.49	Mxs =	8.95	281.711	10	278.655	250	10	c/c	250
		Mxf =	6.03	189.801	10	413.592	400	10	c/c	400
		Mys =	4.53	142.587	10	550.543	500	10	c/c	500
		Myf =	3.74	117.720	10	666.834	630	10	c/c	630
P4	138.53	Mxs =	8.95	206.384	10	380.359	350	10	c/c	350
		Mxf =	8.09	186.552	10	420.793	400	10	c/c	400
		Mys =	5.98	137.897	10	569.267	550	10	c/c	550
		Myf =	5.06	116.682	10	672.770	600	10	c/c	600
P5	149.42	Mxs =	11.45	244.793	10	320.679	300	10	c/c	300
		Mxf =	10.51	224.697	10	349.360	300	10	c/c	300
		Mys =	5.97	127.635	10	615.037	600	10	c/c	600
		Myf =	6.05	129.345	10	606.904	600	10	c/c	600
P6	97.29	Mxs =	5.90	193.712	10	405.241	400	10	c/c	400
		Mxf =	2.49	81.753	10	960.209	630	10	c/c	630
		Mys =	3.00	98.498	10	796.974	630	10	c/c	630
		Myf =	2.36	77.485	10	1013.102	630	10	c/c	630
P7	84.32	Mxs =	5.98	226.544	10	346.510	300	10	c/c	300
		Mxf =	3.30	125.016	10	627.919	600	10	c/c	600
		Mys =	2.50	94.709	10	828.853	630	10	c/c	630
		Myf =	1.90	71.979	10	1090.596	630	10	c/c	630
P8	84.32	Mxs =	4.53	171.613	10	457.424	450	10	c/c	450
		Mxf =	2.70	102.286	10	767.456	630	10	c/c	630
		Mys =	2.50	94.709	10	828.853	630	10	c/c	630
		Myf =	1.90	71.979	10	1090.596	630	10	c/c	630
P9	84.32	Mxs =	5.98	226.544	10	346.510	300	10	c/c	300
		Mxf =	3.30	125.016	10	627.919	600	10	c/c	600
		Mys =	2.50	94.709	10	828.853	630	10	c/c	630
		Myf =	1.90	71.979	10	1090.596	630	10	c/c	630

P10	97.29	Mxs =	5.97	196.010	10	400.489	400	10	c/c	400
		Mxf =	2.49	81.753	10	960.209	630	10	c/c	630
		Mys =	2.75	90.289	10	869.426	630	10	c/c	630
		Myf =	2.36	77.485	10	1013.102	630	10	c/c	630
P11	149.42	Mxs =	12.70	271.517	10	289.116	250	10	c/c	250
		Mxf =	10.38	221.917	10	353.735	350	10	c/c	350
		Mys =	5.90	126.138	10	622.334	600	10	c/c	600
		Myf =	6.04	129.131	10	607.909	600	10	c/c	600
P12	138.53	Mxs =	11.45	264.033	10	297.312	250	10	c/c	250
		Mxf =	8.09	186.552	10	420.793	400	10	c/c	400
		Mys =	5.98	137.897	10	569.267	550	10	c/c	550
		Myf =	5.06	116.682	10	672.770	630	10	c/c	630
P13	125.09	Mxs =	8.95	228.551	10	343.468	300	10	c/c	300
		Mxf =	6.03	153.985	10	509.791	500	10	c/c	500
		Mys =	4.53	115.680	10	678.596	630	10	c/c	630
		Myf =	3.74	95.506	10	821.936	630	10	c/c	630
P14	138.53	Mxs =	8.95	206.384	10	380.359	350	10	c/c	350
		Mxf =	8.09	186.552	10	420.793	400	10	c/c	400
		Mys =	5.98	137.897	10	569.267	550	10	c/c	550
		Myf =	5.06	116.682	10	672.770	630	10	c/c	630
P15	149.42	Mxs =	11.45	244.793	10	320.679	300	10	c/c	300
		Mxf =	10.51	224.697	10	349.360	300	10	c/c	300
		Mys =	5.97	127.635	10	615.037	550	10	c/c	550
		Myf =	6.05	129.345	10	606.904	550	10	c/c	550
C1	182.97	Mxs =	13.06	228.015	10	344.276	300	10	c/c	300
C2	152.47	Mxs =	9.02	188.976	10	415.396	400	10	c/c	400
C3	182.97	Mxs =	13.06	228.015	10	344.276	300	10	c/c	300
C4	182.97	Mxs =	13.06	228.015	10	344.276	300	10	c/c	300
C5	129.88	Mxs =	6.56	161.340	10	486.551	450	10	c/c	450

11.3.11 Appendix D (Beam Reinforcement calculation)

D-1 Longitudinal Reinforcement calculation

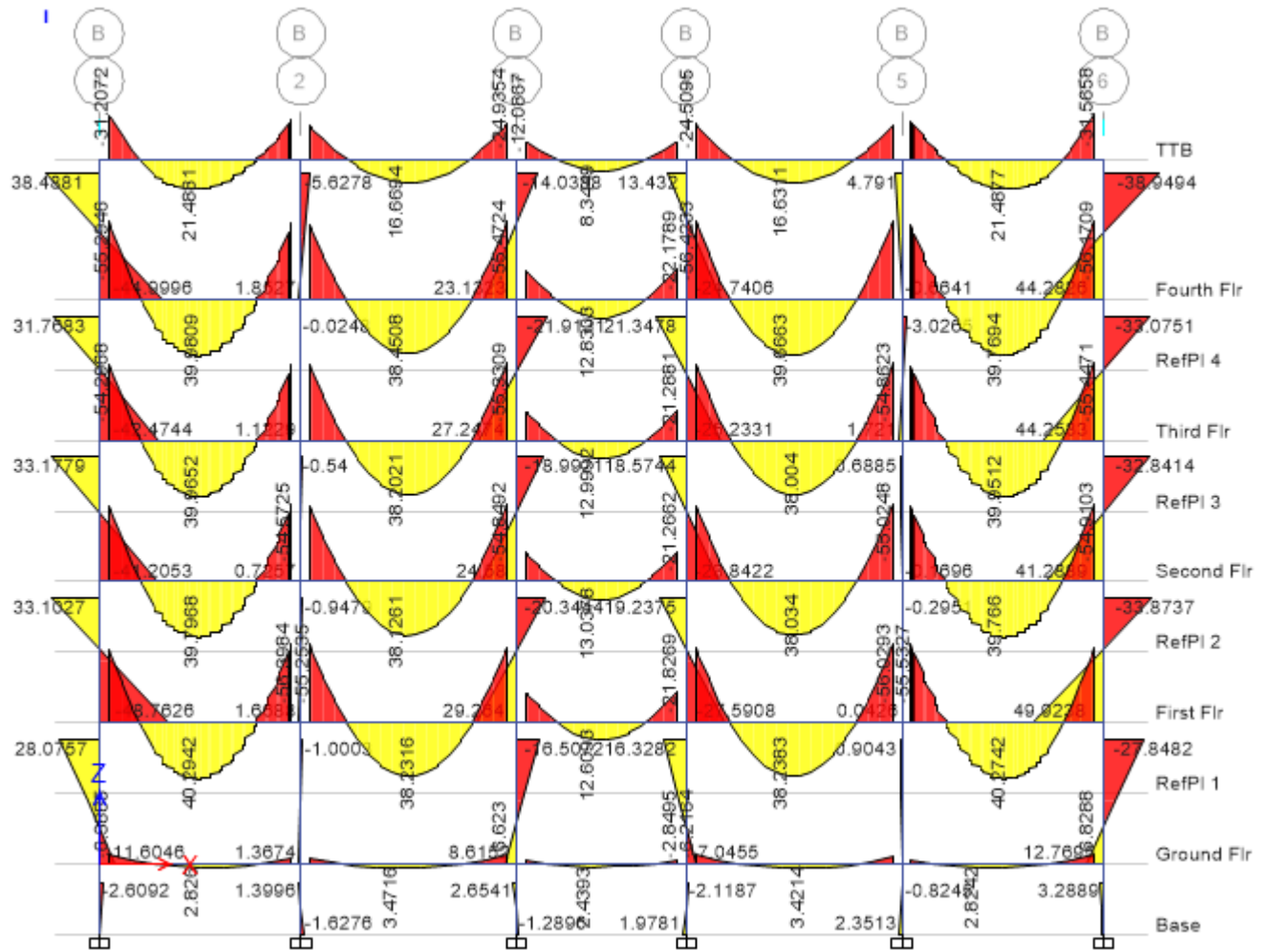


Figure 2 Bending moment diagram of beam in axis B

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

TTB

Type	location	Moment(lb(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	31.21	200	250	0.12484	218.4842	Single	71.5	2000	410.683	410.683	2.043606 3Ø16
Span	1 to 2	21.52	200	250	0.08608	229.2936	Single	71.5	2000	269.8257	269.8257	1.342684 2Ø16
Support	2	27.34	200	250	0.10936	222.9444	Single	71.5	2000	352.5615	352.5615	1.754386 2Ø16
Span	2 to 3	16.67	200	250	0.06668	234.3032	Single	71.5	2000	204.5457	204.5457	1.017843 2Ø16
Support	3	24.95	200	250	0.0998	225.6002	Single	71.5	2000	317.9538	317.9538	1.582174 2Ø16
Span	3 to 4	8.33	200	250	0.03332	242.4189	Single	71.5	2000	98.78967	98.78967	0.491589 2Ø16
Support	4	24.53	200	250	0.09812	226.0597	Single	71.5	2000	311.966	311.966	1.552379 2Ø16
Span	4 to 5	16.63	200	250	0.06652	234.3435	Single	71.5	2000	204.0198	204.0198	1.015226 2Ø16
Support	5	26.69	200	250	0.10676	223.6738	Single	71.5	2000	343.0571	343.0571	1.707091 2Ø16
Span	5 to 6	21.52	200	250	0.08608	229.2936	Single	71.5	2000	269.8257	269.8257	1.342684 2Ø16
Support	6	31.56	200	250	0.12624	218.0703	Single	71.5	2000	416.0768	416.0768	2.070446 3Ø16

4th Floor

Type	location	Moment(lb(m+G23:d(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	54.27	300	350	0.073837	325.4752	Single	150.15	4200	479.3744	479.3744	2.385422 3Ø16
Span	1 to 2	40.01	300	350	0.054435	332.2902	Single	150.15	4200	346.1657	346.1657	1.72256 2Ø16
Support	2	53.99	300	350	0.073456	325.612	Single	150.15	4200	476.7008	476.7008	2.372118 3Ø16
Span	2 to 3	38.48	300	350	0.052354	333.0039	Single	150.15	4200	332.2146	332.2146	1.653138 2Ø16
Support	3	55.48	300	350	0.075483	324.8827	Single	150.15	4200	490.9563	490.9563	2.443055 3Ø16
Span	3 to 4	12.77	300	350	0.017374	344.5487	Single	150.15	4200	106.5549	106.5549	0.530229 2Ø16
Support	4	56.41	300	350	0.076748	324.4257	Single	150.15	4200	499.8893	499.8893	2.487507 3Ø16
Span	4 to 5	39.69	300	350	0.054	332.4397	Single	150.15	4200	343.2426	343.2426	1.708014 2Ø16
Support	5	56.11	300	350	0.07634	324.5732	Single	150.15	4200	497.0047	497.0047	2.473153 3Ø16
Span	5 to 6	39.81	300	350	0.054163	332.3836	Single	150.15	4200	344.3384	344.3384	1.713467 2Ø16
Support	6	56.17	300	350	0.076422	324.5437	Single	150.15	4200	497.5814	497.5814	2.476022 3Ø16

3rd Floor

Type	location	Moment(lb(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	54.27	300	350	0.073837	325.4752	Single	150.15	4200	479.3744	479.3744	2.385422 3Ø16
Span	1 to 2	40.01	300	350	0.054435	332.2902	Single	150.15	4200	346.1657	346.1657	1.72256 2Ø16
Support	2	53.99	300	350	0.073456	325.612	Single	150.15	4200	476.7008	476.7008	2.372118 3Ø16
Span	2 to 3	38.22	300	350	0.052	333.1249	Single	150.15	4200	329.8501	329.8501	1.641372 2Ø16
Support	3	55.47	300	350	0.075469	324.8876	Single	150.15	4200	490.8604	490.8604	2.442578 3Ø16
Span	3 to 4	12.95	300	350	0.017619	344.4706	Single	150.15	4200	108.0813	108.0813	0.537825 2Ø16
Support	4	53.99	300	350	0.073456	325.612	Single	150.15	4200	476.7008	476.7008	2.372118 3Ø16
Span	4 to 5	38.02	300	350	0.051728	333.2179	Single	150.15	4200	328.0324	328.0324	1.632327 2Ø16
Support	5	54.84	300	350	0.074612	325.1964	Single	150.15	4200	484.8247	484.8247	2.412543 3Ø16
Span	5 to 6	39.96	300	350	0.054367	332.3135	Single	150.15	4200	345.7088	345.7088	1.720287 2Ø16
Support	6	55.48	300	350	0.075483	324.8827	Single	150.15	4200	490.9563	490.9563	2.443055 3Ø16

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

2nd Floor

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	53.96	300	350	0.073415	325.6266	Single	150.15	4200	476.4145	476.4145	2.370693	3Ø16
Span	1 to 2	39.83	300	350	0.05419	332.3743	Single	150.15	4200	344.5211	344.5211	1.714377	2Ø16
Support	2	54.53	300	350	0.07419	325.3481	Single	150.15	4200	481.8593	481.8593	2.397787	3Ø16
Span	2 to 3	38.15	300	350	0.051905	333.1574	Single	150.15	4200	329.2138	329.2138	1.638206	2Ø16
Support	3	54.83	300	350	0.074599	325.2013	Single	150.15	4200	484.729	484.729	2.412067	3Ø16
Span	3 to 4	12.99	300	350	0.017673	344.4532	Single	150.15	4200	108.4206	108.4206	0.539513	2Ø16
Support	4	53.82	300	350	0.073224	325.695	Single	150.15	4200	475.0787	475.0787	2.364046	3Ø16
Span	4 to 5	38.05	300	350	0.051769	333.2039	Single	150.15	4200	328.305	328.305	1.633683	2Ø16
Support	5	55.01	300	350	0.074844	325.1131	Single	150.15	4200	486.4521	486.4521	2.420642	3Ø16
Span	5 to 6	39.8	300	350	0.05415	332.3883	Single	150.15	4200	344.2471	344.2471	1.713013	2Ø16
Support	6	54.88	300	350	0.074667	325.1768	Single	150.15	4200	485.2075	485.2075	2.414448	3Ø16

1st Floor

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	51.4	300	350	0.069932	326.8714	Single	150.15	4200	452.084	452.084	2.249622	3Ø16
Span	1 to 2	40.34	300	350	0.054884	332.1358	Single	150.15	4200	349.1831	349.1831	1.737575	2Ø16
Support	2	56.4	300	350	0.076735	324.4306	Single	150.15	4200	499.7931	499.7931	2.487028	3Ø16
Span	2 to 3	38.26	300	350	0.052054	333.1063	Single	150.15	4200	330.2137	330.2137	1.643181	2Ø16
Support	3	52.37	300	350	0.071252	326.4009	Single	150.15	4200	461.2795	461.2795	2.29538	3Ø16
Span	3 to 4	12.55	300	350	0.017075	344.6441	Single	150.15	4200	104.6902	104.6902	0.52095	2Ø16
Support	4	52.48	300	350	0.071401	326.3475	Single	150.15	4200	462.3241	462.3241	2.300578	3Ø16
Span	4 to 5	38.27	300	350	0.052068	333.1016	Single	150.15	4200	330.3046	330.3046	1.643634	2Ø16
Support	5	56.04	300	350	0.076245	324.6077	Single	150.15	4200	496.3321	496.3321	2.469805	3Ø16
Span	5 to 6	40.32	300	350	0.054857	332.1452	Single	150.15	4200	349.0001	349.0001	1.736665	2Ø16
Support	6	52.28	300	350	0.071129	326.4447	Single	150.15	4200	460.4251	460.4251	2.291128	3Ø16

G Floor

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	6.44	300	400	0.006708	397.6178	Single	171.6	4800	46.5643	46.5643	0.231709	2Ø16
Span	1 to 2	2.88	300	400	0.003	398.9382	Single	171.6	4800	20.75486	20.75486	0.103279	2Ø16
Support	2	4.3	300	400	0.004479	398.4126	Single	171.6	4800	31.02904	31.02904	0.154404	2Ø16
Span	2 to 3	3.47	300	400	0.003615	398.72	Single	171.6	4800	25.02041	25.02041	0.124504	2Ø16
Support	3	6.65	300	400	0.006927	397.5396	Single	171.6	4800	48.09215	48.09215	0.239312	2Ø16
Span	3 to 4	2.48	300	400	0.002583	399.086	Single	171.6	4800	17.86562	17.86562	0.088901	2Ø16
Support	4	6.25	300	400	0.00651	397.6885	Single	171.6	4800	45.18247	45.18247	0.224833	2Ø16
Span	4 to 5	3.42	300	400	0.003563	398.7385	Single	171.6	4800	24.65874	24.65874	0.122705	2Ø16
Support	5	4.72	300	400	0.004917	398.2568	Single	171.6	4800	34.07311	34.07311	0.169552	2Ø16
Span	5 to 6	2.83	300	400	0.002948	398.9567	Single	171.6	4800	20.39359	20.39359	0.101481	2Ø16
Support	6	6.88	300	400	0.007167	397.454	Single	171.6	4800	49.76621	49.76621	0.247642	2Ø16

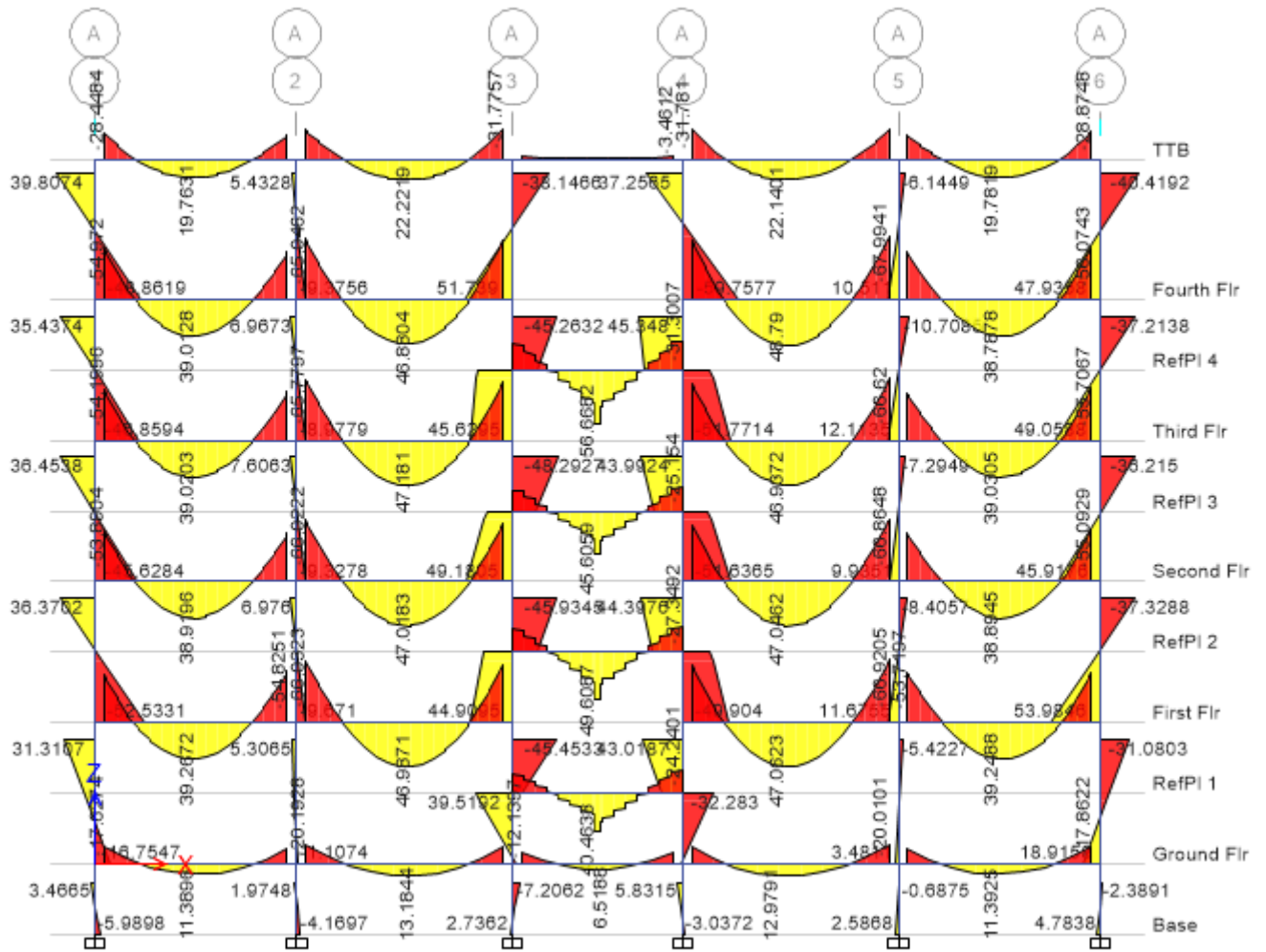


Figure 3 Bending moment diagram of beam in axis A

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

TTB

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	28.44	200	250	0.11376	221.6976	Single	71.5	2000	368.8091	368.8091	1.835236	2Ø16
Span	1 to 2	19.786	200	250	0.079144	231.1118	Single	71.5	2000	246.1324	246.1324	1.224783	2Ø16
Support	2	31.06	200	250	0.12424	218.661	Single	71.5	2000	408.3787	408.3787	2.032139	3Ø16
Span	2 to 3	22.25	200	250	0.089	228.5186	Single	71.5	2000	279.9249	279.9249	1.392938	2Ø16
Support	3	31.73	200	250	0.12692	217.8686	Single	71.5	2000	418.7053	418.7053	2.083526	3Ø16
Span	3 to 4	3.51	200	250	0.01404	246.8631	Single	71.5	2000	40.87747	40.87747	0.203411	2Ø16
Support	4	31.74	200	250	0.12696	217.8567	Single	71.5	2000	418.8601	418.8601	2.084296	3Ø16
Span	4 to 5	22.16	200	250	0.08864	228.6144	Single	71.5	2000	278.6757	278.6757	1.386722	2Ø16
Support	5	31.31	200	250	0.12524	218.3661	Single	71.5	2000	412.2216	412.2216	2.051262	3Ø16
Span	5 to 6	19.8	200	250	0.0792	231.0972	Single	71.5	2000	246.3221	246.3221	1.225727	2Ø16
Support	6	28.85	200	250	0.1154	221.2287	Single	71.5	2000	374.9188	374.9188	1.865639	2Ø16

4th Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	54.95	300	350	0.074762	325.1425	Single	150.15	4200	485.8776	485.8776	2.417783	3Ø16
Span	1 to 2	39.05	300	350	0.053129	332.7384	Single	150.15	4200	337.4047	337.4047	1.678964	2Ø16
Support	2	65.06	300	350	0.088517	320.106	Single	150.15	4200	584.3233	584.3233	2.90766	3Ø16
Span	2 to 3	46.95	300	350	0.063878	329.0112	Single	150.15	4200	410.2588	410.2588	2.041495	3Ø16
Support	3	61.83	300	350	0.084122	321.7339	Single	150.15	4200	552.5039	552.5039	2.749323	3Ø16
Span	3 to 4	57.96	300	350	0.078857	323.6609	Single	150.15	4200	514.8387	514.8387	2.561896	3Ø16
Support	4	59.44	300	350	0.080871	322.9269	Single	150.15	4200	529.185	529.185	2.633285	3Ø16
Span	4 to 5	48.86	300	350	0.066476	328.0964	Single	150.15	4200	428.1392	428.1392	2.13047	3Ø16
Support	5	67.98	300	350	0.09249	318.6185	Single	150.15	4200	613.3991	613.3991	3.052344	4Ø16
Span	5 to 6	38.82	300	350	0.052816	332.8456	Single	150.15	4200	335.3094	335.3094	1.668538	2Ø16
Support	6	56.05	300	350	0.076259	324.6027	Single	150.15	4200	496.4282	496.4282	2.470283	3Ø16

3rd Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	51.89	300	350	0.070599	326.6339	Single	150.15	4200	456.7256	456.7256	2.272719	3Ø16
Span	1 to 2	39.06	300	350	0.053143	332.7337	Single	150.15	4200	337.4958	337.4958	1.679418	2Ø16
Support	2	65.8	300	350	0.089524	319.7305	Single	150.15	4200	591.6635	591.6635	2.944186	3Ø16
Span	2 to 3	47.25	300	350	0.064286	328.8678	Single	150.15	4200	413.0602	413.0602	2.055435	3Ø16
Support	3	62.09	300	350	0.084476	321.6035	Single	150.15	4200	555.0522	555.0522	2.762003	3Ø16
Span	3 to 4	45.93	300	350	0.06249	329.4975	Single	150.15	4200	400.7535	400.7535	1.994195	2Ø16
Support	4	60.69	300	350	0.082571	322.3041	Single	150.15	4200	541.3576	541.3576	2.693857	3Ø16
Span	4 to 5	47	300	350	0.063946	328.9873	Single	150.15	4200	410.7255	410.7255	2.043817	3Ø16
Support	5	66.63	300	350	0.090653	319.3081	Single	150.15	4200	599.9192	599.9192	2.985267	3Ø16
Span	5 to 6	39.06	300	350	0.053143	332.7337	Single	150.15	4200	337.4958	337.4958	1.679418	2Ø16
Support	6	55.66	300	350	0.075728	324.7943	Single	150.15	4200	492.6832	492.6832	2.451648	3Ø16

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

2nd Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	53.82	300	350	0.073224	325.695	Single	150.15	4200	475.0787	475.0787	2.364046	3Ø16
Span	1 to 2	38.95	300	350	0.052993	332.785	Single	150.15	4200	336.4935	336.4935	1.67443	2Ø16
Support	2	66.04	300	350	0.08985	319.6085	Single	150.15	4200	594.0482	594.0482	2.956052	3Ø16
Span	2 to 3	47.08	300	350	0.064054	328.9491	Single	150.15	4200	411.4724	411.4724	2.047534	3Ø16
Support	3	61.89	300	350	0.084204	321.7038	Single	150.15	4200	553.0918	553.0918	2.752248	3Ø16
Span	3 to 4	49.99	300	350	0.068014	327.5526	Single	150.15	4200	438.7681	438.7681	2.18336	3Ø16
Support	4	60.69	300	350	0.082571	322.3041	Single	150.15	4200	541.3576	541.3576	2.693857	3Ø16
Span	4 to 5	47.12	300	350	0.064109	328.93	Single	150.15	4200	411.846	411.846	2.049393	3Ø16
Support	5	66.89	300	350	0.091007	319.1756	Single	150.15	4200	602.5104	602.5104	2.998161	3Ø16
Span	5 to 6	38.93	300	350	0.052966	332.7943	Single	150.15	4200	336.3113	336.3113	1.673524	2Ø16
Support	6	55.04	300	350	0.074884	325.0984	Single	150.15	4200	486.7394	486.7394	2.422071	3Ø16

1st Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	52.41	300	350	0.071306	326.3815	Single	150.15	4200	461.6593	461.6593	2.29727	3Ø16
Span	1 to 2	39.31	300	350	0.053483	332.6171	Single	150.15	4200	339.775	339.775	1.690759	2Ø16
Support	2	66.06	300	350	0.089878	319.5983	Single	150.15	4200	594.2471	594.2471	2.957041	3Ø16
Span	2 to 3	47.05	300	350	0.064014	328.9634	Single	150.15	4200	411.1923	411.1923	2.04614	3Ø16
Support	3	61.06	300	350	0.083075	322.1193	Single	150.15	4200	544.9705	544.9705	2.711836	3Ø16
Span	3 to 4	40.72	300	350	0.055401	331.9578	Single	150.15	4200	352.6613	352.6613	1.754883	2Ø16
Support	4	60.44	300	350	0.082231	322.4289	Single	150.15	4200	538.9189	538.9189	2.681722	3Ø16
Span	4 to 5	47.28	300	350	0.064327	328.8535	Single	150.15	4200	413.3405	413.3405	2.05683	3Ø16
Support	5	66.96	300	350	0.091102	319.1399	Single	150.15	4200	603.2084	603.2084	3.001634	4Ø16
Span	5 to 6	39.29	300	350	0.053456	332.6264	Single	150.15	4200	339.5926	339.5926	1.689852	2Ø16
Support	6	53.87	300	350	0.073293	325.6706	Single	150.15	4200	475.5557	475.5557	2.36642	3Ø16

G Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	17.64	300	400	0.018375	393.4049	Single	171.6	4800	128.9115	128.9115	0.641479	2Ø16
Span	1 to 2	11.39	300	400	0.011865	395.767	Single	171.6	4800	82.7403	82.7403	0.411725	2Ø16
Support	2	20.2	300	400	0.021042	392.429	Single	171.6	4800	147.9869	147.9869	0.7364	2Ø16
Span	2 to 3	13.2	300	400	0.01375	395.0859	Single	171.6	4800	96.05399	96.05399	0.477976	2Ø16
Support	3	17.75	300	400	0.01849	393.3631	Single	171.6	4800	129.7292	129.7292	0.645547	2Ø16
Span	3 to 4	6.48	300	400	0.00675	397.6029	Single	171.6	4800	46.85527	46.85527	0.233157	2Ø16
Support	4	18.28	300	400	0.019042	393.1614	Single	171.6	4800	133.6713	133.6713	0.665164	2Ø16
Span	4 to 5	12.99	300	400	0.013531	395.165	Single	171.6	4800	94.50692	94.50692	0.470277	2Ø16
Support	5	20.01	300	400	0.020844	392.5016	Single	171.6	4800	146.5678	146.5678	0.729338	2Ø16
Span	5 to 6	11.39	300	400	0.011865	395.767	Single	171.6	4800	82.7403	82.7403	0.411725	2Ø16
Support	6	17.88	300	400	0.018625	393.3136	Single	171.6	4800	130.6958	130.6958	0.650357	2Ø16

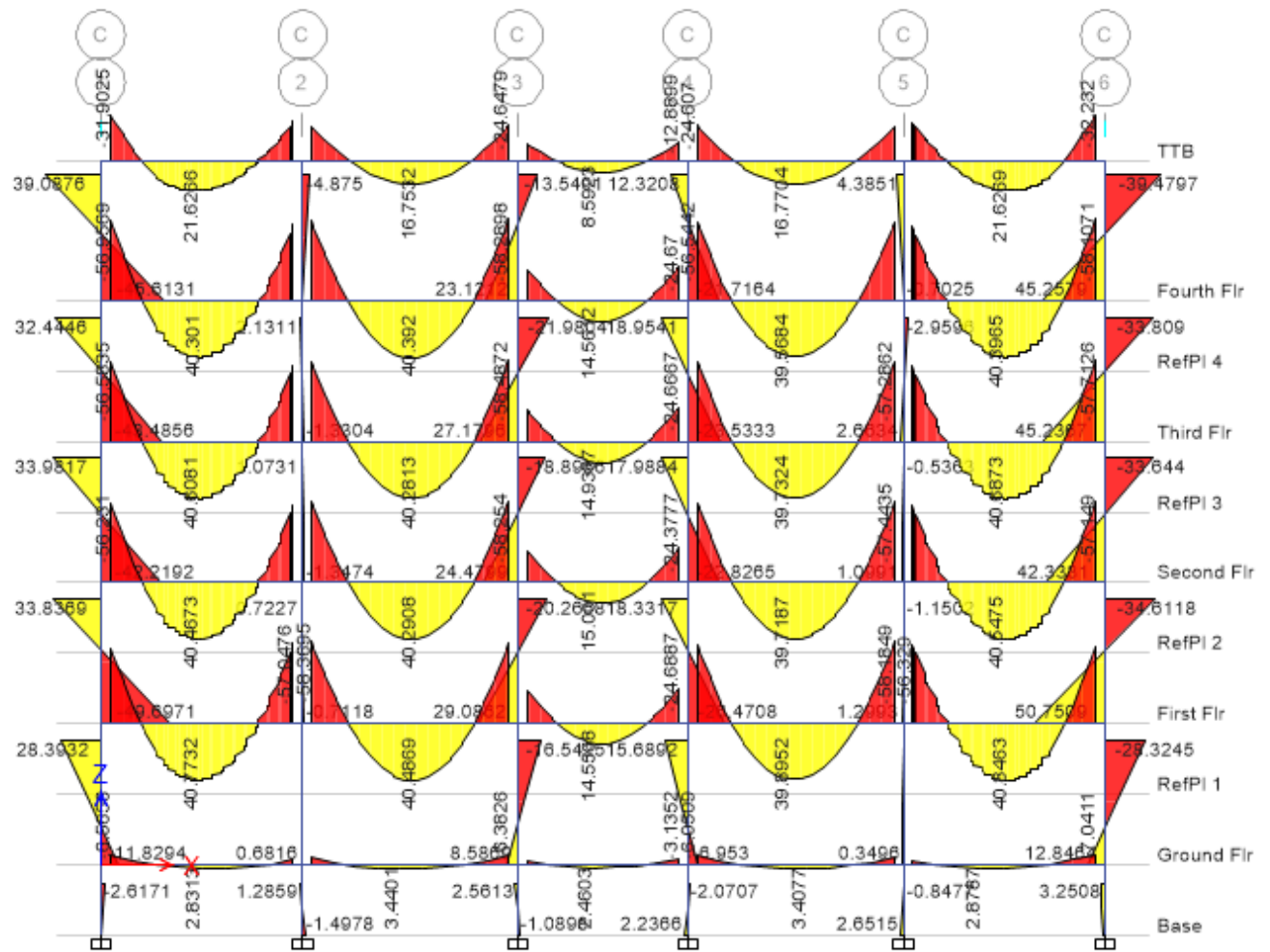


Figure 4 Bending moment diagram of beam in axis C

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

TTB

Type	location	Moment(lb(m	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	1	31.91	200	250	0.12764	217.6545	Single	71.5	2000	421.4947	421.4947	2.097406	3Ø16
Span	1 to 2	21.65	200	250	0.0866	229.156	Single	71.5	2000	271.6187	271.6187	1.351606	2Ø16
Support	2	27.07	200	250	0.10828	223.2481	Single	71.5	2000	348.6049	348.6049	1.734698	2Ø16
Span	2 to 3	16.75	200	250	0.067	234.2224	Single	71.5	2000	205.5982	205.5982	1.02308	2Ø16
Support	3	24.64	200	250	0.09856	225.9396	Single	71.5	2000	313.5316	313.5316	1.560169	2Ø16
Span	3 to 4	8.57	200	250	0.03428	242.1932	Single	71.5	2000	101.7307	101.7307	0.506223	2Ø16
Support	4	24.62	200	250	0.09848	225.9614	Single	71.5	2000	313.2468	313.2468	1.558752	2Ø16
Span	4 to 5	16.77	200	250	0.06708	234.2022	Single	71.5	2000	205.8615	205.8615	1.02439	2Ø16
Support	5	26.86	200	250	0.10744	223.4836	Single	71.5	2000	345.5361	345.5361	1.719427	2Ø16
Span	5 to 6	21.65	200	250	0.0866	229.156	Single	71.5	2000	271.6187	271.6187	1.351606	2Ø16
Support	6	32.23	200	250	0.12892	217.2727	Single	71.5	2000	426.4696	426.4696	2.122161	3Ø16

4th Floor

Type	location	Moment(lb(m+G23:	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	1	56.98	300	350	0.077524	324.1449	Single	150.15	4200	505.3779	505.3779	2.514818	3Ø16
Span	1 to 2	40.34	300	350	0.054884	332.1358	Single	150.15	4200	349.1831	349.1831	1.737575	2Ø16
Support	2	56.86	300	350	0.077361	324.204	Single	150.15	4200	504.2216	504.2216	2.509064	3Ø16
Span	2 to 3	40.42	300	350	0.054993	332.0984	Single	150.15	4200	349.915	349.915	1.741217	2Ø16
Support	3	58.25	300	350	0.079252	323.5173	Single	150.15	4200	517.6442	517.6442	2.575857	3Ø16
Span	3 to 4	14.5	300	350	0.019728	343.7966	Single	150.15	4200	121.2549	121.2549	0.603378	2Ø16
Support	4	56.53	300	350	0.076912	324.3666	Single	150.15	4200	501.044	501.044	2.493252	3Ø16
Span	4 to 5	39.59	300	350	0.053864	332.4864	Single	150.15	4200	342.3297	342.3297	1.703472	2Ø16
Support	5	56.14	300	350	0.076381	324.5585	Single	150.15	4200	497.2931	497.2931	2.474587	3Ø16
Span	5 to 6	40.44	300	350	0.05502	332.089	Single	150.15	4200	350.098	350.098	1.742128	2Ø16
Support	6	58.12	300	350	0.079075	323.5817	Single	150.15	4200	516.3862	516.3862	2.569597	3Ø16

3rd Floor

Type	location	Moment(lb(m	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	1	56.57	300	350	0.076966	324.3469	Single	150.15	4200	501.4289	501.4289	2.495168	3Ø16
Span	1 to 2	40.65	300	350	0.055306	331.9906	Single	150.15	4200	352.0203	352.0203	1.751693	2Ø16
Support	2	57.4	300	350	0.078095	323.9376	Single	150.15	4200	509.4288	509.4288	2.534976	3Ø16
Span	2 to 3	40.31	300	350	0.054844	332.1498	Single	150.15	4200	348.9086	348.9086	1.736209	2Ø16
Support	3	58.46	300	350	0.079537	323.4133	Single	150.15	4200	519.6775	519.6775	2.585975	3Ø16
Span	3 to 4	14.89	300	350	0.020259	343.6266	Single	150.15	4200	124.5779	124.5779	0.619914	2Ø16
Support	4	57.02	300	350	0.077578	324.1251	Single	150.15	4200	505.7635	505.7635	2.516737	3Ø16
Span	4 to 5	39.75	300	350	0.054082	332.4117	Single	150.15	4200	343.7905	343.7905	1.710741	2Ø16
Support	5	57.26	300	350	0.077905	324.0067	Single	150.15	4200	508.0779	508.0779	2.528254	3Ø16
Span	5 to 6	40.73	300	350	0.055415	331.9532	Single	150.15	4200	352.7529	352.7529	1.755339	2Ø16
Support	6	57.74	300	350	0.078558	323.7696	Single	150.15	4200	512.7121	512.7121	2.551314	3Ø16

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

2nd Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	56.2	300	350	0.076463	324.529	Single	150.15	4200	497.8698	497.8698	2.477457	3Ø16
Span	1 to 2	40.5	300	350	0.055102	332.0609	Single	150.15	4200	350.6471	350.6471	1.74486	2Ø16
Support	2	57.68	300	350	0.078476	323.7993	Single	150.15	4200	512.1325	512.1325	2.54843	3Ø16
Span	2 to 3	40.31	300	350	0.054844	332.1498	Single	150.15	4200	348.9086	348.9086	1.736209	2Ø16
Support	3	58.22	300	350	0.079211	323.5322	Single	150.15	4200	517.3538	517.3538	2.574412	3Ø16
Span	3 to 4	14.95	300	350	0.02034	343.6004	Single	150.15	4200	125.0894	125.0894	0.622459	2Ø16
Support	4	56.85	300	350	0.077347	324.209	Single	150.15	4200	504.1252	504.1252	2.508585	3Ø16
Span	4 to 5	39.74	300	350	0.054068	332.4164	Single	150.15	4200	343.6991	343.6991	1.710286	2Ø16
Support	5	57.43	300	350	0.078136	323.9228	Single	150.15	4200	509.7183	509.7183	2.536417	3Ø16
Span	5 to 6	40.59	300	350	0.055224	332.0187	Single	150.15	4200	351.4709	351.4709	1.74896	2Ø16
Support	6	57.13	300	350	0.077728	324.0709	Single	150.15	4200	506.824	506.824	2.522014	3Ø16

1st Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	53.01	300	350	0.072122	326.0897	Single	150.15	4200	467.3622	467.3622	2.325648	3Ø16
Span	1 to 2	40.82	300	350	0.055537	331.911	Single	150.15	4200	353.5773	353.5773	1.759441	2Ø16
Support	2	58.38	300	350	0.079429	323.4529	Single	150.15	4200	518.9027	518.9027	2.582119	3Ø16
Span	2 to 3	40.51	300	350	0.055116	332.0562	Single	150.15	4200	350.7386	350.7386	1.745316	2Ø16
Support	3	57.01	300	350	0.077565	324.1301	Single	150.15	4200	505.6671	505.6671	2.516257	3Ø16
Span	3 to 4	14.5	300	350	0.019728	343.7966	Single	150.15	4200	121.2549	121.2549	0.603378	2Ø16
Support	4	55.52	300	350	0.075537	324.863	Single	150.15	4200	491.34	491.34	2.444964	3Ø16
Span	4 to 5	39.92	300	350	0.054313	332.3322	Single	150.15	4200	345.3433	345.3433	1.718468	2Ø16
Support	5	58.19	300	350	0.07917	323.547	Single	150.15	4200	517.0635	517.0635	2.572967	3Ø16
Span	5 to 6	40.89	300	350	0.055633	331.8782	Single	150.15	4200	354.2186	354.2186	1.762632	2Ø16
Support	6	50.67	300	350	0.068939	327.2245	Single	150.15	4200	445.1825	445.1825	2.215279	3Ø16

Ground Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	6.61	300	400	0.006885	397.5545	Single	171.6	4800	47.80108	47.80108	0.237864	2Ø16
Span	1 to 2	2.82	300	400	0.002938	398.9604	Single	171.6	4800	20.32134	20.32134	0.101121	2Ø16
Support	2	4.56	300	400	0.00475	398.3162	Single	171.6	4800	32.91318	32.91318	0.16378	2Ø16
Span	2 to 3	3.44	300	400	0.003583	398.7311	Single	171.6	4800	24.80341	24.80341	0.123425	2Ø16
Support	3	6.41	300	400	0.006677	397.6289	Single	171.6	4800	46.34608	46.34608	0.230623	2Ø16
Span	3 to 4	2.47	300	400	0.002573	399.0897	Single	171.6	4800	17.79342	17.79342	0.088542	2Ø16
Support	4	6.08	300	400	0.006333	397.7517	Single	171.6	4800	43.94652	43.94652	0.218683	2Ø16
Span	4 to 5	3.41	300	400	0.003552	398.7422	Single	171.6	4800	24.58641	24.58641	0.122345	2Ø16
Support	5	4.88	300	400	0.005083	398.1975	Single	171.6	4800	35.23338	35.23338	0.175325	2Ø16
Span	5 to 6	2.88	300	400	0.003	398.9382	Single	171.6	4800	20.75486	20.75486	0.103279	2Ø16
Support	6	7.09	300	400	0.007385	397.3757	Single	171.6	4800	51.29533	51.29533	0.255251	2Ø16

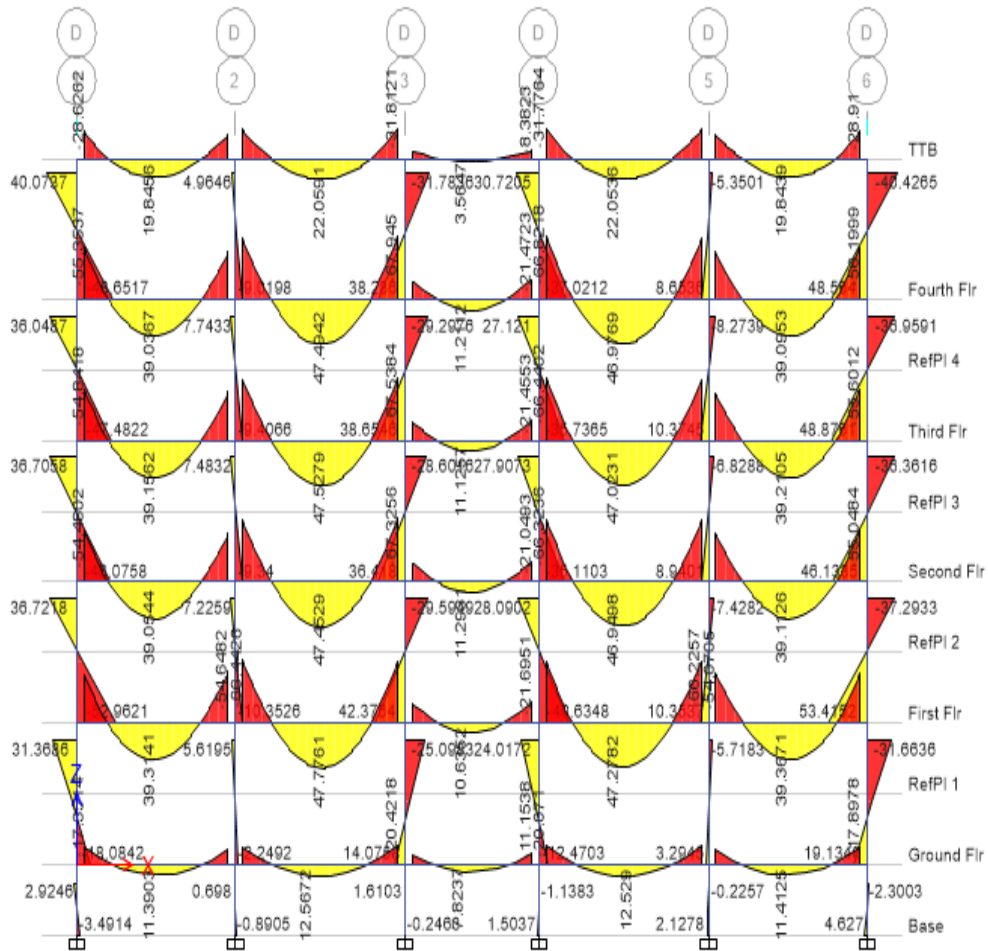


Figure 5 Bending moment diagram of beam in axis D

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

TTB

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	28.61	200	250	0.11444	221.5035	Single	71.5	2000	371.3388	371.3388	1.847824	3Ø16
Span	1 to 2	19.87	200	250	0.07948	231.0244	Single	71.5	2000	247.2708	247.2708	1.230448	2Ø16
Support	2	30.61	200	250	0.12244	219.1895	Single	71.5	2000	401.4916	401.4916	1.997868	2Ø16
Span	2 to 3	22.08	200	250	0.08832	228.6996	Single	71.5	2000	277.5663	277.5663	1.381202	2Ø16
Support	3	31.78	200	250	0.12712	217.8091	Single	71.5	2000	419.4795	419.4795	2.087378	2Ø16
Span	3 to 4	3.52	200	250	0.01408	246.854	Single	71.5	2000	40.99543	40.99543	0.203998	2Ø16
Support	4	31.76	200	250	0.12704	217.8329	Single	71.5	2000	419.1698	419.1698	2.085837	2Ø16
Span	4 to 5	22.07	200	250	0.08828	228.7102	Single	71.5	2000	277.4277	277.4277	1.380512	2Ø16
Support	5	30.64	200	250	0.12256	219.1544	Single	71.5	2000	401.9495	401.9495	2.000147	2Ø16
Span	5 to 6	19.87	200	250	0.07948	231.0244	Single	71.5	2000	247.2708	247.2708	1.230448	2Ø16
Support	6	28.89	200	250	0.11556	221.1829	Single	71.5	2000	375.5165	375.5165	1.868613	3Ø16

4th Floor

Type	location	Moment(k)	b(m+G23)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	55.33	300	350	0.075279	324.9563	Single	150.15	4200	489.5181	489.5181	2.435898	3Ø16
Span	1 to 2	39.07	300	350	0.053156	332.7291	Single	150.15	4200	337.587	337.587	1.679871	2Ø16
Support	2	64.89	300	350	0.088286	320.1922	Single	150.15	4200	582.6397	582.6397	2.899282	3Ø16
Span	2 to 3	47.54	300	350	0.06468	328.7292	Single	150.15	4200	415.7707	415.7707	2.068923	3Ø16
Support	3	67.9	300	350	0.092381	318.6595	Single	150.15	4200	612.5985	612.5985	3.04836	4Ø16
Span	3 to 4	11.13	300	350	0.015143	345.2585	Single	150.15	4200	92.67951	92.67951	0.461184	2Ø16
Support	4	60.3	300	350	0.082041	322.4987	Single	150.15	4200	537.5542	537.5542	2.674931	3Ø16
Span	4 to 5	47.01	300	350	0.063959	328.9825	Single	150.15	4200	410.8189	410.8189	2.044282	3Ø16
Support	5	64.6	300	350	0.087891	320.339	Single	150.15	4200	579.77	579.77	2.885002	3Ø16
Span	5 to 6	39.13	300	350	0.053238	332.7011	Single	150.15	4200	338.1338	338.1338	1.682593	2Ø16
Support	6	56.47	300	350	0.07683	324.3961	Single	150.15	4200	500.4666	500.4666	2.490379	3Ø16

3rd Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	54.88	300	350	0.074667	325.1768	Single	150.15	4200	485.2075	485.2075	2.414448	3Ø16
Span	1 to 2	39.19	300	350	0.05332	332.6731	Single	150.15	4200	338.6808	338.6808	1.685314	2Ø16
Support	2	65.42	300	350	0.089007	319.9235	Single	150.15	4200	587.8918	587.8918	2.925417	3Ø16
Span	2 to 3	47.57	300	350	0.064721	328.7148	Single	150.15	4200	416.0512	416.0512	2.070319	3Ø16
Support	3	67.49	300	350	0.091823	318.8692	Single	150.15	4200	608.4989	608.4989	3.02796	4Ø16
Span	3 to 4	11.04	300	350	0.01502	345.2974	Single	150.15	4200	91.91973	91.91973	0.457403	2Ø16
Support	4	66.4	300	350	0.09034	319.4253	Single	150.15	4200	597.6291	597.6291	2.973871	3Ø16
Span	4 to 5	47.06	300	350	0.064027	328.9586	Single	150.15	4200	411.2857	411.2857	2.046605	3Ø16
Support	5	64.78	300	350	0.088136	320.2479	Single	150.15	4200	581.5508	581.5508	2.893864	3Ø16
Span	5 to 6	39.24	300	350	0.053388	332.6498	Single	150.15	4200	339.1367	339.1367	1.687583	2Ø16
Support	6	55.56	300	350	0.075592	324.8434	Single	150.15	4200	491.7237	491.7237	2.446873	3Ø16

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

2nd Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	54.42	300	350	0.074041	325.4019	Single	150.15	4200	480.8077	480.8077	2.392554	3Ø16
Span	1 to 2	39.09	300	350	0.053184	332.7197	Single	150.15	4200	337.7692	337.7692	1.680778	2Ø16
Support	2	65.81	300	350	0.089537	319.7254	Single	150.15	4200	591.7628	591.7628	2.94468	3Ø16
Span	2 to 3	47.49	300	350	0.064612	328.7531	Single	150.15	4200	415.3032	415.3032	2.066596	3Ø16
Support	3	67.27	300	350	0.091524	318.9816	Single	150.15	4200	606.3016	606.3016	3.017026	4Ø16
Span	3 to 4	11.21	300	350	0.015252	345.224	Single	150.15	4200	93.35501	93.35501	0.464545	2Ø16
Support	4	66.28	300	350	0.090177	319.4864	Single	150.15	4200	596.435	596.435	2.967929	3Ø16
Span	4 to 5	46.99	300	350	0.063932	328.9921	Single	150.15	4200	410.6322	410.6322	2.043353	3Ø16
Support	5	65.37	300	350	0.088939	319.9488	Single	150.15	4200	587.3959	587.3959	2.922949	3Ø16
Span	5 to 6	39.15	300	350	0.053265	332.6917	Single	150.15	4200	338.3161	338.3161	1.6835	2Ø16
Support	6	54.99	300	350	0.074816	325.1229	Single	150.15	4200	486.2606	486.2606	2.419689	3Ø16

1st Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	51.41	300	350	0.069946	326.8665	Single	150.15	4200	452.1787	452.1787	2.250093	3Ø16
Span	1 to 2	39.36	300	350	0.053551	332.5938	Single	150.15	4200	340.231	340.231	1.693029	2Ø16
Support	2	66.43	300	350	0.090381	319.41	Single	150.15	4200	597.9277	597.9277	2.975357	3Ø16
Span	2 to 3	47.82	300	350	0.065061	328.5952	Single	150.15	4200	418.3901	418.3901	2.081957	3Ø16
Support	3	65.71	300	350	0.089401	319.7762	Single	150.15	4200	590.7698	590.7698	2.939738	3Ø16
Span	3 to 4	10.54	300	350	0.01434	345.5132	Single	150.15	4200	87.70189	87.70189	0.436415	2Ø16
Support	4	64.51	300	350	0.087769	320.3845	Single	150.15	4200	578.8799	578.8799	2.880573	3Ø16
Span	4 to 5	47.32	300	350	0.064381	328.8344	Single	150.15	4200	413.7143	413.7143	2.05869	3Ø16
Support	5	66.22	300	350	0.090095	319.5169	Single	150.15	4200	595.8381	595.8381	2.964959	3Ø16
Span	5 to 6	39.4	300	350	0.053605	332.5751	Single	150.15	4200	340.5959	340.5959	1.694844	2Ø16
Support	6	53.3	300	350	0.072517	325.9485	Single	150.15	4200	470.1226	470.1226	2.339384	3Ø16

Ground Floor

Type	location	Moment(k)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	1	17.36	300	400	0.018083	393.5113	Single	171.6	4800	126.831	126.831	0.631126	2Ø16
Span	1 to 2	11.39	300	400	0.011865	395.767	Single	171.6	4800	82.7403	82.7403	0.411725	2Ø16
Support	2	19.7	300	400	0.020521	392.62	Single	171.6	4800	144.2536	144.2536	0.717823	2Ø16
Span	2 to 3	12.57	300	400	0.013094	395.3232	Single	171.6	4800	91.41467	91.41467	0.45489	2Ø16
Support	3	20.44	300	400	0.021292	392.3372	Single	171.6	4800	149.7802	149.7802	0.745323	2Ø16
Span	3 to 4	7.83	300	400	0.008156	397.0998	Single	171.6	4800	56.68851	56.68851	0.282089	2Ø16
Support	4	20.09	300	400	0.020927	392.471	Single	171.6	4800	147.1653	147.1653	0.732311	2Ø16
Span	4 to 5	12.53	300	400	0.013052	395.3383	Single	171.6	4800	91.1203	91.1203	0.453425	2Ø16
Support	5	19.04	300	400	0.019833	392.8718	Single	171.6	4800	139.3314	139.3314	0.693329	2Ø16
Span	5 to 6	11.42	300	400	0.011896	395.7557	Single	171.6	4800	82.96059	82.96059	0.412821	2Ø16
Support	6	17.91	300	400	0.018656	393.3022	Single	171.6	4800	130.9189	130.9189	0.651467	2Ø16

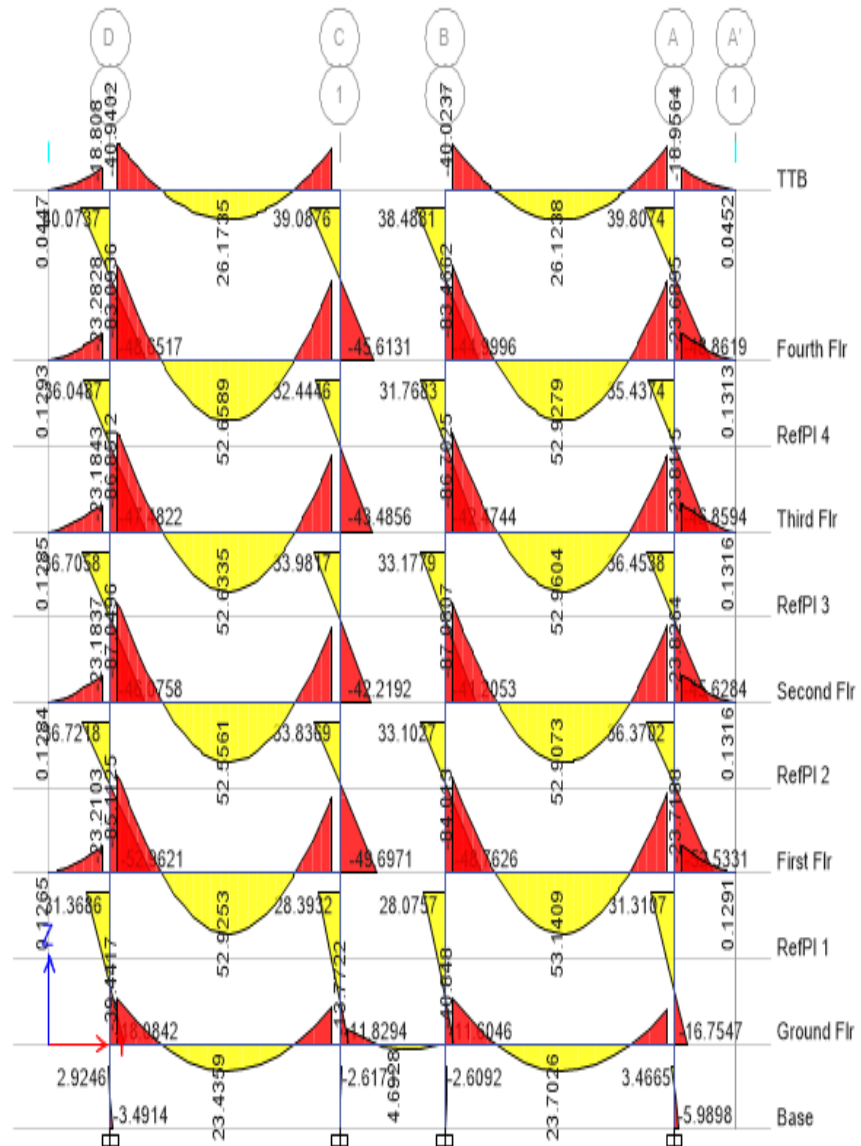


Figure 6 Bending moment diagram of beam in axis 1

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

TTB

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	40.94	200	250	0.16376	206.1949	Single	71.5	2000	570.8249	570.8249	2.84049	3Ø16
Span	D to C	26.17	200	250	0.10468	224.2534	Single	71.5	2000	335.5039	335.5039	1.669506	2Ø16
Support	C	36.26	200	250	0.14504	212.322	Single	71.5	2000	490.9821	490.9821	2.443183	3Ø16
Span	C to B	22.25	200	250	0.089	228.5186	Single	71.5	2000	279.9249	279.9249	1.392938	2Ø16
Support	B	40.02	200	250	0.16008	207.4354	Single	71.5	2000	554.6605	554.6605	2.760054	3Ø16
Span	B to A	26.12	200	250	0.10448	224.309	Single	71.5	2000	334.78	334.78	1.665903	2Ø16
Support	A	28.85	200	250	0.1154	221.2287	Single	71.5	2000	374.9188	374.9188	1.865639	2Ø16

4th Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	83.09	300	350	0.113048	310.6608	Single	150.15	4200	768.945	768.945	3.826359	4Ø16
Span	D to C	52.66	300	350	0.071646	326.26	Single	150.15	4200	464.0342	464.0342	2.309087	3Ø16
Support	C	69.73	300	350	0.094871	317.7196	Single	150.15	4200	630.9699	630.9699	3.139779	4Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	3Ø16
Support	B	83.46	300	350	0.113551	310.46	Single	150.15	4200	772.8685	772.8685	3.845882	4Ø26
Span	B to A	52.93	300	350	0.072014	326.1287	Single	150.15	4200	466.6012	466.6012	2.321861	3Ø16
Support	A	56.05	300	350	0.076259	324.6027	Single	150.15	4200	496.4282	496.4282	2.470283	3Ø16

3rd Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	86.65	300	350	0.117891	308.717	Single	150.15	4200	806.9396	806.9396	4.015424	5Ø16
Span	D to C	52.63	300	350	0.071605	326.2746	Single	150.15	4200	463.7491	463.7491	2.307669	3Ø16
Support	C	65.95	300	350	0.089728	319.6543	Single	150.15	4200	593.1537	593.1537	2.951601	3Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	3Ø16
Support	B	86.22	300	350	0.117306	308.9533	Single	150.15	4200	802.3211	802.3211	3.992442	4Ø16
Span	B to A	52.96	300	350	0.072054	326.1141	Single	150.15	4200	466.8866	466.8866	2.323281	3Ø16
Support	A	55.66	300	350	0.075728	324.7943	Single	150.15	4200	492.6832	492.6832	2.451648	3Ø16

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2nd Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	87.04	300	350	0.118422	308.5023	Single	150.15	4200	811.1355	811.1355	4.036303	5Ø16
Span	D to C	52.55	300	350	0.071497	326.3135	Single	150.15	4200	462.989	462.989	2.303886	3Ø16
Support	C	65.87	300	350	0.089619	319.6949	Single	150.15	4200	592.3589	592.3589	2.947646	3Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	87.07	300	350	0.118463	308.4858	Single	150.15	4200	811.4586	811.4586	4.037911	5Ø16
Span	B to A	52.91	300	350	0.071986	326.1384	Single	150.15	4200	466.411	466.411	2.320915	3Ø16
Support	A	55.04	300	350	0.074884	325.0984	Single	150.15	4200	486.7394	486.7394	2.422071	3Ø16

1st Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	85.11	300	350	0.115796	309.5613	Single	150.15	4200	790.4364	790.4364	3.933302	4Ø16
Span	D to C	52.93	300	350	0.072014	326.1287	Single	150.15	4200	466.6012	466.6012	2.321861	3Ø16
Support	C	67.07	300	350	0.091252	319.0837	Single	150.15	4200	604.3056	604.3056	3.007094	4Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	84.01	300	350	0.114299	310.1611	Single	150.15	4200	778.7115	778.7115	3.874958	4Ø16
Span	B to A	53.14	300	350	0.072299	326.0265	Single	150.15	4200	468.5993	468.5993	2.331804	3Ø16
Support	A	53.87	300	350	0.073293	325.6706	Single	150.15	4200	475.5557	475.5557	2.36642	3Ø16

Ground Floor

Type	location	Moment(kN-m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	39.44	300	400	0.041083	384.9298	Single	171.6	4800	294.5699	294.5699	1.465813	2Ø16
Span	D to C	23.44	300	400	0.024417	391.1867	Single	171.6	4800	172.2687	172.2687	0.857229	2Ø16
Support	C	33.21	300	400	0.034594	387.3909	Single	171.6	4800	246.4634	246.4634	1.22643	2Ø16
Span	C to B	4.69	300	400	0.004885	398.2679	Single	171.6	4800	33.85559	33.85559	0.168469	2Ø16
Support	B	40.85	300	400	0.042552	384.3682	Single	171.6	4800	305.5467	305.5467	1.520435	2Ø16
Span	B to A	23.7	300	400	0.024688	391.0867	Single	171.6	4800	174.2241	174.2241	0.866959	2Ø16
Support	A	17.88	300	400	0.018625	393.3136	Single	171.6	4800	130.6958	130.6958	0.650357	2Ø16

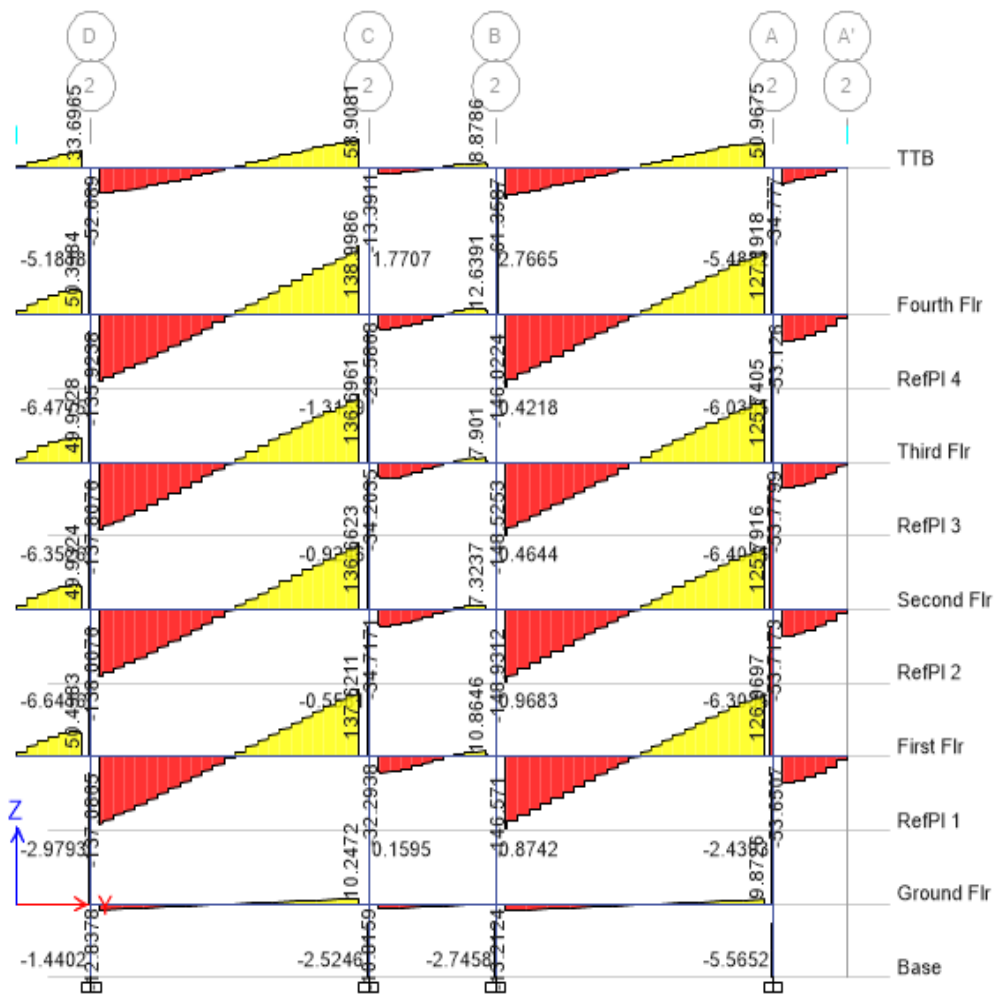


Figure 7 Bending moment diagram of beam in axis 2

TTB

Type	location	Moment(kb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	55.46	200	250	0.22184	183.2163	Double	71.5	2000	870.2594	870.2594	4.330511	5Ø16
Span	D to C	35.37	200	250	0.14148	213.4392	Single	71.5	2000	476.4242	476.4242	2.370741	3Ø16
Support	C	51.39	200	250	0.20556	190.4758	Double	71.5	2000	775.6606	775.6606	3.859776	4Ø16
Span	C to B	1.75	200	250	0.007	248.446	Single	71.5	2000	20.25065	20.25065	0.10077	2Ø16
Support	B	55.19	200	250	0.22076	183.7257	Double	71.5	2000	863.6216	863.6216	4.29748	5Ø16
Span	B to A	35.27	200	250	0.14108	213.5639	Single	71.5	2000	474.8	474.8	2.362659	3Ø16
Support	A	28.85	200	250	0.1154	221.2287	Single	71.5	2000	374.9188	374.9188	1.865639	2Ø16

4th Floor

Type	location	Moment(kb/m+G23)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	133.77	300	350	0.182	279.6406	Double	150.15	4200	1375.281	1375.281	6.843555	7Ø16
Span	D to C	84.5	300	350	0.114966	309.8942	Single	150.15	4200	783.928	783.928	3.900915	4Ø16
Support	C	121.07	300	350	0.164721	288.2149	Single	150.15	4200	1207.683	1207.683	6.00957	7Ø16
Span	C to B	2.19	300	350	0.00298	349.0772	Single	150.15	4200	18.03664	18.03664	0.089752	2Ø16
Support	B	132.16	300	350	0.17981	280.7661	Double	150.15	4200	1353.282	1353.282	6.734087	7Ø26
Span	B to A	82.85	300	350	0.112721	310.7908	Single	150.15	4200	766.4032	766.4032	3.81371	4Ø16
Support	A	56.05	300	350	0.076259	324.6027	Single	150.15	4200	496.4282	496.4282	2.470283	3Ø16

3rd Floor

Type	location	Moment(kb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	137.75	300	350	0.187415	276.8051	Double	150.15	4200	1430.706	1430.706	7.119359	8Ø16
Span	D to C	84.22	300	350	0.114585	310.0468	Single	150.15	4200	780.9459	780.9459	3.886076	4Ø16
Support	C	117.89	300	350	0.160395	290.2621	Single	150.15	4200	1167.669	1167.669	5.810453	6Ø16
Span	C to B	4.07	300	350	0.005537	348.2812	Single	150.15	4200	33.59676	33.59676	0.167181	2Ø16
Support	B	135.53	300	350	0.184395	278.3963	Double	150.15	4200	1399.603	1399.603	6.964586	7Ø16
Span	B to A	82.84	300	350	0.112707	310.7962	Single	150.15	4200	766.2973	766.2973	3.813183	4Ø16
Support	A	55.66	300	350	0.075728	324.7943	Single	150.15	4200	492.6832	492.6832	2.451648	3Ø16

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2nd Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	138.13	300	350	0.187932	276.5302	Double	150.15	4200	1436.079	1436.079	7.146095 8Ø16
Span	D to C	84.15	300	350	0.11449	310.0849	Single	150.15	4200	780.2009	780.2009	3.882369 4Ø16
Support	C	117.87	300	350	0.160367	290.2748	Single	150.15	4200	1167.419	1167.419	5.809212 6Ø16
Span	C to B	4.65	300	350	0.006327	348.0349	Single	150.15	4200	38.41167	38.41167	0.191141 2Ø16
Support	B	136.03	300	350	0.185075	278.0401	Double	150.15	4200	1406.567	1406.567	6.999236 7Ø16
Span	B to A	82.79	300	350	0.112639	310.8233	Single	150.15	4200	765.7681	765.7681	3.81055 4Ø16
Support	A	55.04	300	350	0.074884	325.0984	Single	150.15	4200	486.7394	486.7394	2.422071 3Ø16

1st Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	135.49	300	350	0.18434	278.4248	Double	150.15	4200	1399.047	1399.047	6.961819 7Ø16
Span	D to C	85.01	300	350	0.11566	309.6159	Single	150.15	4200	789.3683	789.3683	3.927987 4Ø16
Support	C	118.35	300	350	0.16102	289.9682	Single	150.15	4200	1173.413	1173.413	5.839037 6Ø16
Span	C to B	1.52	300	350	0.002068	349.3601	Single	150.15	4200	12.50844	12.50844	0.062243 2Ø16
Support	B	132.23	300	350	0.179905	280.7174	Double	150.15	4200	1354.234	1354.234	6.738822 7Ø16
Span	B to A	83.41	300	350	0.113483	310.4872	Single	150.15	4200	772.338	772.338	3.843242 4Ø16
Support	A	53.87	300	350	0.073293	325.6706	Single	150.15	4200	475.5557	475.5557	2.36642 3Ø16

Ground Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	14.45	300	400	0.015052	394.6141	Single	171.6	4800	105.2757	105.2757	0.523864 2Ø16
Span	D to C	5.85	300	400	0.006094	397.8372	Single	171.6	4800	42.27498	42.27498	0.210365 2Ø16
Support	C	7.37	300	400	0.007677	397.2714	Single	171.6	4800	53.33511	53.33511	0.265402 2Ø16
Span	C to B	1.39	300	400	0.001448	399.4882	Single	171.6	4800	10.00331	10.00331	0.049778 2Ø16
Support	B	15.32	300	400	0.015958	394.2851	Single	171.6	4800	111.7073	111.7073	0.555868 2Ø16
Span	B to A	6.23	300	400	0.00649	397.6959	Single	171.6	4800	45.03705	45.03705	0.22411 2Ø16
Support	A	17.88	300	400	0.018625	393.3136	Single	171.6	4800	130.6958	130.6958	0.650357 2Ø16

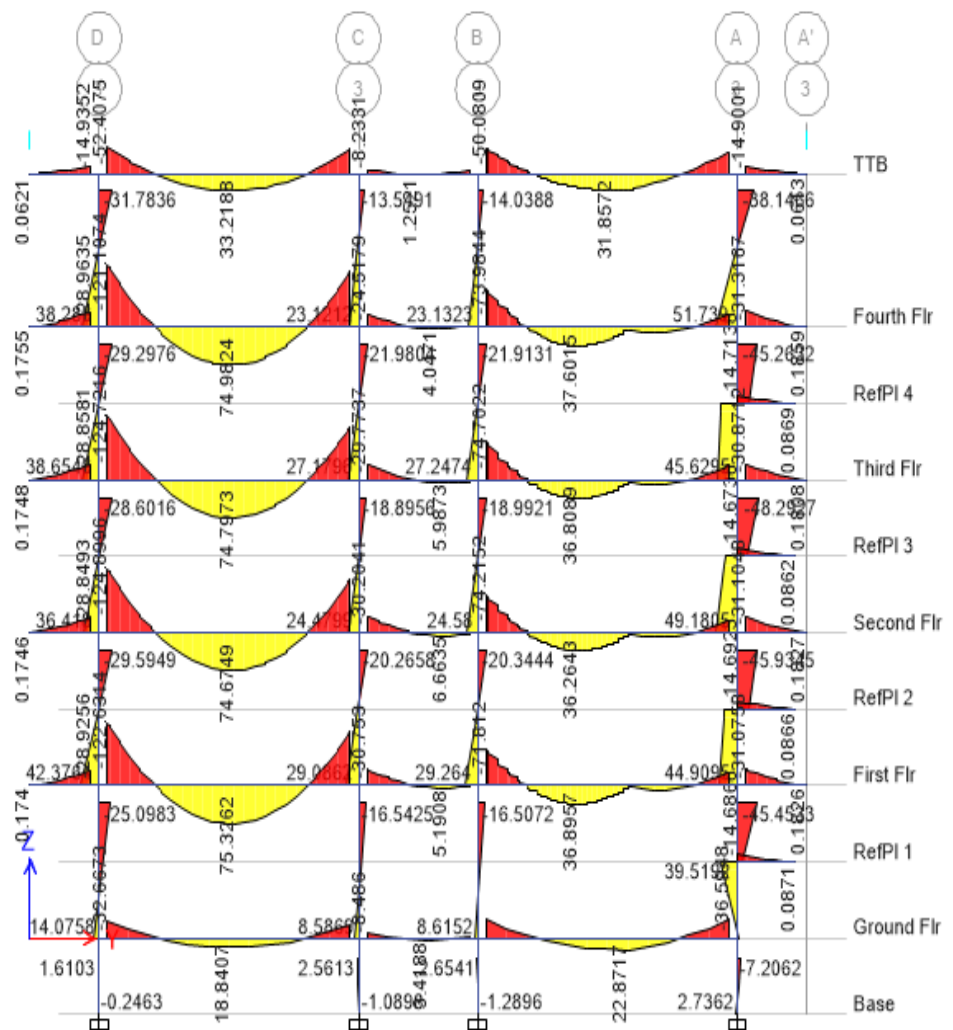


Figure 8 Bending moment diagram of beam in axis 3

TTB

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	52.41	200	250	0.20964	188.7342	Double	71.5	2000	798.356	798.356	3.972711	4Ø16
Span	D to C	33.22	200	250	0.13288	216.0815	Single	71.5	2000	441.9926	441.9926	2.199406	3Ø16
Support	C	48.14	200	250	0.19256	195.7398	Double	71.5	2000	707.0661	707.0661	3.518442	4Ø16
Span	C to B	1.25	200	250	0.005	248.892	Single	71.5	2000	14.43883	14.43883	0.071849	2Ø16
Support	B	58.08	200	250	0.23232	178.0198	Double	71.5	2000	937.975	937.975	4.667471	5Ø16
Span	B to A	31.86	200	250	0.12744	217.714	Single	71.5	2000	420.7192	420.7192	2.093547	3Ø16
Support	A	28.85	200	250	0.1154	221.2287	Single	71.5	2000	374.9188	374.9188	1.865639	2Ø16

4th Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	121.11	300	350	0.164776	288.1889	Single	150.15	4200	1208.191	1208.191	6.012098	7Ø16
Span	D to C	74.98	300	350	0.102014	314.9882	Single	150.15	4200	684.3592	684.3592	3.40545	4Ø16
Support	C	107.47	300	350	0.146218	296.729	Single	150.15	4200	1041.262	1041.262	5.181441	6Ø16
Span	C to B	4.05	300	350	0.00551	348.2897	Single	150.15	4200	33.43085	33.43085	0.166356	2Ø16
Support	B	73.98	300	350	0.100653	315.5125	Single	150.15	4200	674.1098	674.1098	3.354448	4Ø16
Span	B to A	37.6	300	350	0.051156	333.413	Single	150.15	4200	324.2189	324.2189	1.61335	2Ø16
Support	A	56.05	300	350	0.076259	324.6027	Single	150.15	4200	496.4282	496.4282	2.470283	3Ø16

3rd Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	124.72	300	350	0.169687	285.8186	Double	150.15	4200	1254.523	1254.523	6.242649	7Ø16
Span	D to C	74.78	300	350	0.101741	315.0932	Single	150.15	4200	682.3063	682.3063	3.395234	3Ø16
Support	C	104.14	300	350	0.141687	298.7245	Single	150.15	4200	1002.259	1002.259	4.987353	5Ø16
Span	C to B	5.98	300	350	0.008136	347.4687	Single	150.15	4200	49.47872	49.47872	0.246212	2Ø16
Support	B	74.7	300	350	0.101633	315.1352	Single	150.15	4200	681.4855	681.4855	3.39115	4Ø16
Span	B to A	36.81	300	350	0.050082	333.7793	Single	150.15	4200	317.0585	317.0585	1.577719	2Ø16
Support	A	55.66	300	350	0.075728	324.7943	Single	150.15	4200	492.6832	492.6832	2.451648	3Ø16

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2nd Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	124.89	300	350	0.169918	285.7057	Double	150.15	4200	1256.729	1256.729	6.253628	7Ø16
Span	D to C	74.67	300	350	0.101592	315.1509	Single	150.15	4200	681.1778	681.1778	3.389619	4Ø16
Support	C	104.19	300	350	0.141755	298.6947	Single	150.15	4200	1002.84	1002.84	4.990244	5Ø16
Span	C to B	6.66	300	350	0.009061	347.1785	Single	150.15	4200	55.15113	55.15113	0.274438	2Ø16
Support	B	74.21	300	350	0.100966	315.3921	Single	150.15	4200	676.4638	676.4638	3.366161	4Ø16
Span	B to A	36.26	300	350	0.049333	334.0338	Single	150.15	4200	312.0831	312.0831	1.552962	2Ø16
Support	A	55.04	300	350	0.074884	325.0984	Single	150.15	4200	486.7394	486.7394	2.422071	3Ø16

1st Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	122.63	300	350	0.166844	287.197	Single	150.15	4200	1227.58	1227.58	6.108578	7Ø16
Span	D to C	75.33	300	350	0.10249	314.8042	Single	150.15	4200	687.9556	687.9556	3.423346	4Ø16
Support	C	104.77	300	350	0.142544	298.3494	Single	150.15	4200	1009.589	1009.589	5.023832	6Ø16
Span	C to B	5.19	300	350	0.007061	347.8052	Single	150.15	4200	42.90069	42.90069	0.213479	2Ø16
Support	B	71.81	300	350	0.097701	316.6437	Single	150.15	4200	651.9991	651.9991	3.244422	4Ø16
Span	B to A	36.89	300	350	0.05019	333.7422	Single	150.15	4200	317.7828	317.7828	1.581324	2Ø16
Support	A	53.87	300	350	0.073293	325.6706	Single	150.15	4200	475.5557	475.5557	2.36642	3Ø16

Ground Floor

Type	location	Moment(lb/mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	32.66	300	400	0.034021	387.6067	Single	171.6	4800	242.2467	242.2467	1.205447	2Ø16
Span	D to c	18.84	300	400	0.019625	392.9481	Single	171.6	4800	137.8411	137.8411	0.685913	2Ø16
Support	C	27.57	300	400	0.028719	389.5914	Single	171.6	4800	203.4512	203.4512	1.012397	2Ø16
Span	C to B	3.42	300	400	0.003563	398.7385	Single	171.6	4800	24.65874	24.65874	0.122705	2Ø16
Support	B	35.44	300	400	0.036917	386.5137	Single	171.6	4800	263.6099	263.6099	1.311753	2Ø16
Span	B to A	22.87	300	400	0.023823	391.4059	Single	171.6	4800	167.9855	167.9855	0.835915	2Ø16
Support	A	17.88	300	400	0.018625	393.3136	Single	171.6	4800	130.6958	130.6958	0.650357	2Ø16

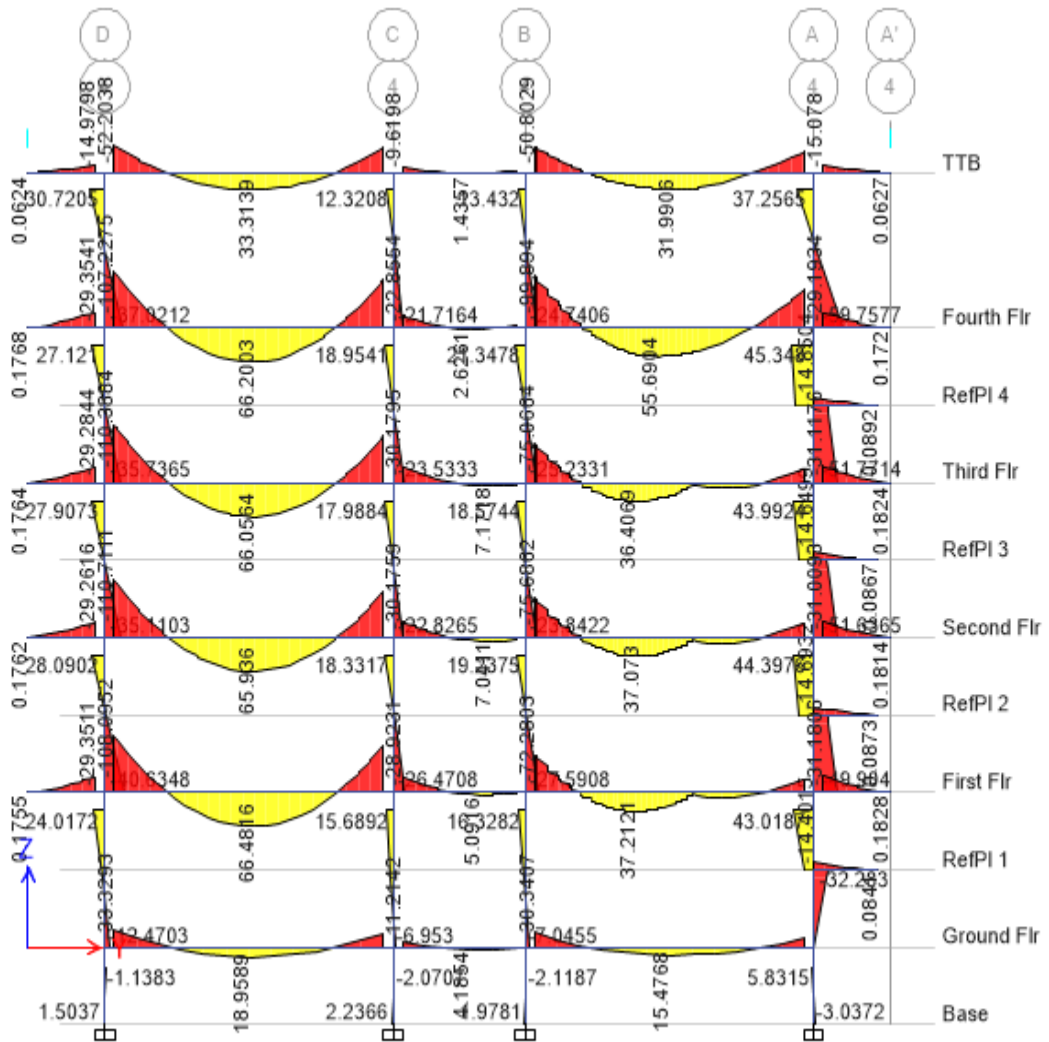


Figure 9 Bending moment diagram of beam in axis 4

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TTB

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	52.2	200	250	0.2088	189.0966	Double	71.5	2000	793.633	793.633	3.949209	4Ø16
Span	D to C	33.31	200	250	0.13324	215.9724	Single	71.5	2000	443.4139	443.4139	2.206478	3Ø16
Support	C	48.15	200	250	0.1926	195.7242	Double	71.5	2000	707.2693	707.2693	3.519453	4Ø16
Span	C to B	1.43	200	250	0.00572	248.7316	Single	71.5	2000	16.52868	16.52868	0.082249	2Ø16
Support	B	50.8	200	250	0.2032	191.4624	Double	71.5	2000	762.8044	762.8044	3.795802	4Ø16
Span	B to A	31.99	200	250	0.12796	217.5592	Single	71.5	2000	422.7365	422.7365	2.103585	3Ø16
Support	A	28.85	200	250	0.1154	221.2287	Single	71.5	2000	374.9188	374.9188	1.865639	2Ø16

4th Floor

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	107.22	300	350	0.145878	296.88	Single	150.15	4200	1038.312	1038.312	5.16676	6Ø16
Span	D to C	66.2	300	350	0.090068	319.5271	Single	150.15	4200	595.6392	595.6392	2.963969	3Ø16
Support	C	93.29	300	350	0.126925	305.0138	Single	150.15	4200	879.3232	879.3232	4.375613	5Ø16
Span	C to B	2.63	300	350	0.003578	348.8913	Single	150.15	4200	21.67198	21.67198	0.107842	2Ø16
Support	B	99.89	300	350	0.135905	301.2254	Single	150.15	4200	953.3743	953.3743	4.7441	5Ø16
Span	B to A	55.69	300	350	0.075769	324.7796	Single	150.15	4200	492.9711	492.9711	2.453081	3Ø16
Support	A	56.05	300	350	0.076259	324.6027	Single	150.15	4200	496.4282	496.4282	2.470283	3Ø16

3rd Floor

Type	location	Moment(kN)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	110.39	300	350	0.15019	294.952	Single	150.15	4200	1075.998	1075.998	5.354289	6Ø16
Span	D to C	66.06	300	350	0.089878	319.5983	Single	150.15	4200	594.2471	594.2471	2.957041	3Ø16
Support	C	89.84	300	350	0.122231	306.9509	Single	150.15	4200	841.4607	841.4607	4.187205	5Ø16
Span	C to B	7.17	300	350	0.009755	346.9605	Single	150.15	4200	59.41171	59.41171	0.29564	2Ø16
Support	B	75.07	300	350	0.102136	314.9409	Single	150.15	4200	685.2836	685.2836	3.41005	4Ø16
Span	B to A	36.41	300	350	0.049537	333.9644	Single	150.15	4200	313.4393	313.4393	1.55971	2Ø16
Support	A	55.66	300	350	0.075728	324.7943	Single	150.15	4200	492.6832	492.6832	2.451648	3Ø16

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

2nd Floor

Type	location	Moment(l	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	110.71	300	350	0.150626	294.7556	Single	150.15	4200	1079.836	1079.836	5.373387	6Ø16
Span	D to C	65.93	300	350	0.089701	319.6644	Single	150.15	4200	592.955	592.955	2.950612	3Ø16
Support	C	89.68	300	350	0.122014	307.04	Single	150.15	4200	839.7182	839.7182	4.178534	5Ø16
Span	C to B	7.04	300	350	0.009578	347.0161	Single	150.15	4200	58.32517	58.32517	0.290233	2Ø16
Support	B	75.68	300	350	0.102966	314.62	Single	150.15	4200	691.5567	691.5567	3.441266	4Ø16
Span	B to A	37.07	300	350	0.050435	333.6588	Single	150.15	4200	319.4133	319.4133	1.589437	2Ø16
Support	A	55.04	300	350	0.074884	325.0984	Single	150.15	4200	486.7394	486.7394	2.422071	3Ø16

1st Floor

Type	location	Moment(l	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	108.09	300	350	0.147061	296.3539	Single	150.15	4200	1048.595	1048.595	5.21793	6Ø16
Span	D to C	66.48	300	350	0.090449	319.3846	Single	150.15	4200	598.4255	598.4255	2.977834	3Ø16
Support	C	90.97	300	350	0.123769	306.3196	Single	150.15	4200	853.8005	853.8005	4.248609	5Ø16
Span	C to B	5.09	300	350	0.006925	347.8478	Single	150.15	4200	42.06894	42.06894	0.20934	2Ø16
Support	B	72.28	300	350	0.09834	316.3995	Single	150.15	4200	656.7731	656.7731	3.268178	4Ø16
Span	B to A	37.21	300	350	0.050626	333.5939	Single	150.15	4200	320.682	320.682	1.59575	2Ø16
Support	A	53.87	300	350	0.073293	325.6706	Single	150.15	4200	475.5557	475.5557	2.36642	3Ø16

Ground Floor

Type	location	Moment(l	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	33.29	300	400	0.034677	387.3595	Single	171.6	4800	247.0771	247.0771	1.229484	2Ø16
Span	D to C	18.96	300	400	0.01975	392.9023	Single	171.6	4800	138.7352	138.7352	0.690362	2Ø16
Support	C	26.67	300	400	0.027781	389.9402	Single	171.6	4800	196.6337	196.6337	0.978472	2Ø16
Span	C to B	4.18	300	400	0.004354	398.457	Single	171.6	4800	30.15975	30.15975	0.150078	2Ø16
Support	B	30.34	300	400	0.031604	388.5139	Single	171.6	4800	224.5132	224.5132	1.117204	2Ø16
Span	B to A	15.47	300	400	0.016115	394.2283	Single	171.6	4800	112.8173	112.8173	0.561392	2Ø16
Support	A	17.88	300	400	0.018625	393.3136	Single	171.6	4800	130.6958	130.6958	0.650357	2Ø16

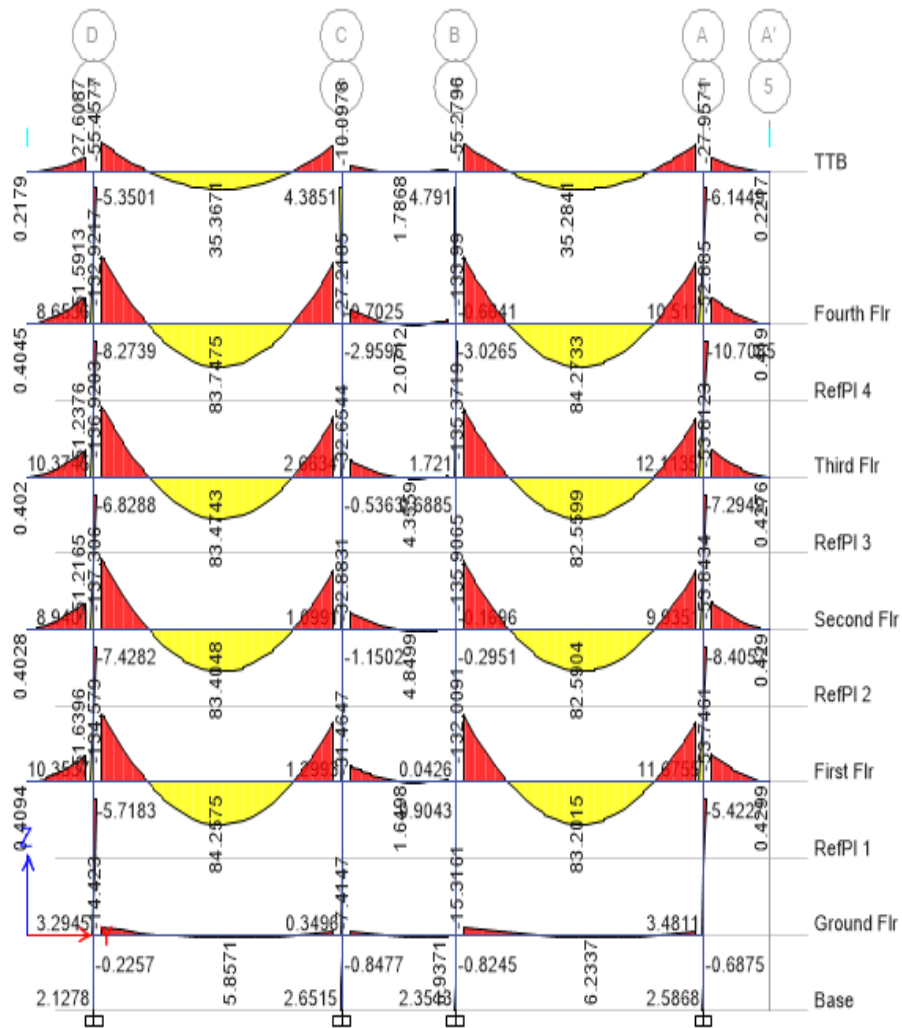


Figure 10 Bending moment diagram of beam in axis 4

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

TTB

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	55.46	200	250	0.22184	183.2163	Double	71.5	2000	870.2594	870.2594	4.330511	5Ø16
Span	D to C	35.37	200	250	0.14148	213.4392	Single	71.5	2000	476.4242	476.4242	2.370741	3Ø16
Support	C	51.34	200	250	0.20536	190.56	Double	71.5	2000	774.5636	774.5636	3.854317	4Ø16
Span	C to B	1.79	200	250	0.00716	248.4102	Single	71.5	2000	20.71651	20.71651	0.103088	2Ø16
Support	B	55.28	200	250	0.22112	183.5564	Double	71.5	2000	865.8278	865.8278	4.308458	5Ø16
Span	B to A	35.28	200	250	0.14112	213.5514	Single	71.5	2000	474.9623	474.9623	2.363467	3Ø16
Support	A	51.71	200	250	0.20684	189.9344	Double	71.5	2000	782.7152	782.7152	3.894881	4Ø16

4th Floor

Type	location	Moment(kN/m+G23)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	132.92	300	350	0.180844	280.2363	Double	150.15	4200	1363.637	1363.637	6.785616	7Ø16
Span	D to C	83.75	300	350	0.113946	310.3025	Single	150.15	4200	775.9478	775.9478	3.861205	4Ø16
Support	C	119.84	300	350	0.163048	289.0111	Single	150.15	4200	1192.121	1192.121	5.932129	6Ø16
Span	C to B	2.07	300	350	0.002816	349.1279	Single	150.15	4200	17.04585	17.04585	0.084822	2Ø16
Support	B	133.99	300	350	0.182299	279.4859	Double	150.15	4200	1378.305	1378.305	6.858605	7Ø16
Span	B to A	84.27	300	350	0.114653	310.0196	Single	150.15	4200	781.4781	781.4781	3.888725	4Ø16
Support	A	119.94	300	350	0.163184	288.9466	Single	150.15	4200	1193.382	1193.382	5.938405	6Ø16

3rd Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	136.92	300	350	0.186286	277.4029	Double	150.15	4200	1419.021	1419.021	7.061212	8Ø16
Span	D to C	83.47	300	350	0.113565	310.4546	Single	150.15	4200	772.9746	772.9746	3.84641	4Ø16
Support	C	116.84	300	350	0.158966	290.9301	Single	150.15	4200	1154.612	1154.612	5.745479	6Ø16
Span	C to B	4.35	300	350	0.005918	348.1623	Single	150.15	4200	35.92034	35.92034	0.178744	2Ø16
Support	B	135.37	300	350	0.184177	278.51	Double	150.15	4200	1397.38	1397.38	6.953523	7Ø16
Span	B to A	82.56	300	350	0.112327	310.9478	Single	150.15	4200	763.335	763.335	3.798443	4Ø16
Support	A	115.32	300	350	0.156898	291.8903	Single	150.15	4200	1135.842	1135.842	5.65208	6Ø16

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

2nd Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	137.31	300	350	0.186816	277.1224	Double	150.15	4200	1424.503	1424.503	7.088492	8Ø16
Span	D to C	83.4	300	350	0.113469	310.4926	Single	150.15	4200	772.2319	772.2319	3.842714	4Ø16
Support	C	116.58	300	350	0.158612	291.0949	Single	150.15	4200	1151.39	1151.39	5.729448	6Ø16
Span	C to B	4.85	300	350	0.006599	347.9498	Single	150.15	4200	40.07357	40.07357	0.199411	2Ø16
Support	B	135.9	300	350	0.184898	278.1328	Double	150.15	4200	1404.754	1404.754	6.990216	7Ø16
Span	B to A	82.59	300	350	0.112367	310.9316	Single	150.15	4200	763.6522	763.6522	3.800021	4Ø16
Support	A	115.22	300	350	0.156762	291.9532	Single	150.15	4200	1134.612	1134.612	5.645962	6Ø16

1st Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	134.47	300	350	0.182952	279.1475	Double	150.15	4200	1384.92	1384.92	6.891519	7Ø16
Span	D to C	84.26	300	350	0.114639	310.025	Single	150.15	4200	781.3717	781.3717	3.888195	4Ø16
Support	C	117.58	300	350	0.159973	290.4597	Single	150.15	4200	1163.806	1163.806	5.791231	6Ø16
Span	C to B	1.65	300	350	0.002245	349.3052	Single	150.15	4200	13.58038	13.58038	0.067578	2Ø16
Support	B	132.01	300	350	0.179605	280.8703	Double	150.15	4200	1351.244	1351.244	6.723947	7Ø16
Span	B to A	83.2	300	350	0.113197	310.6011	Single	150.15	4200	770.1109	770.1109	3.83216	4Ø16
Support	A	117.23	300	350	0.159497	290.6824	Single	150.15	4200	1159.453	1159.453	5.769569	6Ø16

Ground Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	14.42	300	400	0.015021	394.6254	Single	171.6	4800	105.0541	105.0541	0.522761	2Ø16
Span	D to C	5.86	300	400	0.006104	397.8335	Single	171.6	4800	42.34764	42.34764	0.210727	2Ø16
Support	C	7.41	300	400	0.007719	397.2565	Single	171.6	4800	53.62659	53.62659	0.266852	2Ø16
Span	C to B	1.08	300	400	0.001125	399.6025	Single	171.6	4800	7.770135	7.770135	0.038665	2Ø16
Support	B	15.31	300	400	0.015948	394.2888	Single	171.6	4800	111.6333	111.6333	0.5555	2Ø16
Span	B to A	6.23	300	400	0.00649	397.6959	Single	171.6	4800	45.03705	45.03705	0.22411	2Ø16
Support	A	5.78	300	400	0.006021	397.8632	Single	171.6	4800	41.7664	41.7664	0.207834	2Ø16

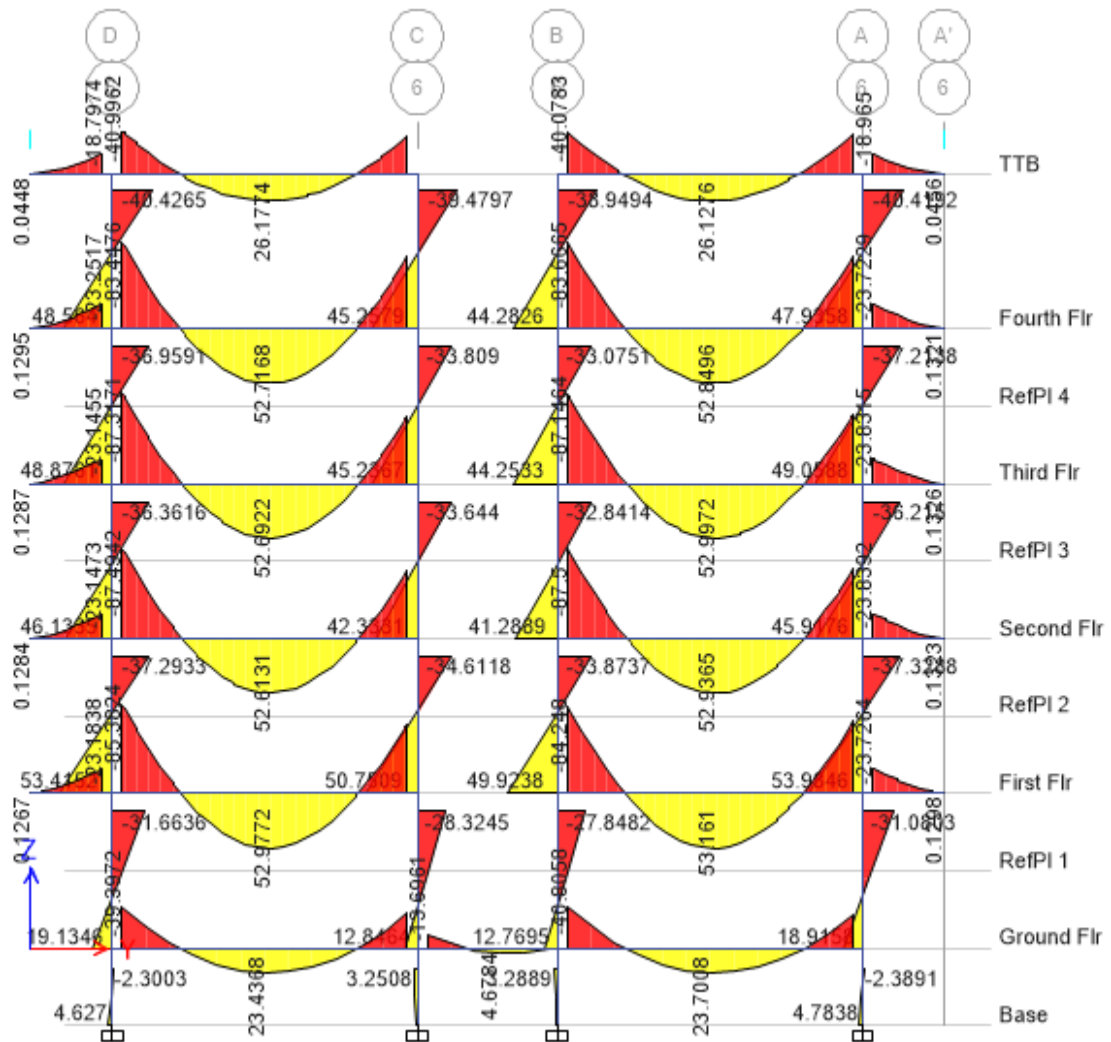


Figure 11 Bending moment diagram of beam in axis 6

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

TTB

Type	location	Moment(Ib(m+G23: d(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	83.45	300	350	0.113537	310.4655	Single	150.15	4200	772.7624	772.7624	3.845354	4Ø16
Span	D to C	52.72	300	350	0.071728	326.2308	Single	150.15	4200	464.6044	464.6044	2.311925	3Ø16
Support	C	69.47	300	350	0.094517	317.8535	Single	150.15	4200	628.3524	628.3524	3.126754	4Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	83.66	300	350	0.113823	310.3514	Single	150.15	4200	774.9917	774.9917	3.856448	4Ø26
Span	B to A	52.85	300	350	0.071905	326.1676	Single	150.15	4200	465.8404	465.8404	2.318075	3Ø16
Support	A	70.02	300	350	0.095265	317.5701	Single	150.15	4200	633.8923	633.8923	3.154321	4Ø16

4th Floor

Type	location	Moment(Ib(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	87.32	300	350	0.118803	308.348	Single	150.15	4200	814.1522	814.1522	4.051315	5Ø16
Span	D to C	52.69	300	350	0.071687	326.2454	Single	150.15	4200	464.3193	464.3193	2.310506	3Ø16
Support	C	65.58	300	350	0.089224	319.8422	Single	150.15	4200	589.4793	589.4793	2.933317	3Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	87.15	300	350	0.118571	308.4417	Single	150.15	4200	812.3202	812.3202	4.042199	5Ø16
Span	B to A	52.99	300	350	0.072095	326.0995	Single	150.15	4200	467.172	467.172	2.324701	3Ø16
Support	A	67.21	300	350	0.091442	319.0123	Single	150.15	4200	605.7027	605.7027	3.014046	4Ø16

3rd Floor

Type	location	Moment(Ib(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark	
Support	D	87.49	300	350	0.119034	308.2542	Single	150.15	4200	815.9854	815.9854	4.060437	5Ø16
Span	D to C	52.61	300	350	0.071578	326.2843	Single	150.15	4200	463.559	463.559	2.306723	3Ø16
Support	C	65.52	300	350	0.089143	319.8727	Single	150.15	4200	588.8839	588.8839	2.930354	3Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	87.5	300	350	0.119048	308.2487	Single	150.15	4200	816.0933	816.0933	4.060974	5Ø16
Span	B to A	52.91	300	350	0.071986	326.1384	Single	150.15	4200	466.411	466.411	2.320915	3Ø16
Support	A	66.94	300	350	0.091075	319.1501	Single	150.15	4200	603.0089	603.0089	3.000641	4Ø16

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

2nd Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	85.38	300	350	0.116163	309.4136	Single	150.15	4200	793.3223	793.3223	3.947663	4Ø16
Span	D to C	52.97	300	350	0.072068	326.1092	Single	150.15	4200	466.9817	466.9817	2.323754	3Ø16
Support	C	66.89	300	350	0.091007	319.1756	Single	150.15	4200	602.5104	602.5104	2.998161	3Ø16
Span	C to B	0	300	350	0	350	Single	150.15	4200	0	0	0	2Ø16
Support	B	84.28	300	350	0.114667	310.0141	Single	150.15	4200	781.5846	781.5846	3.889255	4Ø16
Span	B to A	53.16	300	350	0.072327	326.0167	Single	150.15	4200	468.7897	468.7897	2.332751	3Ø16
Support	A	69.33	300	350	0.094327	317.9255	Single	150.15	4200	626.944	626.944	3.119745	4Ø16

1st Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	39.39	300	400	0.041031	384.9497	Single	171.6	4800	294.1812	294.1812	1.46388	2Ø16
Span	D to C	23.43	300	400	0.024406	391.1906	Single	171.6	4800	172.1935	172.1935	0.856855	2Ø16
Support	C	33.25	300	400	0.034635	387.3752	Single	171.6	4800	246.7702	246.7702	1.227957	2Ø16
Span	C to B	4.68	300	400	0.004875	398.2717	Single	171.6	4800	33.78309	33.78309	0.168109	2Ø16
Support	B	40.8	300	400	0.0425	384.3882	Single	171.6	4800	305.1568	305.1568	1.518495	2Ø16
Span	B to A	23.7	300	400	0.024688	391.0867	Single	171.6	4800	174.2241	174.2241	0.866959	2Ø16
Support	A	31.31	300	400	0.032615	388.1351	Single	171.6	4800	231.9173	231.9173	1.154047	2Ø16

Ground Floor

Type	location	Moment(kN/m)	b(mm)	d(mm)	k	Z(mm)	beam type	As,min	As,max	AS,cal	AS,prov	No prov	Remark
Support	D	39.39	300	400	0.041031	384.9497	Single	171.6	4800	294.1812	294.1812	1.46388	2Ø16
Span	D to C	23.43	300	400	0.024406	391.1906	Single	171.6	4800	172.1935	172.1935	0.856855	2Ø16
Support	C	33.25	300	400	0.034635	387.3752	Single	171.6	4800	246.7702	246.7702	1.227957	2Ø16
Span	C to B	4.68	300	400	0.004875	398.2717	Single	171.6	4800	33.78309	33.78309	0.168109	2Ø16
Support	B	40.8	300	400	0.0425	384.3882	Single	171.6	4800	305.1568	305.1568	1.518495	2Ø16
Span	B to A	23.7	300	400	0.024688	391.0867	Single	171.6	4800	174.2241	174.2241	0.866959	2Ø16
Support	A	31.31	300	400	0.032615	388.1351	Single	171.6	4800	231.9173	231.9173	1.154047	2Ø16

11.3.11.1.1 D-2 shear reinforcement calculatio

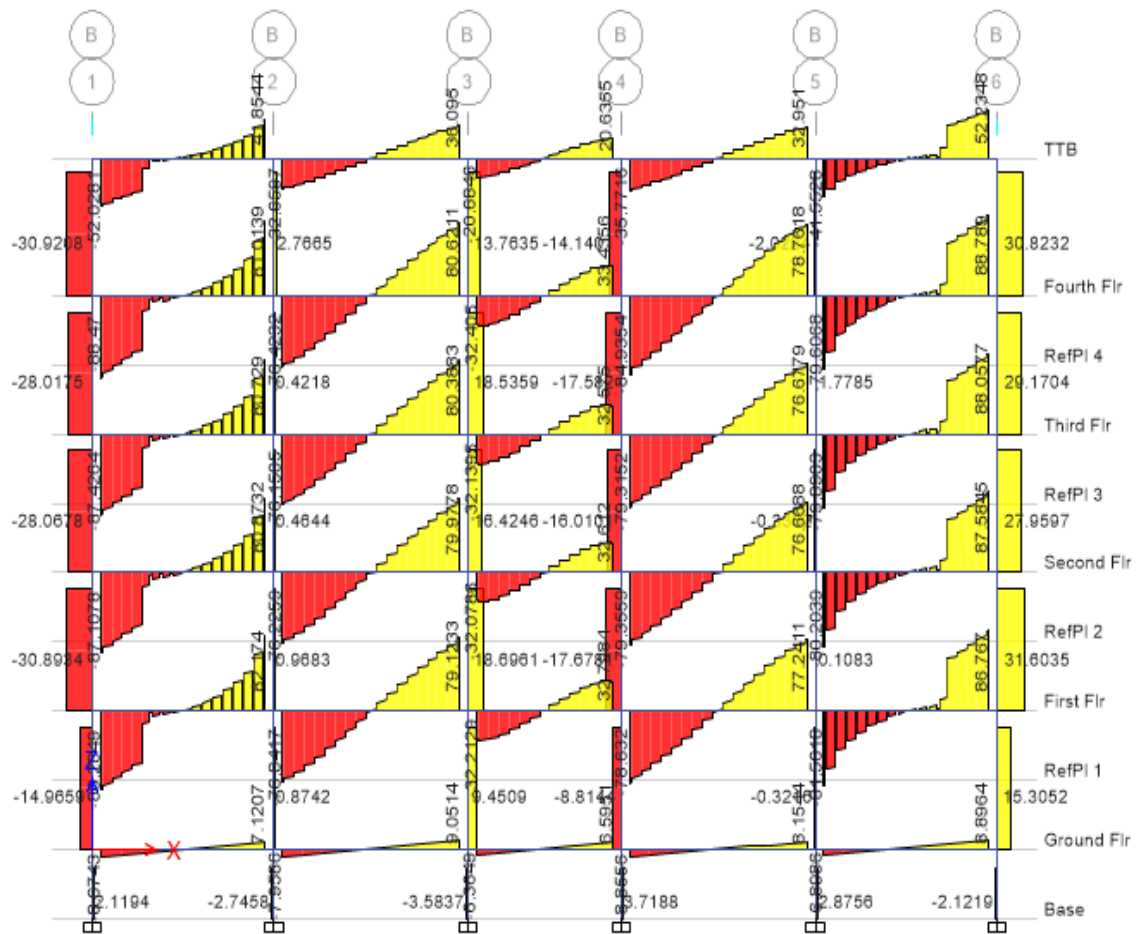


Figure 12 Axis -B shear result

TTB							
span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	145.4981	40.53488	275.4	133.85806	352.809	133.8581	130
B2-3	145.4981	29.21259	275.4	185.73911	352.809	185.7391	180
B3-4	145.4981	15.4956	275.4	350.15878	352.809	350.1588	350
B4-5	145.4981	28.65704	275.4	189.33989	352.809	189.3399	180
B5-6	145.4981	43.09321	275.4	125.91124	352.809	125.9112	125

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

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	67.60739	321.3	93.632373	294.0075	93.63237	90
B2-3	203.6974	64.9798	321.3	97.418577	294.0075	97.41858	95
B3-4	203.6974	24.43324	321.3	259.08309	294.0075	259.0831	255
B4-5	203.6974	66.20893	321.3	95.610062	294.0075	95.61006	95
B5-6	203.6974	69.53949	321.3	91.030869	294.0075	91.03087	90

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	66.80499	321.3	94.75699	294.0075	94.75699	90
B2-3	203.6974	64.70188	321.3	97.83704	294.0075	97.83704	95
B3-4	203.6974	23.20885	321.3	272.75113	294.0075	272.7511	270
B4-5	203.6974	62.11654	321.3	101.90909	294.0075	101.9091	100
B5-6	203.6974	68.93824	321.3	91.824798	294.0075	91.8248	90

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	66.54688	321.3	95.124516	294.0075	95.12452	95
B2-3	203.6974	64.25743	321.3	98.513744	294.0075	98.51374	95
B3-4	203.6974	23.25239	321.3	272.24044	294.0075	272.2404	270
B4-5	203.6974	62.06769	321.3	101.9893	294.0075	101.9893	100
B5-6	203.6974	68.54363	321.3	92.353442	294.0075	92.35344	90

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	65.89151	321.3	96.070643	294.0075	96.07064	95
B2-3	203.6974	63.30373	321.3	99.997895	294.0075	99.9979	95
B3-4	203.6974	23.49087	321.3	269.47661	294.0075	269.4766	265
B4-5	203.6974	62.06382	321.3	101.99565	294.0075	101.9957	100
B5-6	203.6974	67.90004	321.3	93.228809	294.0075	93.22881	90

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	271.5965	6.616471	367.2	1093.4167	252.0064	252.0064	250
B2-3	271.5965	7.304618	367.2	990.40909	252.0064	252.0064	250
B3-4	271.5965	4.878608	367.2	1482.9148	252.0064	252.0064	250
B4-5	271.5965	6.808692	367.2	1062.5478	252.0064	252.0064	250
B5-6	271.5965	7.119306	367.2	1016.1889	252.0064	252.0064	250

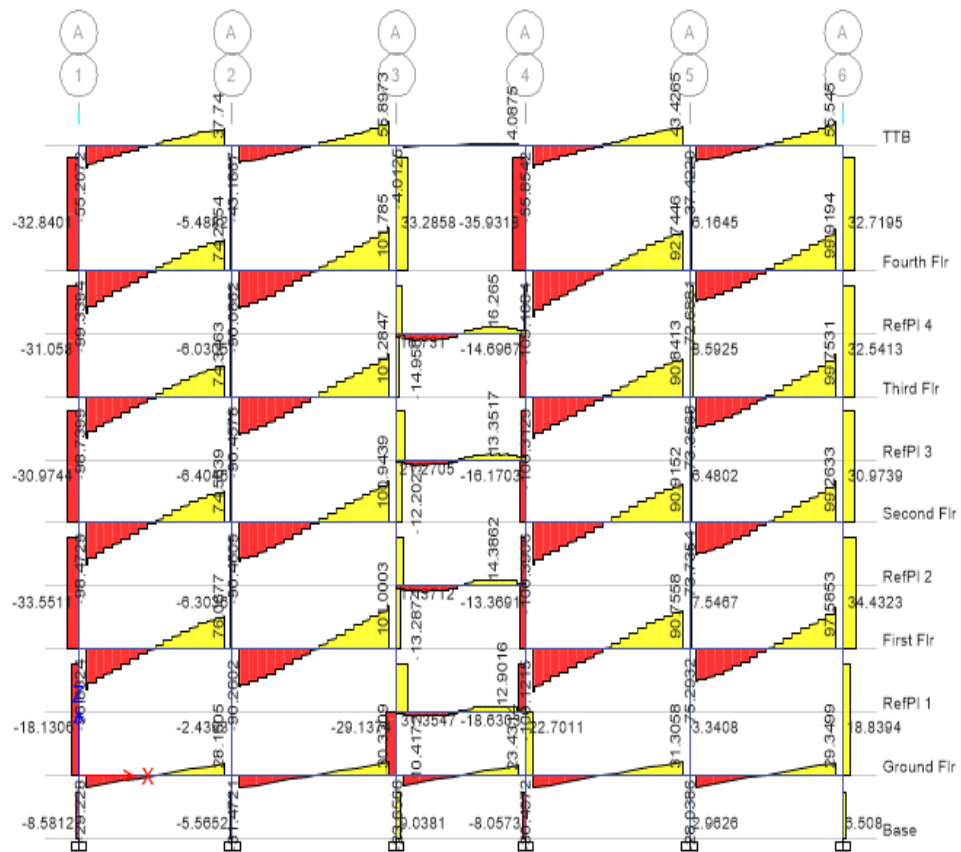


Figure 13 Axis -A shear result

TTB							
span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	145.4981	43.62041	275.4	124.38948	352.809	124.3895	120
B2-3	145.4981	44.41676	275.4	122.15931	352.809	122.1593	120
B3-4	145.4981	3.079513	275.4	1761.941	352.809	352.809	350
B4-5	145.4981	44.29745	275.4	122.48833	352.809	122.4883	120
B5-6	145.4981	43.88731	275.4	123.633	352.809	123.633	120

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	76.04435	321.3	83.244051	294.0075	83.24405	80
B2-3	203.6974	80.40259	321.3	78.731787	294.0075	78.73179	75
B3-4	203.6974	5.610488	321.3	1128.2869	294.0075	294.0075	290
B4-5	203.6974	84.07665	321.3	75.291298	294.0075	75.2913	75
B5-6	203.6974	78.44777	321.3	80.693684	294.0075	80.69368	80

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	75.71395	321.3	83.60732	294.0075	83.60732	80
B2-3	203.6974	80.0074	321.3	79.120686	294.0075	79.12069	75
B3-4	203.6974	4.635035	321.3	1365.7373	294.0075	294.0075	290
B4-5	203.6974	77.29902	321.3	81.892888	294.0075	81.89289	80
B5-6	203.6974	78.31721	321.3	80.82821	294.0075	80.82821	80

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	75.38105	321.3	83.976547	294.0075	83.97655	80
B2-3	203.6974	79.73819	321.3	79.387808	294.0075	79.38781	75
B3-4	203.6974	4.989972	321.3	1268.5923	294.0075	294.0075	290
B4-5	203.6974	77.35889	321.3	81.829505	294.0075	81.8295	80
B5-6	203.6974	77.93266	321.3	81.227045	294.0075	81.22705	80

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	74.25127	321.3	85.254299	294.0075	85.2543	85
B2-3	203.6974	81.82788	321.3	77.360429	294.0075	77.36043	75
B3-4	203.6974	4.508177	321.3	1404.1684	294.0075	294.0075	290
B4-5	203.6974	77.28461	321.3	81.908159	294.0075	81.90816	80
B5-6	203.6974	76.61525	321.3	82.623761	294.0075	82.62376	80

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	271.5965	21.88805	367.2	330.52552	252.0064	252.0064	250
B2-3	271.5965	24.08743	367.2	300.3458	252.0064	252.0064	250
B3-4	271.5965	17.21067	367.2	420.35308	252.0064	252.0064	250
B4-5	271.5965	24.8167	367.2	291.5198	252.0064	252.0064	250
B5-6	271.5965	23.03508	367.2	314.06712	252.0064	252.0064	250

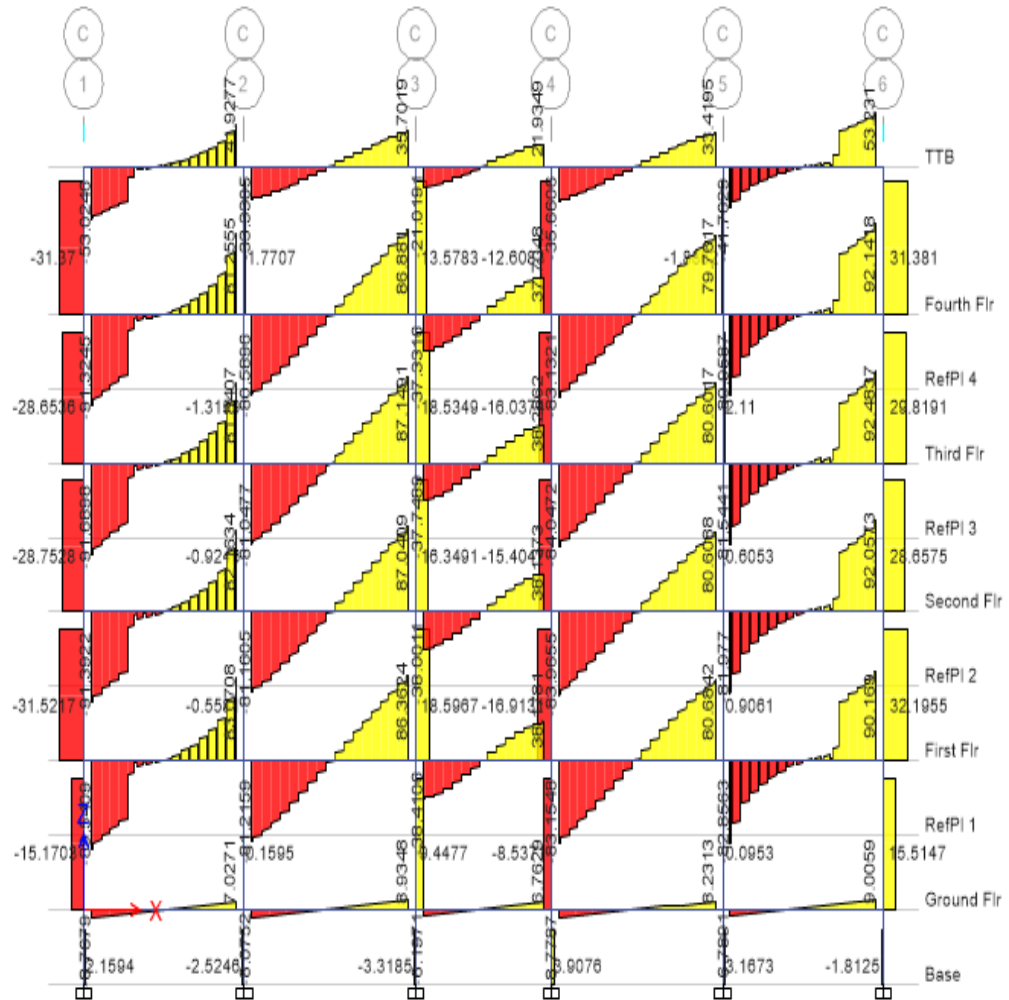


Figure 14 Axis -C shear result

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	145.4981	41.77097	275.4	129.89691	352.809	129.8969	125
B2-3	145.4981	29.02976	275.4	186.90891	352.809	186.9089	185
B3-4	145.4981	16.77833	275.4	323.38855	352.809	323.3885	320
B4-5	145.4981	28.29744	275.4	191.74596	352.809	191.746	190
B5-6	145.4981	41.87034	275.4	129.58863	352.809	129.5886	125

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	69.78875	321.3	90.705737	294.0075	90.70574	90
B2-3	203.6974	69.35682	321.3	91.270623	294.0075	91.27062	90
B3-4	203.6974	28.60322	321.3	221.31212	294.0075	221.3121	220
B4-5	203.6974	64.70417	321.3	97.833578	294.0075	97.83358	95
B5-6	203.6974	72.11388	321.3	87.781164	294.0075	87.78116	85

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	69.9469	321.3	90.500648	294.0075	90.50065	90
B2-3	203.6974	69.57084	321.3	90.989844	294.0075	90.98984	90
B3-4	203.6974	29.04428	321.3	217.95136	294.0075	217.9514	215
B4-5	203.6974	65.31092	321.3	96.924678	294.0075	96.92468	95
B5-6	203.6974	72.38146	321.3	87.456648	294.0075	87.45665	85

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	69.84049	321.3	90.638545	294.0075	90.63855	90
B2-3	203.6974	69.38395	321.3	91.234928	294.0075	91.23493	90
B3-4	203.6974	28.88217	321.3	219.17466	294.0075	219.1747	215
B4-5	203.6974	65.35282	321.3	96.862531	294.0075	96.86253	95
B5-6	203.6974	72.1621	321.3	87.722502	294.0075	87.7225	85

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	68.3096	321.3	92.669849	294.0075	92.66985	90
B2-3	203.6974	68.43249	321.3	92.503429	294.0075	92.50343	90
B3-4	203.6974	28.96227	321.3	218.56848	294.0075	218.5685	215
B4-5	203.6974	65.2858	321.3	96.961975	294.0075	96.96198	95
B5-6	203.6974	70.56988	321.3	89.70172	294.0075	89.70172	85

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	271.5965	6.722453	367.2	1076.1785	252.0064	252.0064	250
B2-3	271.5965	7.202205	367.2	1004.4924	252.0064	252.0064	250
B3-4	271.5965	5.028428	367.2	1438.732	252.0064	252.0064	250
B4-5	271.5965	6.730299	367.2	1074.9239	252.0064	252.0064	250
B5-6	271.5965	7.256804	367.2	996.93471	252.0064	252.0064	130

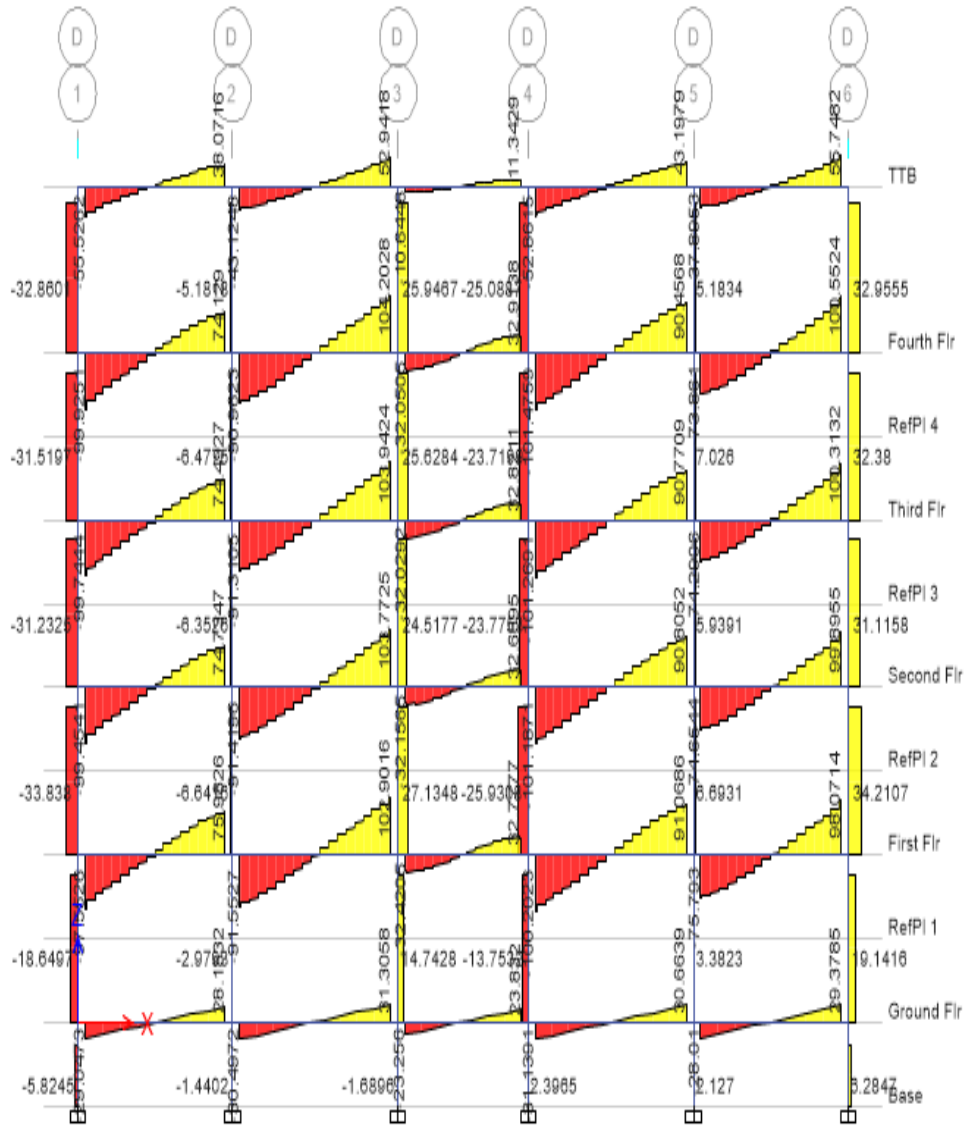


Figure 15 Axis -D shear result

TTB

span	Vmax	VED	Smax	Scal	Smin	SproV	Remark
B1-2	145.4981	43.87246	275.4	123.67486	352.809	123.6749	120
B2-3	145.4981	42.07377	275.4	128.96206	352.809	128.9621	125
B3-4	145.4981	7.227068	275.4	750.77745	352.809	352.809	350
B4-5	145.4981	42.00995	275.4	129.15796	352.809	129.158	125
B5-6	145.4981	44.04786	275.4	123.18236	352.809	123.1824	120

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	76.49271	321.3	82.756126	294.0075	82.75613	80
B2-3	203.6974	82.18192	321.3	77.02716	294.0075	77.02716	75
B3-4	203.6974	21.50568	321.3	294.35197	294.0075	294.0075	290
B4-5	203.6974	78.19798	321.3	80.951454	294.0075	80.95145	80
B5-6	203.6974	78.94475	321.3	80.1857	294.0075	80.1857	80

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	76.35407	321.3	82.906382	294.0075	82.90638	80
B2-3	203.6974	82.10678	321.3	77.097652	294.0075	77.09765	75
B3-4	203.6974	21.38235	321.3	296.04974	294.0075	294.0075	290
B4-5	203.6974	78.17045	321.3	80.979961	294.0075	80.97996	80
B5-6	203.6974	78.75695	321.3	80.376905	294.0075	80.3769	80

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	76.26159	321.3	83.006919	294.0075	83.00692	80
B2-3	203.6974	81.84255	321.3	77.346559	294.0075	77.34656	75
B3-4	203.6974	21.17537	321.3	298.94359	294.0075	294.0075	290
B4-5	203.6974	77.97266	321.3	81.185379	294.0075	81.18538	80
B5-6	203.6974	78.42901	321.3	80.71299	294.0075	80.71299	80

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	203.6974	61.00109	321.3	103.77258	294.0075	103.7726	100
B2-3	203.6974	81.28463	321.3	77.877457	294.0075	77.87746	75
B3-4	203.6974	21.36824	321.3	296.24524	294.0075	294.0075	290
B4-5	203.6974	77.21379	321.3	81.983279	294.0075	81.98328	80
B5-6	203.6974	76.99689	321.3	82.214229	294.0075	82.21423	80

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
B1-2	271.5965	21.9057	367.2	330.2593	252.0064	252.0064	250
B2-3	271.5965	24.94197	367.2	290.05567	252.0064	252.0064	250
B3-4	271.5965	17.61424	367.2	410.72234	252.0064	252.0064	250
B4-5	271.5965	24.37373	367.2	296.81798	252.0064	252.0064	250
B5-6	271.5965	23.05752	367.2	313.76138	252.0064	252.0064	250

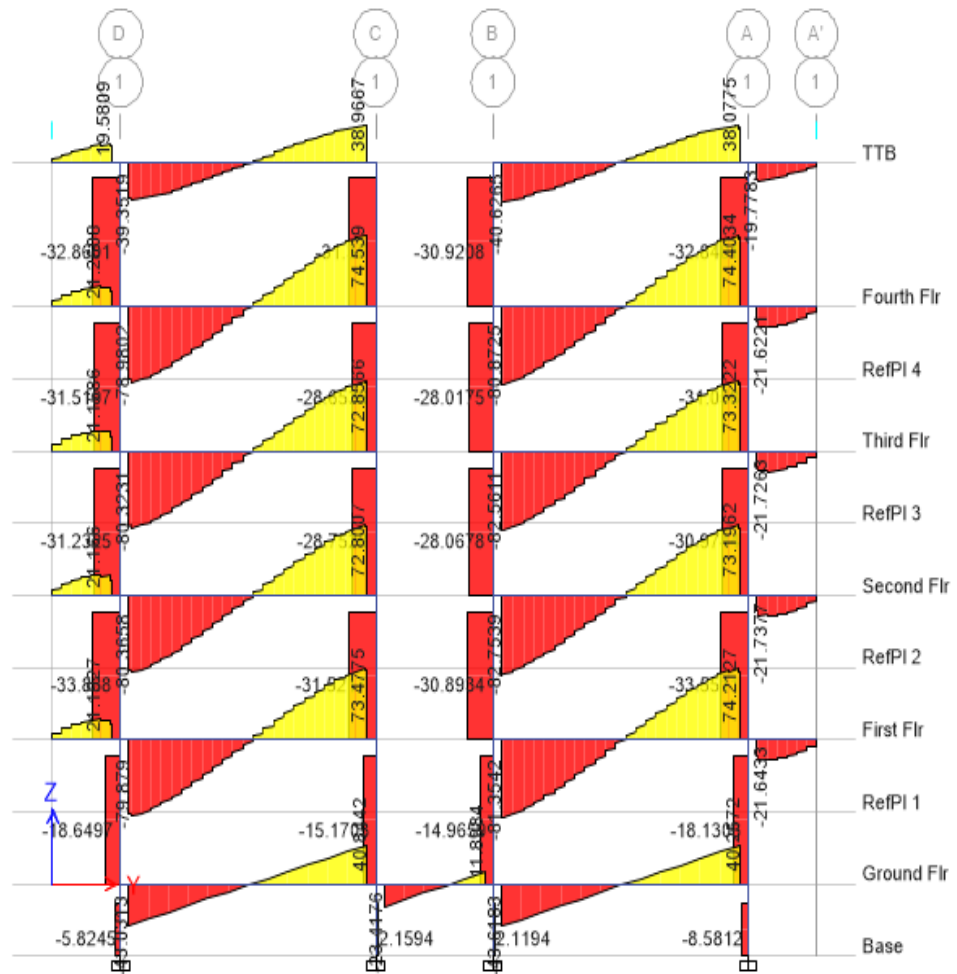


Figure 16 Axis -1 shear result

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	145.4981	32.73319	275.4	165.76204	352.809	165.762	165
1-CD	145.4981	34.63075	275.4	156.67924	352.809	156.6792	150

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	203.6974	64.17139	321.3	98.645833	294.0075	98.64583	130
1-CD	203.6974	68.07379	321.3	92.990858	294.0075	92.99086	130

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	203.6974	65.40252	321.3	96.788932	294.0075	96.78893	130
1-CD	203.6974	69.4253	321.3	91.180586	294.0075	91.18059	130

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	203.6974	65.45117	321.3	96.716984	294.0075	96.71698	130
1-CD	203.6974	69.65744	321.3	90.876722	294.0075	90.87672	130

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	2036.974	198.5573	3213	318.81168	294.0075	294.0075	290
1-CD	203.6974	68.47926	321.3	92.440257	294.0075	92.44026	130

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
1-AB	271.5965	35.41857	367.2	204.25895	252.0064	204.2589	200
1-BC	271.5965	16.22305	367.2	445.94322	252.0064	252.0064	250
1-CD	271.5965	35.98447	367.2	201.04671	252.0064	201.0467	200

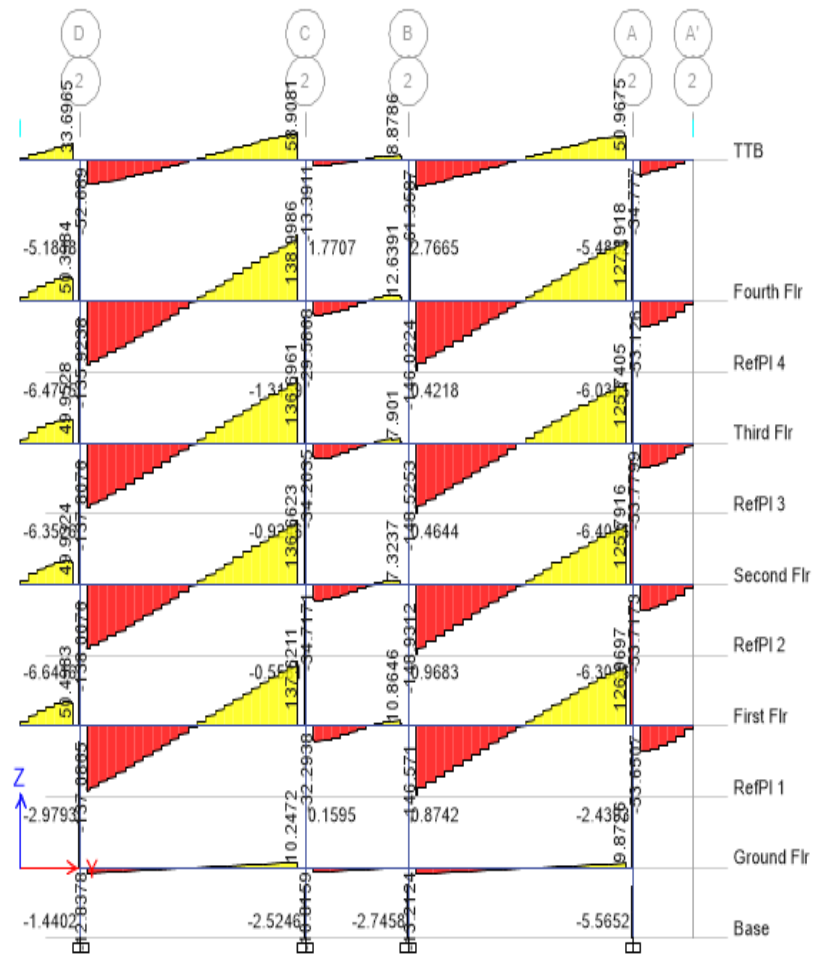


Figure 17 Shear force diagram of beam in axis 2

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

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	145.4981	49.89339	275.4	108.7503	352.809	108.7503	105
2-BC	145.4981	8.385687	275.4	647.0454	352.809	352.809	350
2-CD	145.4981	52.02232	275.4	104.2998	352.809	104.2998	100

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	203.6974	117.3597	321.3	53.93878	294.0075	53.93878	50
2-BC	203.6974	17.66935	321.3	358.2612	294.0075	294.0075	290
2-CD	203.6974	122.9132	321.3	51.50171	294.0075	51.50171	50

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	203.6974	115.9985	321.3	54.57176	294.0075	54.57176	50
2-BC	203.6974	22.38807	321.3	282.7505	294.0075	282.7505	280
2-CD	203.6974	125.02	321.3	50.63382	294.0075	50.63382	50

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	203.6974	116.1668	321.3	54.49267	294.0075	54.49267	50
2-BC	203.6974	22.78421	321.3	277.8346	294.0075	277.8346	270
2-CD	203.6974	125.3617	321.3	50.49582	294.0075	50.49582	50

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	203901	137.7995	321621.3	45983.98	294.0075	294.0075	290
2-BC	203.6974	19.92491	321.3	317.7047	294.0075	294.0075	290
2-CD	203.6974	123.375	321.3	51.30894	294.0075	51.30894	50

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
2-AB	271.5965	10.73708	367.2	673.792	252.0064	252.0064	250
2-BC	271.5965	8.542697	367.2	846.8707	252.0064	252.0064	250
2-CD	271.5965	11.11362	367.2	650.9635	252.0064	252.0064	250

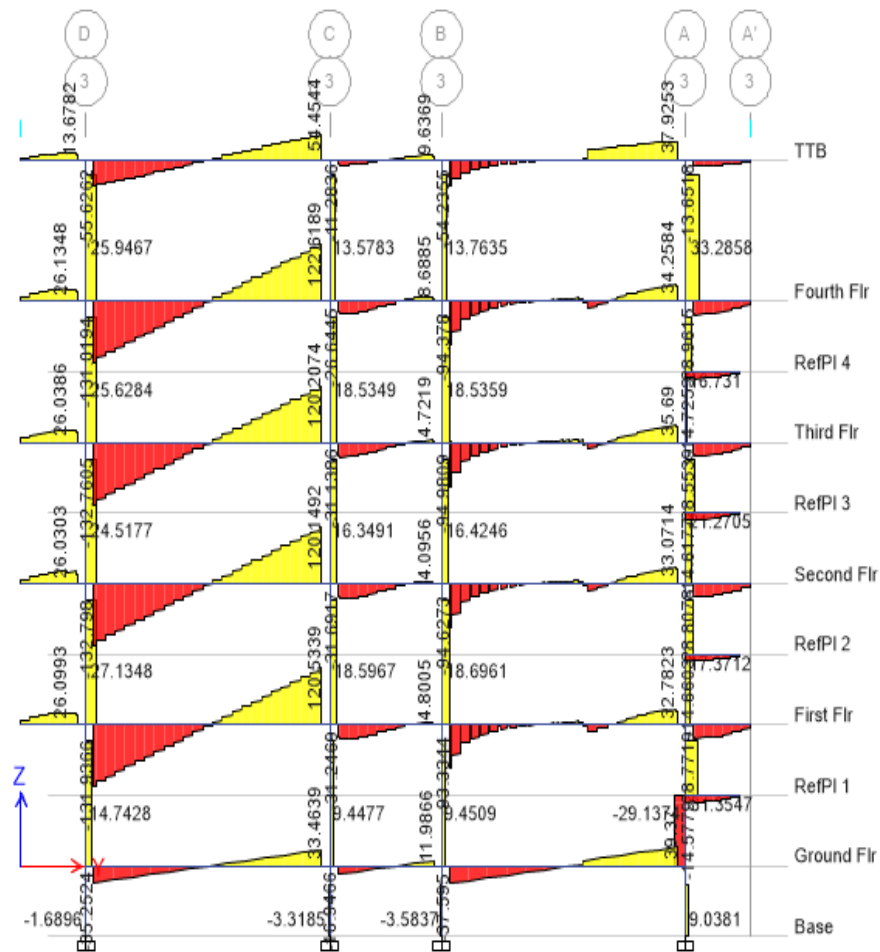


Figure 18 Shear force diagram of beam in axis 3

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

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
3-AB	145.4981	47.70324	275.4	113.7432	352.809	113.7432	110
3-BC	145.4981	6.484536	275.4	836.7476	352.809	352.809	350
3-CD	145.4981	46.55148	275.4	116.5574	352.809	116.5574	110

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
3-AB	203.6974	111.7501	321.3	56.64639	294.0075	56.64639	55
3-BC	203.6974	20.90754	321.3	302.7731	294.0075	294.0075	290
3-CD	203.6974	75.0696	321.3	84.32495	294.0075	84.32495	80

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
3-AB	203.6974	111.7817	321.3	56.63039	294.0075	56.63039	55
3-BC	203.6974	21.64828	321.3	292.4131	294.0075	292.4131	290
3-CD	203.6974	74.79012	321.3	84.64005	294.0075	84.64005	80

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
3-AB	203.6974	111.0566	321.3	57.00013	294.0075	57.00013	55
3-BC	203.6974	21.04664	321.3	300.772	294.0075	294.0075	290
3-CD	203.6974	73.20013	321.3	86.47853	294.0075	86.47853	85

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
3-AB	271.5965	29.04937	367.2	249.0436	252.0064	249.0436	245
3-BC	271.5965	11.13827	367.2	649.5229	252.0064	252.0064	250
3-CD	271.5965	32.78174	367.2	220.6887	252.0064	220.6887	220

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	145.4981	47.65462	275.4	113.8593	352.809	113.8593	110
4-BC	145.4981	7.886554	275.4	687.9963	352.809	352.809	350
4-CD	145.4981	47.02948	275.4	115.3727	352.809	115.3727	110

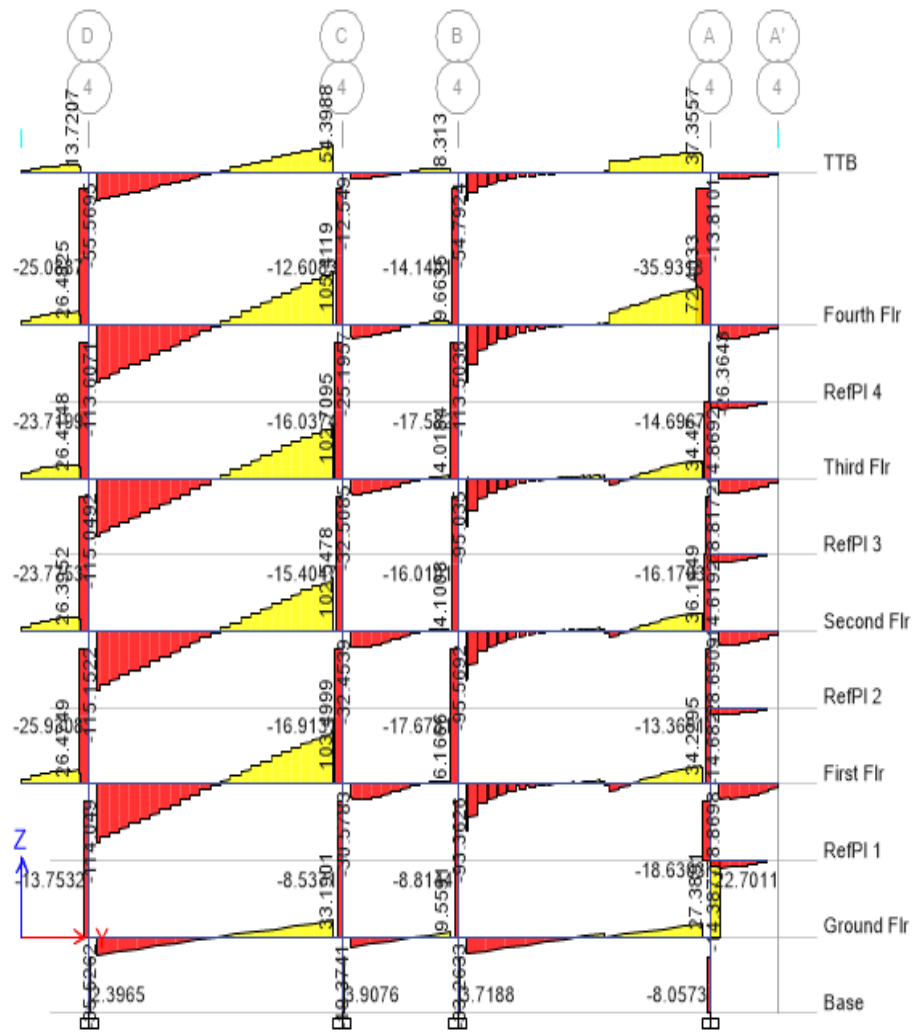


Figure 19 Shear force diagram of beam in axis 4

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

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	145.4981	47.65462	275.4	113.8593	352.809	113.8593	110
4-BC	145.4981	7.886554	275.4	687.9963	352.809	352.809	350
4-CD	145.4981	47.02948	275.4	115.3727	352.809	115.3727	110

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	203901	105.5486	321621.3	60034.65	294.0075	294.0075	290
4-BC	203.6974	15.4256	321.3	410.3723	294.0075	294.0075	290
4-CD	203.6974	93.03275	321.3	68.04314	294.0075	68.04314	65

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	203.6974	96.74442	321.3	65.43261	294.0075	65.43261	65
4-BC	203.6974	22.91745	321.3	276.2192	294.0075	276.2192	270
4-CD	203.6974	75.11236	321.3	84.27695	294.0075	84.27695	80

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	203.6974	96.92845	321.3	65.30838	294.0075	65.30838	65
4-BC	203.6974	22.88465	321.3	276.6151	294.0075	276.6151	270
4-CD	203.6974	75.67553	321.3	83.64976	294.0075	83.64976	80

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	203.6974	95.99985	321.3	65.94011	294.0075	65.94011	65
4-BC	203.6974	20.51415	321.3	308.5792	294.0075	294.0075	290
4-CD	203.6974	73.36718	321.3	86.28163	294.0075	86.28163	85

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
4-AB	271.5965	29.30861	367.2	246.8408	252.0064	246.8408	245
4-BC	271.5965	13.49798	367.2	535.9735	252.0064	252.0064	250
4-CD	271.5965	28.22438	367.2	256.3231	252.0064	252.0064	250

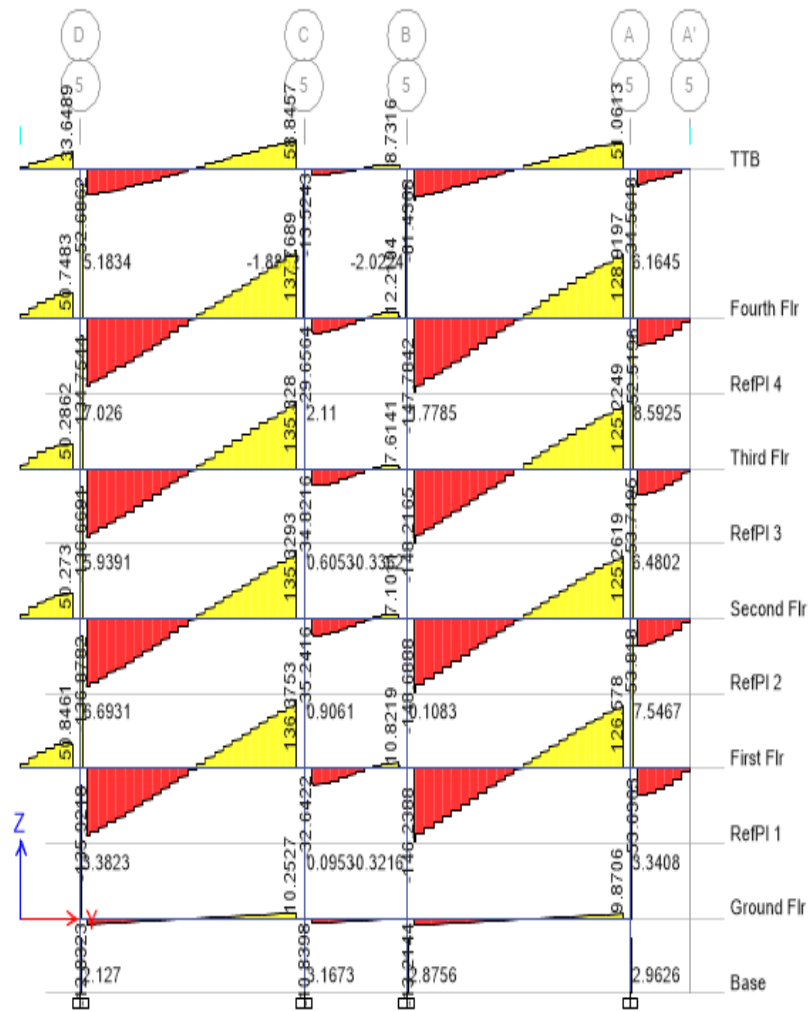


Figure 20 Shear force diagram of beam in axis 5

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

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	145.4981	49.84054	275.4	108.8656	352.809	108.8656	105
5-BC	145.4981	8.505279	275.4	637.9473	352.809	352.809	350
5-CD	145.4981	52.24548	275.4	103.8543	352.809	103.8543	100

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	203.6974	113.4285	321.3	55.80822	294.0075	55.80822	55
5-BC	203.6974	17.80474	321.3	355.5368	294.0075	294.0075	290
5-DC	203.6974	124.2712	321.3	50.93893	294.0075	50.93893	50

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	203.6974	114.9715	321.3	55.05923	294.0075	55.05923	55
5-BC	203.6974	22.96904	321.3	275.5988	294.0075	275.5988	270
5-CD	203.6974	124.7601	321.3	50.73931	294.0075	50.73931	50

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	203.6974	115.1003	321.3	54.99758	294.0075	54.99758	50
5-BC	203.6974	23.29184	321.3	271.7794	294.0075	271.7794	270
5-CD	203.6974	125.1576	321.3	50.57814	294.0075	50.57814	50

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	203.6974	115.1448	321.3	54.97634	294.0075	54.97634	50
5-BC	203.6974	20.31173	321.3	311.6543	294.0075	294.0075	290
5-CD	203.6974	122.9716	321.3	51.47724	294.0075	51.47724	50

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
5-AB	271.5965	10.74309	367.2	673.4151	252.0064	252.0064	250
5-BC	271.5965	8.626939	367.2	838.601	252.0064	252.0064	250
5-CD	271.5965	11.10507	367.2	651.4648	252.0064	252.0064	250

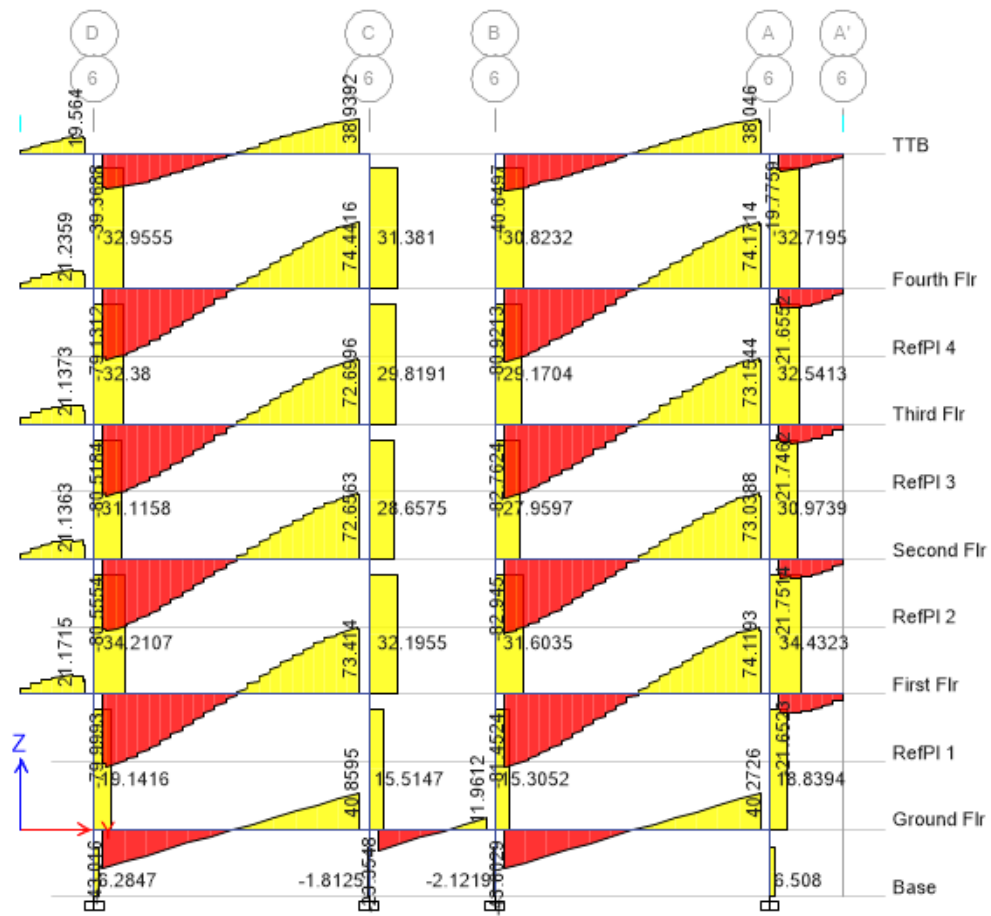


Figure 21 Shear force diagram of beam in axis 6

TTB

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	145.4981	32.73355	275.4	165.7602	352.809	165.7602	160
6-CD	145.4981	34.64851	275.4	156.5989	352.809	156.5989	155

4th Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	203.6974	65.17744	321.3	97.12318	294.0075	97.12318	95
6-CD	203.6974	68.56191	321.3	92.32881	294.0075	92.32881	90

3rd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	203.6974	65.6053	321.3	96.48977	294.0075	96.48977	95
6-CD	203.6974	69.74812	321.3	90.75857	294.0075	90.75857	90

2nd Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	203.6974	65.55698	321.3	96.56089	294.0075	96.56089	95
6-CD	203.6974	69.59458	321.3	90.95881	294.0075	90.95881	90

1st Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	203.6974	65.17744	321.3	97.12318	294.0075	97.12318	95
6-CD	203.6974	68.56191	321.3	92.32881	294.0075	92.32881	90

Ground Floor

span	Vmax	VED	Smax	Scal	Smin	Sprov	Remark
6-AB	271.5965	35.44688	367.2	204.0958	252.0064	204.0958	200
6-BC	271.5965	16.14855	367.2	448.0006	252.0064	252.0064	250
6-CD	271.5965	36.01235	367.2	200.8911	252.0064	200.8911	200

12 Appendix F – (column design)F-1

Table 5 Effective length calculation

Location	Column	bc	hc	bb	hb	Lc	Lbxl	Lbxr	Lbyl	Lbyr	K1x	K2x	K1y	K2y	Lox	Loy
4th floor	C1	400	400	250	300	2800	0	4300	6100	1620	5.82	5.82	1.73	1.73	2699.59	2511.52
4th floor	C2	400	400	250	300	2800	4300	4600	6100	1620	3.01	3.01	1.73	1.73	2617.94	2511.52
4th floor	C3	400	400	250	300	2800	4600	0	6100	1620	6.23	6.23	1.73	1.73	2705.70	2511.52
4th floor	C4	400	400	250	300	2800	0	4600	6100	1620	6.23	6.23	1.73	1.73	2705.70	2511.52
4th floor	C5	400	400	250	300	2800	4600	4300	6100	1620	3.01	3.01	1.73	1.73	2617.94	2511.52
4th floor	C6	400	400	250	300	2800	4300	0	6100	1620	5.82	5.82	1.73	1.73	2699.59	2511.52
4th floor	C7	400	400	250	300	2800	0	4300	2800	6100	5.82	5.82	2.60	2.60	2699.59	2593.40
4th floor	C8	400	400	250	300	2800	4300	4600	2800	6100	3.01	3.01	2.60	2.60	2617.94	2593.40
4th floor	C9	400	400	250	300	2800	4600	3600	2800	6100	2.74	2.74	2.60	2.60	2602.22	2593.40
4th floor	C10	400	400	250	300	2800	3600	4600	2800	6100	2.74	2.74	2.60	2.60	2602.22	2593.40
4th floor	C11	400	400	250	300	2800	4600	4300	2800	6100	3.01	3.01	2.60	2.60	2617.94	2593.40
4th floor	C12	400	400	250	300	2800	4300	0	2800	6100	5.82	5.82	2.60	2.60	2699.59	2593.40
4th floor	C13	400	400	250	300	2800	0	4300	6100	2800	5.82	5.82	2.60	2.60	2699.59	2593.40
4th floor	C14	400	400	250	300	2800	4300	4600	6100	2800	3.01	3.01	2.60	2.60	2617.94	2593.40
4th floor	C15	400	400	250	300	2800	4600	3600	6100	2800	2.74	2.74	2.60	2.60	2602.22	2593.40
4th floor	C16	400	400	250	300	2800	3600	4600	6100	2800	2.74	2.74	2.60	2.60	2602.22	2593.40
4th floor	C17	400	400	250	300	2800	4600	4300	6100	2800	3.01	3.01	2.60	2.60	2617.94	2593.40
4th floor	C18	400	400	250	300	2800	4300	0	6100	2800	5.82	5.82	2.60	2.60	2699.59	2593.40
4th floor	C19	400	400	250	300	2800	0	4300	1620	6100	5.82	5.82	1.73	1.73	2699.59	2511.52
4th floor	C20	400	400	250	300	2800	4300	4600	1620	6100	3.01	3.01	1.73	1.73	2617.94	2511.52
4th floor	C21	400	400	250	300	2800	4600	0	1620	6100	6.23	6.23	1.73	1.73	2705.70	2511.52
4th floor	C22	400	400	250	300	2800	0	4600	1620	6100	6.23	6.23	1.73	1.73	2705.70	2511.52
4th floor	C23	400	400	250	300	2800	4600	4300	1620	6100	3.01	3.01	1.73	1.73	2617.94	2511.52
4th floor	C24	400	400	250	300	2800	4300	0	1620	6100	5.82	5.82	1.73	1.73	2699.59	2511.52

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

Table 6 Slenderness check

	Location	Column	Mx(top)	Mx(bott)	My(top)	My(bott)	Ned	M01X	M02X	M01Y	M02Y	λ_x	λ_y	λ_{limx}	λ_{limy}	Slender
4th floor	C1	15.39	39.04	39.8	48.86	132.22	18.03	41.68	40.16	51.50	31.17	29.00	80.79	58.66	Short	Short
4th floor	C2	18.16	49.48	5.43	9.37	202.3	22.21	53.53	5.87	13.42	30.23	29.00	66.23	65.05	Short	Short
4th floor	C3	22.53	55.37	38.14	51.74	128.57	25.10	57.94	38.64	54.31	31.24	29.00	81.89	63.90	Short	Short
4th floor	C4	24.14	28.86	37.25	59.75	128.08	26.70	31.42	37.78	62.31	31.24	29.00	55.07	70.83	Short	Short
4th floor	C5	18.25	51.4	6.14	10.51	202.08	22.29	55.44	6.59	14.55	30.23	29.00	66.93	64.32	Short	Short
4th floor	C6	15.27	39.45	40.42	47.93	132.49	17.92	42.10	40.78	50.58	31.17	29.00	81.15	56.92	Short	Short
4th floor	C7	42.48	43.67	38.49	44.99	111.91	44.72	45.91	39.38	47.23	31.17	29.95	50.30	60.01	Short	Short
4th floor	C8	54.67	70.11	5.63	1.85	170.82	58.09	73.53	3.01	9.05	30.23	29.95	51.04	76.67	Short	Short
4th floor	C9	48.62	41.98	14.04	23.13	144.46	44.87	51.51	14.94	26.02	30.05	29.95	50.55	68.67	Short	Short
4th floor	C10	49.93	56.11	13.43	24.74	143.08	52.79	58.97	14.49	27.60	30.05	29.95	49.32	72.02	Short	Short
4th floor	C11	54.88	71.13	4.79	0.67	170.71	58.29	74.54	1.84	8.20	30.23	29.95	51.50	82.82	Short	Short
4th floor	C12	42.6	43.09	38.95	44.28	112.13	44.84	45.33	39.85	46.52	31.17	29.95	49.20	58.39	Short	Short
4th floor	C13	36.93	56.97	39.09	45.61	111.17	39.15	59.19	39.87	47.83	31.17	29.95	72.20	60.23	Short	Short
4th floor	C14	43.8	54.57	4.87	0.55	174.47	47.29	58.06	1.50	8.36	30.23	29.95	49.14	84.41	Short	Short
4th floor	C15	41.93	67.91	13.64	23.12	146.15	44.85	70.83	14.54	26.04	30.05	29.95	64.68	69.23	Short	Short
4th floor	C16	40.95	61.42	12.32	21.72	148.49	43.92	64.39	13.20	24.69	30.05	29.95	61.23	70.11	Short	Short
4th floor	C17	43.59	54.27	4.38	0.71	174.48	47.08	57.76	1.65	7.87	30.23	29.95	49.11	82.69	Short	Short
4th floor	C18	36.82	57.51	39.48	45.25	111.35	39.05	59.74	40.26	47.48	31.17	29.95	72.69	59.18	Short	Short
4th floor	C19	20.59	24.87	40.07	48.65	133.67	23.26	27.54	40.54	51.32	31.17	29.00	54.23	57.71	Short	Short
4th floor	C20	24.05	37.95	4.96	9.02	203.32	28.12	42.02	5.52	13.09	30.23	29.00	52.99	65.70	Short	Short
4th floor	C21	40.83	52.46	31.78	38.28	151.91	43.87	55.50	32.66	41.32	31.24	29.00	54.09	54.10	Short	Short
4th floor	C22	41.29	42.95	30.72	37.02	152.58	44.34	46.00	31.61	40.07	31.24	29.00	43.68	54.08	Short	Short
4th floor	C23	24.19	36.89	5.35	8.65	203.15	28.25	40.95	5.92	12.71	30.23	29.00	51.95	63.50	Short	Short
4th floor	C24	20.7	24.38	40.42	48.56	133.87	23.38	27.06	40.89	51.24	31.17	29.00	52.96	57.14	Short	Short

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

Table 7 Fourth Reinforcement

	Location	Column	MEDx	MEDy	Vsd	(N)sdx	(N)sdy	ω	AS	As,min	As,max	Asprov	No of bar	Long. Reinf
4th floor	C1	41.68	51.50	0.06	0.05	0.06	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C2	53.53	13.42	0.09	0.06	0.01	0.01	65.14	320	6400	320.00	1.02	4Ø20	Ø8 c/c 400
4th floor	C6	42.10	50.58	0.06	0.05	0.06	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C7	45.91	47.23	0.05	0.05	0.05	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
4th floor	C8	73.53	9.05	0.08	0.08	0.01	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C9	51.51	26.02	0.06	0.06	0.03	0.12	781.62	320	6400	781.62	2.49	4Ø20	Ø8 c/c 400
4th floor	C10	58.97	27.60	0.06	0.07	0.03	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
4th floor	C11	74.54	8.20	0.08	0.08	0.01	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C12	45.33	46.52	0.05	0.05	0.05	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
4th floor	C13	59.19	47.83	0.05	0.07	0.05	0.17	1107.30	320	6400	1107.30	3.53	4Ø20	Ø8 c/c 400
4th floor	C14	58.06	8.36	0.08	0.06	0.01	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
4th floor	C15	70.83	26.04	0.06	0.08	0.03	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C16	64.39	24.69	0.07	0.07	0.03	0.18	1172.43	320	6400	1172.43	3.73	4Ø20	Ø8 c/c 400
4th floor	C17	57.76	7.87	0.08	0.06	0.01	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
4th floor	C18	59.74	47.48	0.05	0.07	0.05	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
4th floor	C19	27.54	51.32	0.06	0.03	0.06	0.14	911.89	320	6400	911.89	2.90	4Ø20	Ø8 c/c 400
4th floor	C20	42.02	13.09	0.09	0.05	0.01	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
4th floor	C21	55.50	41.32	0.07	0.06	0.05	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
4th floor	C22	46.00	40.07	0.07	0.05	0.04	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
4th floor	C23	40.95	12.71	0.09	0.05	0.01	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
4th floor	C24	27.06	51.24	0.06	0.03	0.06	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400

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

Table 8 Effective length calculation

Location	Column	bc	hc	bb	hb	Lc	Lbxl	Lbxr	Lbyl	Lbyr	K1x	K2x	K1y	K2y	Lox	Loy
3rd Floor	C1	400	400	300	350	2800	0	4300	6100	1620	3.06	3.06	0.91	0.91	2620.33	2336.72
3rd Floor	C2	400	400	300	350	2800	4300	4600	6100	1620	1.58	1.58	0.91	0.91	2489.62	2336.72
3rd Floor	C3	400	400	300	350	2800	4600	0	6100	1620	3.27	3.27	0.91	0.91	2630.63	2336.72
3rd Floor	C4	400	400	300	350	2800	0	4600	6100	1620	3.27	3.27	0.91	0.91	2630.63	2336.72
3rd Floor	C5	400	400	300	350	2800	4600	4300	6100	1620	1.58	1.58	0.91	0.91	2489.62	2336.72
3rd Floor	C6	400	400	300	350	2800	4300	0	6100	1620	3.06	3.06	0.91	0.91	2620.33	2336.72
3rd Floor	C7	400	400	300	350	2800	0	4300	2800	6100	3.06	3.06	1.36	1.36	2620.33	2452.73
3rd Floor	C8	400	400	300	350	2800	4300	4600	2800	6100	1.58	1.58	1.36	1.36	2489.62	2452.73
3rd Floor	C9	400	400	300	350	2800	4600	3600	2800	6100	1.44	1.44	1.36	1.36	2465.87	2452.73
3rd Floor	C10	400	400	300	350	2800	3600	4600	2800	6100	1.44	1.44	1.36	1.36	2465.87	2452.73
3rd Floor	C11	400	400	300	350	2800	4600	4300	2800	6100	1.58	1.58	1.36	1.36	2489.62	2452.73
3rd Floor	C12	400	400	300	350	2800	4300	0	2800	6100	3.06	3.06	1.36	1.36	2620.33	2452.73
3rd Floor	C13	400	400	300	350	2800	0	4300	6100	2800	3.06	3.06	1.36	1.36	2620.33	2452.73
3rd Floor	C14	400	400	300	350	2800	4300	4600	6100	2800	1.58	1.58	1.36	1.36	2489.62	2452.73
3rd Floor	C15	400	400	300	350	2800	4600	3600	6100	2800	1.44	1.44	1.36	1.36	2465.87	2452.73
3rd Floor	C16	400	400	300	350	2800	3600	4600	6100	2800	1.44	1.44	1.36	1.36	2465.87	2452.73
3rd Floor	C17	400	400	300	350	2800	4600	4300	6100	2800	1.58	1.58	1.36	1.36	2489.62	2452.73
3rd Floor	C18	400	400	300	350	2800	4300	0	6100	2800	3.06	3.06	1.36	1.36	2620.33	2452.73
3rd Floor	C19	400	400	300	350	2800	0	4300	1620	6100	3.06	3.06	0.91	0.91	2620.33	2336.72
3rd Floor	C20	400	400	300	350	2800	4300	4600	1620	6100	1.58	1.58	0.91	0.91	2489.62	2336.72
3rd Floor	C21	400	400	300	350	2800	4600	0	1620	6100	3.27	3.27	0.91	0.91	2630.63	2336.72
3rd Floor	C22	400	400	300	350	2800	0	4600	1620	6100	3.27	3.27	0.91	0.91	2630.63	2336.72
3rd Floor	C23	400	400	300	350	2800	4600	4300	1620	6100	1.58	1.58	0.91	0.91	2489.62	2336.72
3rd Floor	C24	400	400	300	350	2800	4300	0	1620	6100	3.06	3.06	0.91	0.91	2620.33	2336.72

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

Table 9 Slenderness check

Location	Column	Mx(top)	Mx(bott)	My(top)	My(bott)	Ned	M01X	M02X	M01Y	M02Y	λ_x	λ_y	λ_{limx}	λ_{limy}	Slender	Slender
3rd Floor	C1	17.01	31.42	35.43	46.85	346.34	23.94	38.35	35.91	53.78	25.93	23.13	42.37	40.66	Short	Short
3rd Floor	C2	28.26	44.42	6.97	8.98	602.3	40.31	56.47	7.78	21.03	24.64	23.13	29.46	39.73	Short	Short
3rd Floor	C3	98.62	83.54	45.26	45.63	546.65	94.47	109.55	47.15	56.56	26.04	23.13	26.26	27.16	Short	Short
3rd Floor	C4	72.21	58.19	45.34	51.77	460.64	67.40	81.42	46.69	60.98	26.04	23.13	29.79	31.91	Short	Short
3rd Floor	C5	29.19	44.36	10.71	12.11	604.23	41.27	56.44	11.54	24.19	24.64	23.13	28.89	36.48	Short	Short
3rd Floor	C6	16.21	30.86	37.21	49.06	346.94	23.15	37.80	37.67	56.00	25.93	23.13	42.80	40.43	Short	Short
3rd Floor	C7	51.9	56.32	31.76	42.47	303.37	57.97	62.39	32.92	48.54	25.93	24.28	32.44	43.00	Short	Short
3rd Floor	C8	79.45	91.9	0.024	1.11	136.39	82.18	94.63	1.67	3.84	24.64	24.28	52.19	79.43	Short	Short
3rd Floor	C9	46.9	52.73	21.91	27.24	399.57	54.89	60.72	23.01	35.23	24.41	24.28	29.19	38.39	Short	Short
3rd Floor	C10	59.8	57.68	21.34	25.23	426.11	66.20	68.32	22.66	33.75	24.41	24.28	25.96	36.52	Short	Short
3rd Floor	C11	81.04	92.6	3.02	1.61	538.48	91.81	103.37	3.45	13.79	24.64	24.28	25.64	45.81	Short	Short
3rd Floor	C12	52.56	56.88	33.07	44.25	303.95	58.64	62.96	34.24	50.33	25.93	24.28	32.32	42.87	Short	Short
3rd Floor	C13	26.16	45.49	32.44	43.48	299.11	32.14	51.47	33.08	49.46	25.93	24.28	45.59	43.70	Short	Short
3rd Floor	C14	43.05	63.63	2.13	1.33	558.22	54.21	74.79	2.41	13.29	24.64	24.28	30.25	47.11	Short	Short
3rd Floor	C15	35.43	54.85	21.98	27.17	465.06	44.73	64.15	22.87	36.47	24.41	24.28	34.08	36.47	Short	Short
3rd Floor	C16	26.99	44.17	18.95	23.13	442.05	35.83	53.01	19.67	31.97	24.41	24.28	35.70	37.82	Short	Short
3rd Floor	C17	41.84	62.18	2.96	2.66	555.83	52.96	73.30	3.72	14.08	24.64	24.28	30.39	44.64	Short	Short
3rd Floor	C18	25.49	44.98	33.81	45.24	299.96	31.49	50.98	34.44	51.24	25.93	24.28	45.81	43.50	Short	Short
3rd Floor	C19	41.73	40.82	36.05	47.48	352.66	47.87	48.78	37.01	54.53	25.93	23.13	28.05	39.87	Short	Short
3rd Floor	C20	56.7	58.51	7.74	9.41	609.16	68.88	70.69	9.12	21.59	24.64	23.13	21.55	37.95	Slender	Short
3rd Floor	C21	62.21	67.38	29.29	38.65	477.09	71.75	76.92	30.73	48.19	26.04	23.13	25.75	35.65	Slender	Short
3rd Floor	C22	52.56	56.62	27.12	35.73	455.6	61.67	65.73	28.35	44.84	26.04	23.13	26.16	36.67	Short	Short
3rd Floor	C23	56.13	57.75	8.27	10.37	607.38	68.28	69.90	9.64	22.52	24.64	23.13	21.51	37.84	Slender	Short
3rd Floor	C24	42.49	41.42	36.95	48.87	353.56	48.49	49.56	37.92	55.94	25.93	23.13	28.13	39.85	Short	Short

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

Table 10 3rd floor Column Reinforcement

Location	Column	MEDx	MEDy	Vsd	(N)sdx	(N)sdy	ω	AS	As,min	As,max	Asprov	No of bar	long. Rein	Shear Rein
3rd Floor	C1	38.35	53.78	0.15	0.04	0.06	0.1	651.35	320	6400	651.35	2.07	5Ø20	Ø8 c/c 400
3rd Floor	C2	56.47	21.03	0.27	0.06	0.02	0	0.00	320	6400	320.00	1.02	4Ø20	Ø8 c/c 400
3rd Floor	C3	109.55	56.56	0.24	0.12	0.06	0.5	3256.76	320	6400	3256.76	10.37	5Ø20	Ø8 c/c 400
3rd Floor	C4	81.42	60.98	0.20	0.09	0.07	0.2	1302.71	320	6400	1302.71	4.15	FALSE	Ø8 c/c 400
3rd Floor	C5	56.44	24.19	0.27	0.06	0.03	0	0.00	320	6400	320.00	1.02	4Ø20	Ø8 c/c 400
3rd Floor	C6	37.80	56.00	0.15	0.04	0.06	0.1	651.35	320	6400	651.35	2.07	5Ø20	Ø8 c/c 400
3rd Floor	C7	62.39	48.54	0.13	0.07	0.05	0.5	3256.76	320	6400	3256.76	10.37	FALSE	Ø8 c/c 400
3rd Floor	C8	94.63	3.84	0.06	0.10	0.00	0.05	325.68	320	6400	325.68	1.04	5Ø20	Ø8 c/c 400
3rd Floor	C9	60.72	35.23	0.18	0.07	0.04	0.02	130.27	320	6400	320.00	1.02	4Ø20	Ø8 c/c 400
3rd Floor	C10	68.32	33.75	0.19	0.08	0.04	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
3rd Floor	C11	103.37	13.79	0.24	0.11	0.02	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
3rd Floor	C12	62.96	50.33	0.13	0.07	0.06	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
3rd Floor	C13	51.47	49.46	0.13	0.06	0.05	0.17	1107.30	320	6400	1107.30	3.53	4Ø20	Ø8 c/c 400
3rd Floor	C14	74.79	13.29	0.25	0.08	0.01	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
3rd Floor	C15	64.15	36.47	0.21	0.07	0.04	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
3rd Floor	C16	53.01	31.97	0.20	0.06	0.04	0.18	1172.43	320	6400	1172.43	3.73	4Ø20	Ø8 c/c 400
3rd Floor	C17	73.30	14.08	0.25	0.08	0.02	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
3rd Floor	C18	50.98	51.24	0.13	0.06	0.06	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
3rd Floor	C19	48.78	54.53	0.16	0.05	0.06	0.14	911.89	320	6400	911.89	2.90	4Ø20	Ø8 c/c 400
3rd Floor	C20	70.69	21.59	0.27	0.08	0.02	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
3rd Floor	C21	76.92	48.19	0.21	0.08	0.05	0.2	1302.71	320	6400	1302.71	4.15	5Ø20	Ø8 c/c 400
3rd Floor	C22	65.73	44.84	0.20	0.07	0.05	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400
3rd Floor	C23	69.90	22.52	0.27	0.08	0.02	0.1	651.35	320	6400	651.35	2.07	4Ø20	Ø8 c/c 400
3rd Floor	C24	49.56	55.94	0.16	0.05	0.06	0.15	977.03	320	6400	977.03	3.11	4Ø20	Ø8 c/c 400

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

Table 11 2nd floor Effective length calculation

	Location	Column	bc	hc	bb	hb	Lc	Lbxl	Lbxr	Lbyl	Lbyr	K1x	K2x	K1y	K2y	Lox	Loy
	2nd floor	C1	500	500	300	350	2800	0	4300	6100	1620	7.46	7.46	2.22	2.22	2720.38	2564.17
	2nd floor	C2	500	500	300	350	2800	4300	4600	6100	1620	3.86	3.86	2.22	2.22	2653.72	2564.17
	2nd floor	C3	500	500	300	350	2800	4600	0	6100	1620	7.98	7.98	2.22	2.22	2725.29	2564.17
	2nd floor	C4	500	500	300	350	2800	0	4600	6100	1620	7.98	7.98	2.22	2.22	2725.29	2564.17
	2nd floor	C5	500	500	300	350	2800	4600	4300	6100	1620	3.86	3.86	2.22	2.22	2653.72	2564.17
	2nd floor	C6	500	500	300	350	2800	4300	0	6100	1620	7.46	7.46	2.22	2.22	2720.38	2564.17
	2nd floor	C7	500	500	300	350	2800	0	4300	2800	6100	7.46	7.46	3.33	3.33	2720.38	2633.35
	2nd floor	C8	500	500	300	350	2800	4300	4600	2800	6100	3.86	3.86	3.33	3.33	2653.72	2633.35
	2nd floor	C9	500	500	300	350	2800	4600	3600	2800	6100	3.50	3.50	3.33	3.33	2640.69	2633.35
	2nd floor	C10	500	500	300	350	2800	3600	4600	2800	6100	3.50	3.50	3.33	3.33	2640.69	2633.35
	2nd floor	C11	500	500	300	350	2800	4600	4300	2800	6100	3.86	3.86	3.33	3.33	2653.72	2633.35
	2nd floor	C12	500	500	300	350	2800	4300	0	2800	6100	7.46	7.46	3.33	3.33	2720.38	2633.35
	2nd floor	C13	500	500	300	350	2800	0	4300	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
	2nd floor	C14	500	500	300	350	2800	4300	4600	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
	2nd floor	C15	500	500	300	350	2800	4600	3600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
	2nd floor	C16	500	500	300	350	2800	3600	4600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
	2nd floor	C17	500	500	300	350	2800	4600	4300	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
	2nd floor	C18	500	500	300	350	2800	4300	0	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
	2nd floor	C19	500	500	300	350	2800	0	4300	1620	6100	7.46	7.46	2.22	2.22	2720.38	2564.17
	2nd floor	C20	500	500	300	350	2800	4300	4600	1620	6100	3.86	3.86	2.22	2.22	2653.72	2564.17
	2nd floor	C21	500	500	300	350	2800	4600	0	1620	6100	7.98	7.98	2.22	2.22	2725.29	2564.17
	2nd floor	C22	500	500	300	350	2800	0	4600	1620	6100	7.98	7.98	2.22	2.22	2725.29	2564.17
	2nd floor	C23	500	500	300	350	2800	4600	4300	1620	6100	3.86	3.86	2.22	2.22	2653.72	2564.17
	2nd floor	C24	500	500	300	350	2800	4300	0	1620	6100	7.46	7.46	2.22	2.22	2720.38	2564.17

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

Table 12 2nd floor Slenderness check

Location	Column	Mx(top)	Mx(bott)	My(top)	My(bott)	Ned	M01X	M02X	M01Y	M02Y	λ_x	λ_y	λ_{limx}	λ_{limy}	Slender	Slender
2nd floor	C1	21.08	28.22	36.45	45.62	558.9	32.26	39.40	37.10	56.80	26.92	25.38	34.15	40.57	Short	Short
2nd floor	C2	29.83	39.1	6.97	9.67	1002.19	49.87	59.14	7.97	29.71	26.27	25.38	24.80	41.44	Slender	Short
2nd floor	C3	74.97	60.62	45.93	44.91	933.73	79.29	93.64	46.50	64.60	26.97	25.38	25.58	29.39	Slender	Short
2nd floor	C4	65.36	61.92	44.39	49.9	935.89	80.64	84.08	46.00	68.62	26.97	25.38	22.19	30.84	Slender	Short
2nd floor	C5	29.31	38.72	8.41	11.66	1002.85	49.37	58.78	9.40	31.72	26.27	25.38	24.89	40.61	Slender	Short
2nd floor	C6	21.15	28.21	36.21	45.91	560.29	32.36	39.42	36.86	57.12	26.92	25.38	34.03	40.83	Short	Short
2nd floor	C7	44.35	55.97	33.17	41.2	495.29	54.26	65.88	34.26	51.11	26.92	26.06	36.08	42.39	Short	Short
2nd floor	C8	71.17	90.41	0.54	0.72	898.14	89.13	108.37	2.32	18.68	26.27	26.06	26.83	48.17	Short	Short
2nd floor	C9	43.51	55.26	18.99	24.18	649.86	56.51	68.26	20.12	37.18	26.14	26.06	31.35	41.65	Short	Short
2nd floor	C10	41.21	54.72	18.57	23.84	675	54.71	68.22	19.66	37.34	26.14	26.06	31.67	41.38	Short	Short
2nd floor	C11	70.87	90.35	0.68	0.17	899.2	88.85	108.33	1.95	18.66	26.27	26.06	26.88	48.76	Short	Short
2nd floor	C12	44.36	56.09	32.84	41.28	496.69	54.29	66.02	33.93	51.21	26.92	26.06	36.08	42.66	Short	Short
2nd floor	C13	32.55	41.94	33.95	42.22	485.69	42.26	51.65	34.80	51.93	26.92	26.06	36.66	42.82	Short	Short
2nd floor	C14	44.02	56.59	1.07	1.34	946.35	62.95	75.52	2.33	20.27	26.27	26.06	25.81	47.21	Slender	Short
2nd floor	C15	37.92	48.39	18.89	24.47	787.67	53.67	64.14	19.96	40.22	26.14	26.06	28.18	39.30	Short	Short
2nd floor	C16	30.31	39.16	17.98	22.83	743.13	45.17	54.02	18.88	37.69	26.14	26.06	29.03	40.30	Short	Short
2nd floor	C17	43.16	55.52	0.53	1.09	942.5	62.01	74.37	1.77	19.94	26.27	26.06	25.85	48.09	Slender	Short
2nd floor	C18	32.61	41.95	33.64	42.33	487.14	42.35	51.69	34.49	52.07	26.92	26.06	36.56	43.08	Short	Short
2nd floor	C19	31.59	40.31	36.7	46.07	572.77	43.05	51.77	37.56	57.53	26.92	25.38	33.25	40.09	Short	Short
2nd floor	C20	43.63	55.31	7.48	9.34	1017.07	63.97	75.65	8.76	29.68	26.27	25.38	24.55	40.36	Slender	Short
2nd floor	C21	41.54	65.85	28.6	36.42	803.48	57.61	81.92	29.75	52.49	26.97	25.38	32.22	36.63	Short	Short
2nd floor	C22	44.12	55.7	27.91	35.11	759.57	59.31	70.89	29.10	50.30	26.97	25.38	28.70	37.29	Short	Short
2nd floor	C23	42.89	54.36	6.83	8.94	1013.73	63.16	74.63	8.09	29.21	26.27	25.38	24.57	40.95	Slender	Short
2nd floor	C24	31.59	40.38	36.36	46.13	574.36	43.08	51.87	37.22	57.62	26.92	25.38	33.24	40.30	Short	Short

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

Table 13 2nd floor Column Reinforcement

Location	Column	MEDx	MEDy	Vsd	(N)sdx	(N)sdy	ω	AS	As,min	As,max	Asprov	No of bar	Long. Rein	Shear Rein
2nd floor	C1	39.40	56.80	0.16	0.02	0.03	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C2	59.14	29.71	0.28	0.03	0.02	0.01	1017.77	500	10000	500.00	1.59	4Ø20	Ø8 c/c 400
2nd floor	C3	93.64	64.60	0.26	0.05	0.04	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C4	84.08	68.62	0.26	0.05	0.04	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C5	58.78	31.72	0.28	0.03	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C6	39.42	57.12	0.16	0.02	0.03	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C7	65.88	51.11	0.14	0.04	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
2nd floor	C8	108.37	18.68	0.25	0.06	0.01	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C9	68.26	37.18	0.18	0.04	0.02	0.12	1221.29	500	10000	1221.29	3.89	4Ø20	Ø8 c/c 400
2nd floor	C10	68.22	37.34	0.19	0.04	0.02	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
2nd floor	C11	108.33	18.66	0.25	0.06	0.01	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C12	66.02	51.21	0.14	0.04	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
2nd floor	C13	51.65	51.93	0.14	0.03	0.03	0.17	1730.16	500	10000	1730.16	5.51	6Ø20	Ø8 c/c 400
2nd floor	C14	75.52	20.27	0.27	0.04	0.01	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C15	64.14	40.22	0.22	0.04	0.02	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C16	54.02	37.69	0.21	0.03	0.02	0.18	1831.93	500	10000	1831.93	5.83	6Ø20	Ø8 c/c 400
2nd floor	C17	74.37	19.94	0.27	0.04	0.01	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C18	51.69	52.07	0.14	0.03	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
2nd floor	C19	51.77	57.53	0.16	0.03	0.03	0.14	1424.83	500	10000	1424.83	4.54	5Ø20	Ø8 c/c 400
2nd floor	C20	75.65	29.68	0.29	0.04	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C21	81.92	52.49	0.23	0.05	0.03	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
2nd floor	C22	70.89	50.30	0.21	0.04	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
2nd floor	C23	74.63	29.21	0.29	0.04	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
2nd floor	C24	51.87	57.62	0.16	0.03	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400

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

Table 14 1st floor Effective length calculation

Location	Column	bc	hc	bb	hb	Lc	Lbxl	Lbxr	Lbyl	Lbyr	K1x	K2x	K1y	K2y	Lox	Loy
1st Floor	C1A	500	600	300	350	2800	0	4300	6100	1620	12.89	12.89	3.84	3.84	2752.79	2653.10
1st Floor	C2A	500	600	300	350	2800	4300	4600	6100	1620	6.66	6.66	3.84	3.84	2711.45	2653.10
1st Floor	C3A	500	600	300	350	2800	4600	0	6100	1620	13.79	13.79	3.84	3.84	2755.77	2653.10
1st Floor	C4A	500	600	300	350	2800	0	4600	6100	1620	13.79	13.79	3.84	3.84	2755.77	2653.10
1st Floor	C5A	500	600	300	350	2800	4600	4300	6100	1620	6.66	6.66	3.84	3.84	2711.45	2653.10
1st Floor	C6A	500	600	300	350	2800	4300	0	6100	1620	12.89	12.89	3.84	3.84	2752.79	2653.10
1st Floor	C1B	500	600	300	350	2800	0	4300	2800	6100	12.89	12.89	5.75	5.75	2752.79	2698.47
1st Floor	C2B	500	600	300	350	2800	4300	4600	2800	6100	6.66	6.66	5.75	5.75	2711.45	2698.47
1st Floor	C3B	500	600	300	350	2800	4600	3600	2800	6100	6.06	6.06	5.75	5.75	2703.17	2698.47
1st Floor	C4B	500	600	300	350	2800	3600	4600	2800	6100	6.06	6.06	5.75	5.75	2703.17	2698.47
1st Floor	C5D	500	600	300	350	2800	4600	4300	2800	6100	6.66	6.66	5.75	5.75	2711.45	2698.47
1st Floor	C6B	500	600	300	350	2800	4300	0	2800	6100	12.89	12.89	5.75	5.75	2752.79	2698.47
1st Floor	C1C	500	500	300	350	2800	0	4300	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
1st Floor	C2C	500	500	300	350	2800	4300	4600	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
1st Floor	C3C	500	500	300	350	2800	4600	3600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
1st Floor	C4C	500	500	300	350	2800	3600	4600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
1st Floor	C5C	500	500	300	350	2800	4600	4300	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
1st Floor	C6C	500	500	300	350	2800	4300	0	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
1st Floor	C1D	500	600	300	350	2800	0	4300	1620	6100	12.89	12.89	3.84	3.84	2752.79	2653.10
1st Floor	C2D	500	600	300	350	2800	4300	4600	1620	6100	6.66	6.66	3.84	3.84	2711.45	2653.10
1st Floor	C3D	500	600	300	350	2800	4600	0	1620	6100	13.79	13.79	3.84	3.84	2755.77	2653.10
1st Floor	C4D	500	600	300	350	2800	0	4600	1620	6100	13.79	13.79	3.84	3.84	2755.77	2653.10
1st Floor	C5D	500	600	300	350	2800	4600	4300	1620	6100	6.66	6.66	3.84	3.84	2711.45	2653.10
1st Floor	C6D	500	600	300	350	2800	4300	0	1620	6100	12.89	12.89	3.84	3.84	2752.79	2653.10

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

Table 15 1st floor Slenderness check

Location	Column	Mx(top)	Mx(bott)	My(top)	My(bott)	Ned	M01X	M02X	M01Y	M02Y	λ_x	λ_y	λ_{limx}	λ_{limy}	Slender	Slender
1st Floor	C1	23.88	28.1	36.37	52.53	771.09	39.30	43.52	37.16	67.95	27.25	26.26	28.81	41.68	Short	Short
1st Floor	C2	34.1	43.52	6.97	9.67	1402.32	62.15	71.57	8.21	37.72	26.84	26.26	22.29	39.73	Slender	Short
1st Floor	C3	82.75	78.39	45.93	44.91	1329.73	104.98	109.34	47.01	72.52	27.28	26.26	20.37	28.95	Slender	Short
1st Floor	C4	68.59	44.17	44.39	49.9	1316.23	70.49	94.91	45.80	76.22	27.28	26.26	26.48	30.41	Slender	Short
1st Floor	C5	33.79	42.64	8.4	11.67	1402.08	61.83	70.68	9.64	39.71	26.84	26.26	22.12	39.06	Slender	Short
1st Floor	C6	23.59	25.14	37.32	53.98	773.08	39.05	40.60	38.10	69.44	27.25	26.26	26.65	41.56	Slender	Short
1st Floor	C7	43.59	68.75	33.1	48.79	687.07	57.33	82.49	34.25	62.53	27.25	26.71	38.48	44.13	Short	Short
1st Floor	C8	71.93	109.74	0.95	1.66	1259.83	97.13	134.94	2.89	26.86	26.84	26.71	27.72	45.03	Short	Short
1st Floor	C9	41.15	61.41	20.34	29.26	898.47	59.12	79.38	21.52	47.23	26.75	26.71	31.99	41.67	Short	Short
1st Floor	C10	43.06	65.44	19.23	27.59	924.69	61.55	83.93	20.46	46.08	26.75	26.71	31.91	41.46	Short	Short
1st Floor	C11	72.15	110.26	0.29	0.04	1260.11	97.35	135.46	1.99	25.49	26.84	26.71	27.75	45.86	Short	Short
1st Floor	C12	43.9	69.79	33.87	49.92	689.11	57.68	83.57	35.02	63.70	27.25	26.71	38.61	43.98	Short	Short
1st Floor	C13	35.33	44.31	33.83	49.69	671.92	48.77	57.75	34.81	63.13	26.92	26.06	30.24	40.60	Short	Short
1st Floor	C14	48.73	66.05	0.72	0.58	1335.53	75.44	92.76	2.09	27.43	26.27	26.06	22.23	40.71	Slender	Short
1st Floor	C15	42.13	57.16	20.26	29.08	1111	64.35	79.38	21.55	51.30	26.14	26.06	24.45	35.19	Slender	Short
1st Floor	C16	34.02	43.81	18.33	26.47	1143.72	56.89	66.68	19.47	49.34	26.14	26.06	22.94	35.37	Slender	Short
1st Floor	C17	47.33	64.31	1.15	1.29	1330.15	73.93	90.91	2.63	27.89	26.27	26.06	22.28	40.34	Slender	Short
1st Floor	C18	35.07	43.38	34.61	50.75	673.86	48.55	56.86	35.58	64.23	26.92	26.06	29.87	40.45	Short	Short
1st Floor	C19	31.07	50.26	36.72	52.96	792.64	46.92	66.11	37.66	68.81	27.25	26.26	35.30	41.10	Short	Short
1st Floor	C20	44.82	73.16	7.22	10.35	1425.55	73.33	101.67	8.69	38.86	26.84	26.26	26.02	39.25	Slender	Short
1st Floor	C21	51.54	79.13	29.59	42.37	1129.86	74.14	101.73	31.07	64.97	27.28	26.26	29.00	36.48	Short	Short
1st Floor	C22	43.7	69.39	28.09	40.63	1063.35	64.97	90.66	29.39	61.90	27.28	26.26	30.27	37.71	Short	Short
1st Floor	C23	44.19	72.58	7.43	10.35	1420.73	72.60	100.99	8.88	38.76	26.84	26.26	26.13	39.17	Slender	Short
1st Floor	C24	31.41	51.29	37.29	53.41	794.79	47.31	67.19	38.24	69.31	27.25	26.26	35.46	40.88	Short	Short

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

Table 16 1st floor Column Reinforcement

Location	Column	MEDx	MEDy	Vsd	(N)sdx	(N)sdy	ω	AS	As,min	As,max	Asprov	No of bar	Long. Rein	Shear Rein
1st Floor	C1	43.52	67.95	0.18	0.02	0.03	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C2	71.57	37.72	0.33	0.03	0.01	0.01	122.13	600	12000	600.00	1.91	4Ø20	Ø8 c/c 400
1st Floor	C3	109.34	72.52	0.31	0.04	0.03	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C4	94.91	76.22	0.31	0.04	0.03	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
1st Floor	C5	70.68	39.71	0.33	0.03	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
1st Floor	C6	40.60	69.44	0.18	0.02	0.03	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C7	82.49	62.53	0.16	0.03	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
1st Floor	C8	134.94	26.86	0.30	0.05	0.01	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C9	79.38	47.23	0.21	0.03	0.02	0.12	1465.54	600	12000	1465.54	4.67	5Ø20	Ø8 c/c 400
1st Floor	C10	83.93	46.08	0.22	0.03	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
1st Floor	C11	135.46	25.49	0.30	0.05	0.01	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C12	83.57	63.70	0.16	0.03	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
1st Floor	C13	57.75	63.13	0.19	0.03	0.04	0.17	1730.16	500	10000	1730.16	5.51	6Ø20	Ø8 c/c 400
1st Floor	C14	92.76	27.43	0.38	0.05	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
1st Floor	C15	79.38	51.30	0.31	0.04	0.03	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
1st Floor	C16	66.68	49.34	0.32	0.04	0.03	0.18	1831.93	500	10000	1831.93	5.83	6Ø20	Ø8 c/c 400
1st Floor	C17	90.91	27.89	0.38	0.05	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
1st Floor	C18	56.86	64.23	0.19	0.03	0.04	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
1st Floor	C19	66.11	68.81	0.19	0.03	0.03	0.14	1709.80	600	12000	1709.80	5.45	6Ø20	Ø8 c/c 400
1st Floor	C20	101.67	38.86	0.34	0.04	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
1st Floor	C21	101.73	64.97	0.27	0.04	0.03	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
1st Floor	C22	90.66	61.90	0.25	0.04	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
1st Floor	C23	100.99	38.76	0.33	0.04	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
1st Floor	C24	67.19	69.31	0.19	0.03	0.03	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400

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

Table 17 Ground floor Effective length calculation

Location	Column	bc	hc	bb	hb	Lc	Lbxl	Lbxr	Lbyl	Lbyr	K1x	K2x	K1y	K2y	Lox	Loy
G+0	C1	500	600	300	350	2800	0	4300	6100	1620	12.89	12.89	3.84	3.84	2752.79	2653.10
G+0	C2	500	600	300	350	2800	4300	4600	6100	1620	6.66	6.66	3.84	3.84	2711.45	2653.10
G+0	C3	500	600	300	350	2800	4600	0	6100	1620	13.79	13.79	3.84	3.84	2755.77	2653.10
G+0	C4	500	600	300	350	2800	0	4600	6100	1620	13.79	13.79	3.84	3.84	2755.77	2653.10
G+0	C5	500	600	300	350	2800	4600	4300	6100	1620	6.66	6.66	3.84	3.84	2711.45	2653.10
G+0	C6	500	600	300	350	2800	4300	0	6100	1620	12.89	12.89	3.84	3.84	2752.79	2653.10
G+0	C7	500	600	300	350	2800	0	4300	2800	6100	12.89	12.89	5.75	5.75	2752.79	2698.47
G+0	C8	500	600	300	350	2800	4300	4600	2800	6100	6.66	6.66	5.75	5.75	2711.45	2698.47
G+0	C9	500	600	300	350	2800	4600	3600	2800	6100	6.06	6.06	5.75	5.75	2703.17	2698.47
G+0	C10	500	600	300	350	2800	3600	4600	2800	6100	6.06	6.06	5.75	5.75	2703.17	2698.47
G+0	C11	500	600	300	350	2800	4600	4300	2800	6100	6.66	6.66	5.75	5.75	2711.45	2698.47
G+0	C12	500	600	300	350	2800	4300	0	2800	6100	12.89	12.89	5.75	5.75	2752.79	2698.47
G+0	C13	500	500	300	350	2800	0	4300	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
G+0	C14	500	500	300	350	2800	4300	4600	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
G+0	C15	500	500	300	350	2800	4600	3600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
G+0	C16	500	500	300	350	2800	3600	4600	6100	2800	3.50	3.50	3.33	3.33	2640.69	2633.35
G+0	C17	500	500	300	350	2800	4600	4300	6100	2800	3.86	3.86	3.33	3.33	2653.72	2633.35
G+0	C18	500	500	300	350	2800	4300	0	6100	2800	7.46	7.46	3.33	3.33	2720.38	2633.35
G+0	C19	500	600	300	350	2800	0	4300	1620	6100	12.89	12.89	3.84	3.84	2752.79	2653.10
G+0	C20	500	600	300	350	2800	4300	4600	1620	6100	6.66	6.66	3.84	3.84	2711.45	2653.10
G+0	C21	500	600	300	350	2800	4600	0	1620	6100	13.79	13.79	3.84	3.84	2755.77	2653.10
G+0	C22	500	600	300	350	2800	0	4600	1620	6100	13.79	13.79	3.84	3.84	2755.77	2653.10
G+0	C23	500	600	300	350	2800	4600	4300	1620	6100	6.66	6.66	3.84	3.84	2711.45	2653.10
G+0	C24	500	600	300	350	2800	4300	0	1620	6100	12.89	12.89	3.84	3.84	2752.79	2653.10

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

Table 18 Ground floor Slenderness check

Location	Column	Mx(top)	Mx(bott)	My(top)	My(bott)	Ned	M01X	M02X	M01Y	M02Y	λ_x	λ_y	λ_{limx}	λ_{limy}	Slender	Slender
G+0	C1	29.83	12.12	31.31	16.75	982.78	31.78	49.49	17.39	50.97	27.25	26.26	33.87	43.51	Short	Short
G+0	C2	36.89	4.32	5.31	1.1	1805.09	40.42	72.99	1.91	41.41	26.84	26.26	27.08	39.07	Short	Short
G+0	C3	108.36	40.09	39.51	12.13	1718.71	74.46	142.73	13.62	73.88	27.28	26.26	28.53	36.70	Short	Short
G+0	C4	8.21	6.16	34.28	2.86	1620.81	38.58	40.63	3.63	66.70	27.28	26.26	18.71	41.03	Slender	Short
G+0	C5	37.19	4.49	5.42	3.48	1804.11	40.57	73.27	4.29	41.50	26.84	26.26	27.09	37.73	Short	Short
G+0	C6	30.46	12.08	31.08	18.91	985.31	31.79	50.17	19.55	50.79	27.25	26.26	34.10	42.05	Short	Short
G+0	C7	32.25	39.8	28.07	11.66	876.81	49.79	57.34	12.66	45.61	27.25	26.71	28.19	48.22	Short	Short
G+0	C8	52.22	40.27	1	1.37	1625.83	72.79	84.74	2.46	33.89	26.84	26.71	20.94	40.51	Slender	Short
G+0	C9	32.75	47.98	16.51	8.62	1146.02	55.67	70.90	9.73	39.43	26.75	26.71	27.12	43.09	Short	Short
G+0	C10	29.38	38.05	16.32	7.04	1174.2	52.86	61.53	8.10	39.80	26.75	26.71	24.63	43.84	Slender	Short
G+0	C11	51.67	40.11	0.9	0.12	1625.45	72.62	84.18	1.57	33.41	26.84	26.71	20.85	41.15	Slender	Short
G+0	C12	61.63	39.83	27.84	12.77	879.42	57.42	79.22	13.92	45.43	27.25	26.71	33.01	47.17	Short	Short
G+0	C13	38.56	9.95	28.39	11.823	857.08	27.09	55.70	12.36	45.53	26.92	26.06	37.98	44.71	Short	Short
G+0	C14	49.95	8.04	0.21	0.65	1724.11	42.52	84.43	1.06	35.13	26.27	26.06	26.40	36.85	Short	Short
G+0	C15	40.12	1.85	16.54	8.59	1433.96	30.53	68.80	9.20	45.22	26.14	26.06	30.40	36.21	Short	Short
G+0	C16	37.54	5.78	15.68	6.95	1342.22	32.62	64.38	7.60	42.52	26.14	26.06	29.84	38.05	Short	Short
G+0	C17	49.76	7.89	0.18	0.34	1717.06	42.23	84.10	1.02	34.68	26.27	26.06	26.49	36.94	Short	Short
G+0	C18	39.22	9.96	28.32	12.85	859.54	27.15	56.41	13.39	45.51	26.92	26.06	38.09	43.93	Short	Short
G+0	C19	21.98	36.26	31.36	18.08	1010.33	42.19	56.47	18.92	51.57	27.25	26.26	30.09	42.09	Short	Short
G+0	C20	30.38	28.21	5.62	2.25	1835.49	64.92	67.09	3.55	42.33	26.84	26.26	17.16	37.86	Slender	Short
G+0	C21	41.02	47.72	25.09	14.07	1455.04	70.12	76.82	15.47	54.19	27.28	26.26	20.71	37.22	Slender	Short
G+0	C22	32.25	39.22	24.01	12.47	1665.42	65.56	72.53	13.78	57.32	27.28	26.26	19.58	35.90	Slender	Short
G+0	C23	29.88	27.87	5.72	3.29	1829.18	64.45	66.46	4.58	42.30	26.84	26.26	17.14	37.36	Slender	Short
G+0	C24	21.38	36.31	31.66	19.13	1013.06	41.64	56.57	19.96	51.92	27.25	26.26	30.40	41.49	Short	Short

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

Table 19 Ground floor Column Reinforcement

Location	Column	MEDx	MEDy	Vsd	(N)sdx	(N)sdy	ω	AS	As,min	As,max	Asprov	No of bar	long. Rein	Shear Rein
G+0	C1	49.49	50.97	0.23	0.02	0.02	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C2	72.99	41.41	0.42	0.03	0.02	0.01	1221.13	600	12000	600.00	1.91	4Ø20	Ø8 c/c 400
G+0	C3	142.73	73.88	0.40	0.06	0.03	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C4	40.63	66.70	0.38	0.02	0.03	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
G+0	C5	73.27	41.50	0.42	0.03	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
G+0	C6	50.17	50.79	0.23	0.02	0.02	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C7	57.34	45.61	0.21	0.02	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
G+0	C8	84.74	33.89	0.38	0.03	0.01	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C9	70.90	39.43	0.27	0.03	0.02	0.12	1465.54	600	12000	1465.54	4.67	5Ø20	Ø8 c/c 400
G+0	C10	61.53	39.80	0.28	0.02	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
G+0	C11	84.18	33.41	0.38	0.03	0.01	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C12	79.22	45.43	0.21	0.03	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
G+0	C13	55.70	45.53	0.24	0.03	0.03	0.17	1730.16	500	10000	1730.16	5.51	6Ø20	Ø8 c/c 400
G+0	C14	84.43	35.13	0.49	0.05	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
G+0	C15	68.80	45.22	0.41	0.04	0.03	0.2	2035.48	500	10000	2035.48	6.48	7Ø20	Ø8 c/c 400
G+0	C16	64.38	42.52	0.38	0.04	0.02	0.18	1831.93	500	10000	1831.93	5.83	6Ø20	Ø8 c/c 400
G+0	C17	84.10	34.68	0.49	0.05	0.02	0.1	1017.74	500	10000	1017.74	3.24	4Ø20	Ø8 c/c 400
G+0	C18	56.41	45.51	0.24	0.03	0.03	0.15	1526.61	500	10000	1526.61	4.86	5Ø20	Ø8 c/c 400
G+0	C19	56.47	51.57	0.24	0.02	0.02	0.14	1709.80	600	12000	1709.80	5.45	6Ø20	Ø8 c/c 400
G+0	C20	67.09	42.33	0.43	0.03	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
G+0	C21	76.82	54.19	0.34	0.03	0.02	0.2	2442.57	600	12000	2442.57	7.78	8Ø20	Ø8 c/c 400
G+0	C22	72.53	57.32	0.39	0.03	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400
G+0	C23	66.46	42.30	0.43	0.03	0.02	0.1	1221.29	600	12000	1221.29	3.89	4Ø20	Ø8 c/c 400
G+0	C24	56.57	51.92	0.24	0.02	0.02	0.15	1831.93	600	12000	1831.93	5.83	6Ø20	Ø8 c/c 400

