Civil Engineering Department

Wolkite University, College of Engineering and Technology

Structural Design of B+G+4 Mixed-Use Building in Addis Ababa

A B.Sc. thesis/project (CENG5281) submitted as a partial fulfilment of the requirements for *the Degree of Bachelor Science in Civil Engineering*

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As members of the examining board of the final B.Sc. open defence, we verify that we have read and evaluated the final BSc thesis/project prepared by Ribka Desalegn, Habte Digafe, Alazar Asfaw, Robel Tesfalem, Desta Fikadu and Tewabech Demissie entitled Structural Design of B+G+4 Mixed use Building and recommended for acceptance as a fulfilment of the requirement of B.Sc. in Civil Engineering

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Internal Examiner	Signature	Date

Summary

Engineering has traditionally been taught by specialists as a series of separate courses, and it has been assumed that graduates will be able to design the knowledge and understanding gained from these project as required to undertake real world design projects.

A final year course at Wolkite university of Civil Engineering, thesis that design different types of building. It has been developed various fundamental courses such as reinforcement concrete, structural design and foundation engineering to name a few. This project is a structural design of a G+4 mixed use building. The proposed building is located in Addis Ababa city. This report mainly focuses on the project of a G+4 mixed use (such shop, cafeteria and residential on this project) design to be this building safe and economical.

On this project the analysis and designing set ups on chapter, each chapters contains objective, general introduction, design procedures, code provisions or requirements and design solution. Design of the project is well interpreted in this document able to perform design of roof, design of slab, design of frame structure and design of foundation on its own chapter. The design philosophy adopted for the project is the limit state design for all aspects or parts of the structure according to Ethiopian Building Code of Standards in European Norms (ES EN).

Finally this document has discussion and conclusion by the person who did this project, also reference and appendix.

Acknowledgement

Our greatest thank from the depth of heart is to God for giving us courage, strength and mainly health throughout school time and full help provided by him for successful accomplishment of this final project for the B.Sc. Degree in civil engineering.

Our deepest gratitude and appreciation goes to our project advisors, Ms. Betelhem Asmare & Mr. Tibebu Tesfaye, for they valuable advice and endless technical assistance which brought this project approach in the successful completion.

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In general, we want to pass grateful acknowledgement for those we have helped us in the process either in advice or in material until the project comes to completion.

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List of Abbreviations

RC: Reinforced Concrete

Υ: Unit weight

ϕ: Diameter of reinforcement bars

f_{yk}: The characteristic strength of reinforcement steel in tension and compression

f_{yd}: The design yield strength of reinforcement steel in tension and compression

f_{cu}: Cubic compressive strength of concrete

f_{ck}: Characteristic cylindrical compressive strength of concrete

f_{cd}: Design compressive strength of concrete in compression

f_{ctd}: Design compressive strength of concrete in tension

Υ_s , Υ_c: Partial safety factors for concrete and steel, respectively

ε_{cu}: Maximum compressive strain on the concrete

E: Modulus of elasticity

HCB: Hollow concrete block

d: Effective depth for the center of reinforcement bars

A_c: Gross area of concrete section

A_s: Area of tensile reinforcement bar

${A_s}'$: Area of compressive reinforcement bar

λ: Slenderness ratio

r: Radius of gyration

X: Neutral axis depth

I: Second moment of inertia

P_o: Ultimate axial load capacity of column

N_{sd}: design values of internal axial load

M_u: Ultimate flexural capacity of the section

M_{bal}: Balanced moment capacity of column

M_o: First order moment

e_o, e_a, e_2 : First order, accidental and second order eccentricity, respectively

L_{eff}: Effective length

b, h: Dimensions of rectangular section (width, depth) respectively

$\frac{1}{r_{bal}}$: Curvature at the balanced load

ACI: American Concrete Institution

EBCS: Ethiopian Building Code Standards

ES EN: Ethiopian Standard based on Euro Norms

E_c: Modulus of elasticity of concrete

E_s: Modulus of elasticity of steel

EI: Flexural stiffness

M_1: The algebraically bigger of M_{top} and M_{bottom}

M_2: The algebraically smaller of M_{top} and M_{bottom}

μ_{sd}: Relative moment ($\frac{M_{sd}}{f_{cd}A_c h}$)

ω: Mechanical reinforcement ratio

c_d: Dynamic coefficient

c_{ALT}: Altitude factor

c_e: Exposure factor

c_e: Roughness coefficient

c_{TEM}: Temporary factor

v_{ref}: Reference wind velocity factor

z_o: Roughness length

z_o: Minimum height

e: External, exposure

i: Internal

ref: Reference

θ: Angle

$\emptyset$: Upwind slope, diameter of bar

γ: Both unit weight

γ: Partial safety factor (safety or serviceability)

i.e. this means

Introduction

1.1. General

Building structures are solids, which are composed of architectural and structural parts. The structural part of the building supports the body of the building preventing it from any collapse or failure. Therefore, structural design involves the determination of the different sections of the skeletal part of the building to make it stable and sustainable throughout its design life.

A structural design is executed in such a way that the building will remain fit with appropriate degrees of reliability and in an economic way. It should sustain all the actions and influences during execution and use. Therefore, structural design focuses on structural safety and serviceability with due durability. It must also optimize the cost expended in building the structure and maintenance.

This structural design is executed based on the Ethiopian Standard based on Euro Norms (ES EN 2015). This code follows the Limit State design approach. Limit state is a state beyond which the structure no longer satisfies the design performance requirements.

1.2. Background

General information about the project

- This project is B+G+4 Mixed used building,
- The building allocated on Addis Ababa,
- The height of the building is +18.60 m.
- Type of roof is flat roof with marble chip floor finish.
- The total area of the building 500 m^2.

1.3. Objectives

The main objective of the project is to carry out a structural design and analysis of a B+G+4 mixed use building. The prime objective of design is structural safety and serviceability. In case the structure fails, it must be in such a way it will minimize risks and loss.

The other objective of thesis is to enable students to experience real life engineering problem solving, design, team work, project execution and management. To satisfy program requirements, the projects must have certain components such as problem definition, research, scheduling, solution analysis, design and communication of results.

1.4. Specific objective

It needs specific tasks to accomplish the structural design and analysis of our building.

- Roof design
- Floor slab design
- Building circulation design its stair case design
- Lateral load analysis its wind load and earthquake load
- Frame analysis using SAP & frame element design
- Foundation design

1.5. Building types

There are many types of buildings which can be classified based on their purpose:

Residential buildings that provide the facility for private houses, apartments, Educational building: buildings which function for learning process. Institutional buildings which are built for the purpose of health, recoveries etc. Assembly buildings which are used for accommodation of many peoples together egg. Recreation, cinema halls, conference halls. Business building used for a business purpose like banks, private libraries etc. Industrial building are used for fabricating or manufacturing purposes like brick industry, cement factory, steel industry etc. Storage building: buildings which are used for storage purposes such as ware house. Hazard building are used for storing dangerous materials or explosive materials such as gases, acids, explosives etc. This project used for mixed use building for business and residential.

To design a given structure first analyze the structure.

Analysis of structure:-It is the analysis of a given structure by modelling of the loads and the structural frames to obtain internal forces (i.e. axial, shear, torsional, or stresses), deflections, and verifies that no unstable failure can occur.

Structural design:-Structural design can be defined as a mixture of art and science, combining the engineer's feeling for the behavior of a structure with a sound knowledge of the principles of statics, dynamics, mechanics of material, and structural analysis, to design safe, economical and durable structure that will serve its intended purpose. In other word structural design involves proportioning the dimension of the member, internal reinforcement (number and sizes of reinforcing bars) of RC structures and selecting an appropriate section for those of wood and steel structure elements to withstand an imposed load over it.

Once the structural form has been determined, the actual design begins with those elements that are subjected to the primary loads the structure is intended to carry, and proceeds in sequence to the various supporting members until it reaches to the foundation. The analysis and design of a building is starts at the building roof then sequentially it ends at the foundation. Therefore, design of a building generally involves the design of the following elements of the building according to their design steps;

- Design of roof
- Design of floor slabs
- Design of stair case
- Design of beams, columns and shear wall
- Design of footing

To analyze and design a structure, it is necessary to establish criteria for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. Safety, stability, strength, serviceability, economy and aesthetics.

1.6. Design criterion for building

The severe conditions which can be expected to occur in the life time of the building include

- Determined situation, which refer to the conditions of normal use;
- Transient situations, which refer to temporary conditions applicable to the structural, e.g. during execution or repair;
- Seismic situations, which refer to exceptional conditions applicable to the structure when it is subjected to seismic event;
- Accidental situation, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact

These and other considerations are included in this document.(ES EN 1990, 2015)

Methodology

Overall framework of this project is illustrated in Figure 1. The architectural drawing was received from the Civil Engineering Department B.Sc. Thesis/Project Coordinator. Next, the structural design of the building will be conducted using Ethiopian building code 2015. This included analysis and design of roof, slab, staircase, beams, columns and footing.

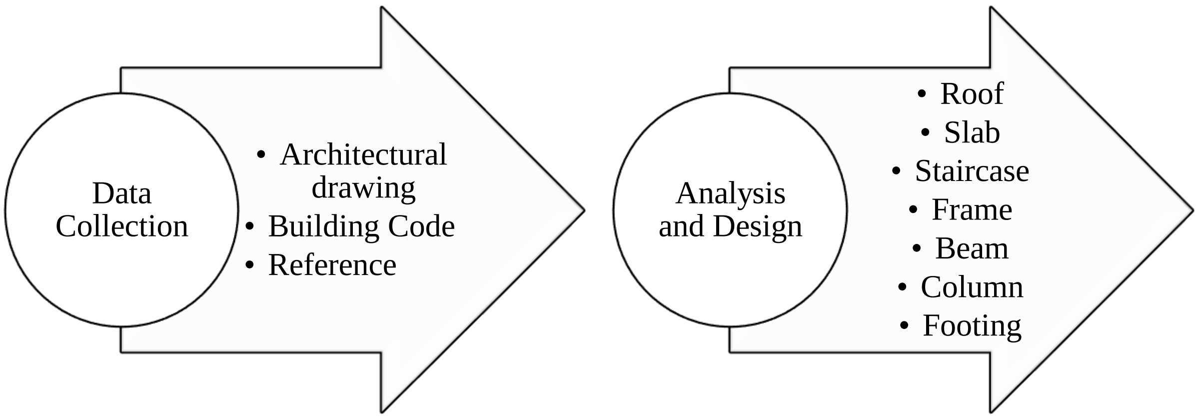


Figure 2. 1- The overall framework of this Project

2.1. Study Area

The project allocated on Addis Ababa, the effect of study area on structural analysis I simply identified. The earthquake is zone 2 on the new code describe on (ES EN 8, 2015). The building is near to main asphalt road it effect on the material selection and consideration.

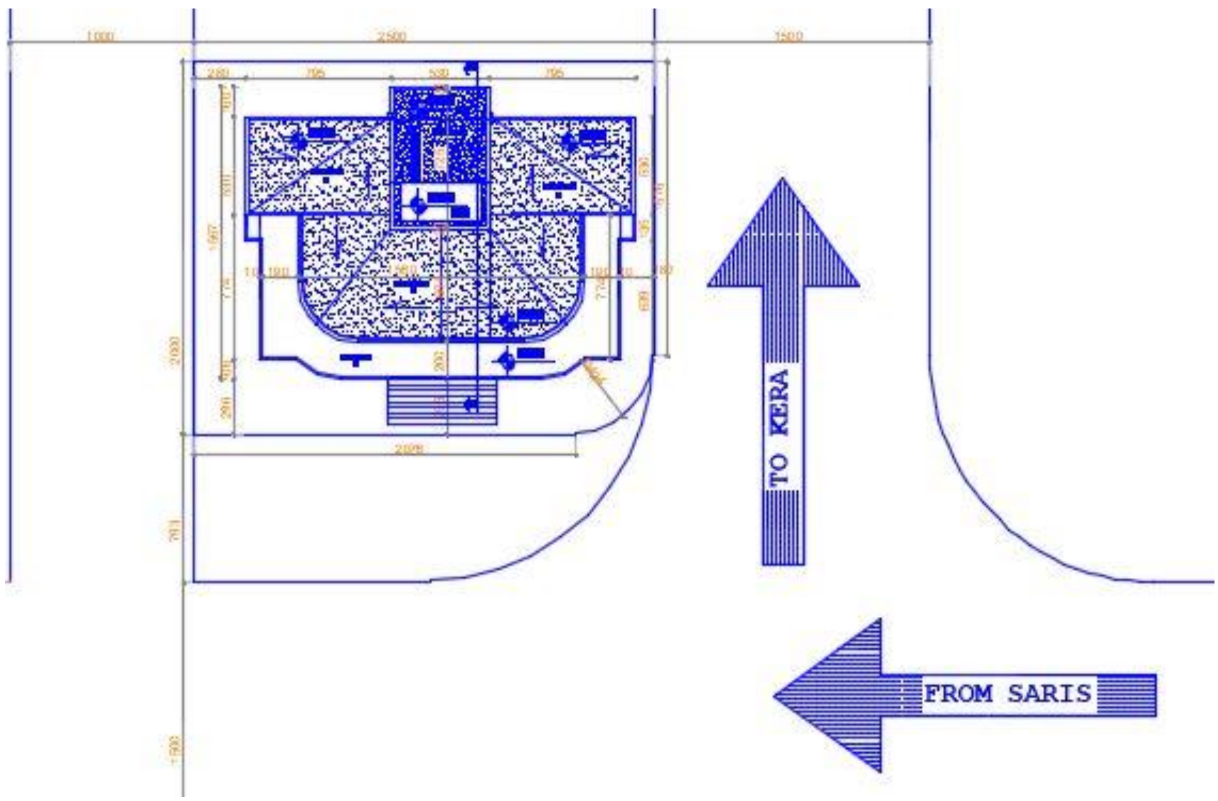


Figure 2. 2- The study area of the Project

2.2. Data Collections

The architectural drawing was received from the Civil Engineering Department B.Sc. thesis/project Coordinator. The building used to carry out the structural design is limit state and building codes used on EN 1992 (Euro code 2, EC2) deals with the design of reinforced concrete structures –buildings, bridges and other civil engineering works.

The ES EN codes serve at least four distinct functions;

- Ensure adequate structural safety, by specifying certain essential minimum requirements for design.
- Render the task of the designer relatively simple; often, the results of sophisticated analyses are made available in the form of a simple formula or chart.
- Ensure a measure of consistency among different designers.
- Have some legal validity, in that they protect the structural designer from any liability due to structural failures that are caused by inadequate supervision and/or faulty material and construction

Table-2.1 EN ES determination

EN No	Eurocode	The structural Eurocodes
EN 1990	ES EN 0	Basis of structural design
EN 1991	ES EN 1	Actions on structures
	Part 1-1	General actions – Densities, self-weight and imposed loads
	Part 1-2	General actions on structures exposed to fire
	Part 1-3	General actions – Snow loads
	Part 1-4	General actions – Wind loads
	Part 1-5	General actions – Thermal actions
	Part 1-6	Actions during execution
	Part 1-7	Accidental actions from impact and explosions
	Part 2	Traffic loads on bridges
	Part 3	Actions induced by cranes and machinery
	Part 4	Actions in silos and tanks
EN 1992	ES EN 2	Design of concrete structures
	Part 1-1	General rules and rules for buildings
	Part 1-2	General rules – structural fire design
	Part 2	Reinforced and pre stresses concrete bridges
	Part 3	Liquid retaining and containing structures
EN 1993	ES EN 3	Design of steel structures
EN 1994	ES EN 4	Design of composite steel and concrete structures
EN 1995	ES EN 5	Design of timber structures
EN 1996	ES EN 6	Design of masonry structures
EN 1997	ES EN 7	Geotechnical design
EN 1998	ES EN 8	Design of structures for earthquake resistance
EN 1999	ES EN 9	Design of aluminum alloy structures
	Part 1-1	Densities, self-weight, impose loaded for buildings
	Part 1-2	Actions on structures exposed to fire
	Part 1-4	Wind actions
	Part 4	Soils and tanks

2.3. Material Specification

The first step in design is to select construction materials which are capable of resisting the applied load. Considering the availability in market we select concrete and steel reinforcement as follows.

Concrete

- Class I workmanship and ordinary loading condition is used
- Ordinary Reinforced concrete class C-25/30, which is C-25(super-structure) and C-30(sub-structure).
- Unit weight of concrete, ϒc = 25KN/m^3
- Partial safety factor for concrete γc= 1.5
- Characteristics of compressive strength of concrete,

  fck =0.8*25 =20Mpa

$fcd = 0.85 * \frac{20}{1.5} = 11.33Mpa$

Where: fck - Characteristic of compressive strength of concrete.

γc- Partial Safety factor for ordinary loading=1.5.

$fctd = \frac{fctk}{\gamma c}$

Where: fctd - Design of tensile strength of concrete.

fctk -Characteristic tensile strength of concrete.

γc- Partial Safety factor for ordinary loading=1.5.

Steel

- S-400 deformed bars
- Partial safety factor γs=1.15
- Characteristic strength, fyk=400N/mm2
- Design strength,

$$fyd = \frac{fyk}{\gamma s} = \frac{400}{1.15} = 347.83Mpa$$

Where: fyk- Grade of steel, S-400

γs - Reinforcing steel factor =1.15 Mpa for ordinary loading and class I

- Secant modulus of elasticity Ecm (GPa)=200 Gpa
- Modulus of elasticity (Es)= 200Gpa

Safety factors for load

- 1.35 for dead load
- 1.50 for live load

2.4. Geotechnical data

For a reason, a geotechnical report could not be found, the allowable bearing capacity of the site will be based on recommendations of EBCS 7 and geotechnical report of a site next to our proposed project. The bearing capacity is 300kpa.(ES EN 1997, Part 1, 2015)

2.5. Data Analysis and Design

The overall method to analyze and design a structure, it is necessary to establish criteria's or requirements for determining whether a given structure is acceptable for use in a specified circumstance or for use directly as a design objective that must be met. The overall method depends up on:-

- Safety
- Durability,
- Economy, and
- Aesthetics appearance.

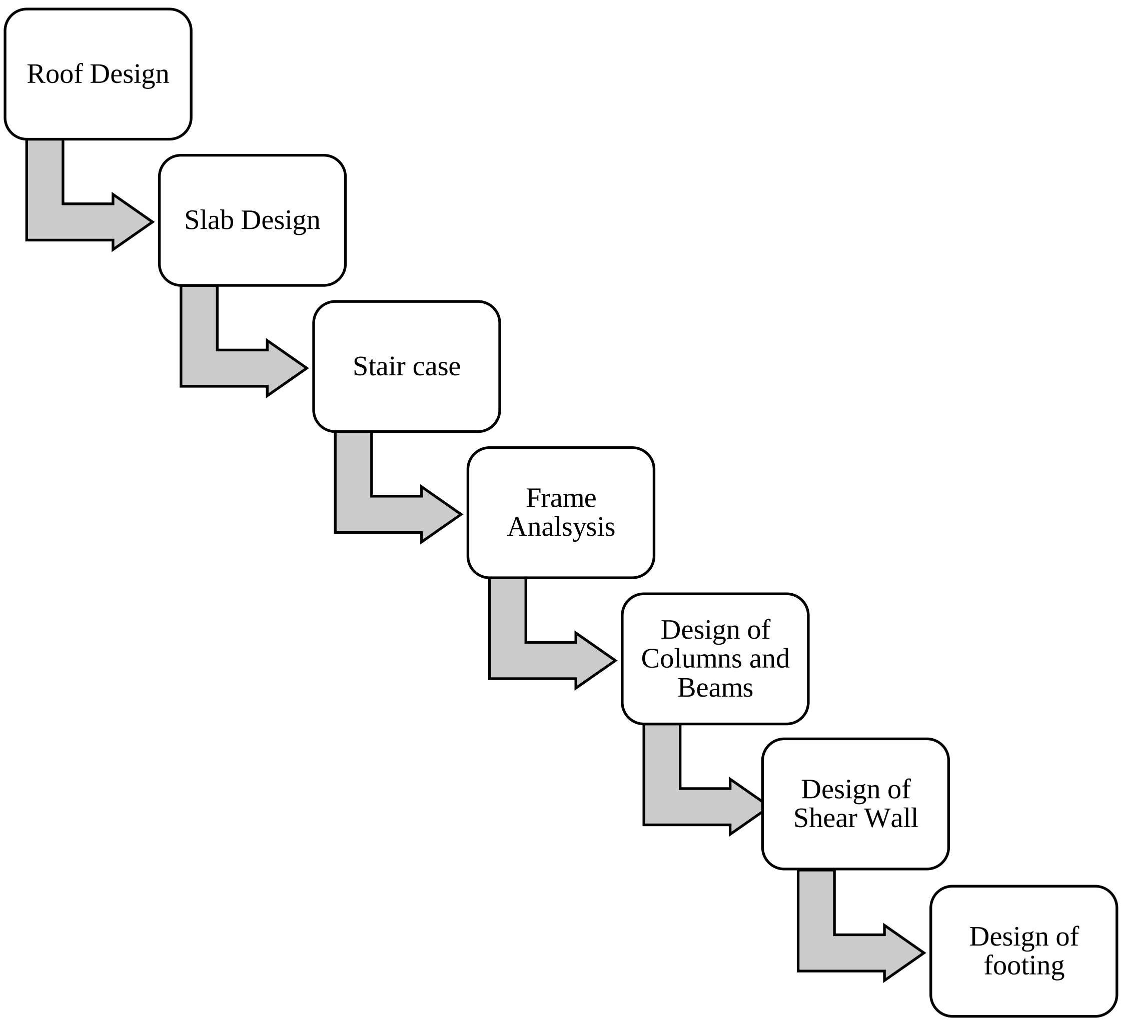


Figure 2.3- Overall framework of design and analysis

Roof design

Roof is the most upper part of a building that protects from any kind of weather. It is subjected to different kinds of loads such as wind load, its own self weight, and the loads of the persons who goes on the roof for maintenance and snow loads too. Types of roof on this project is flat roof.

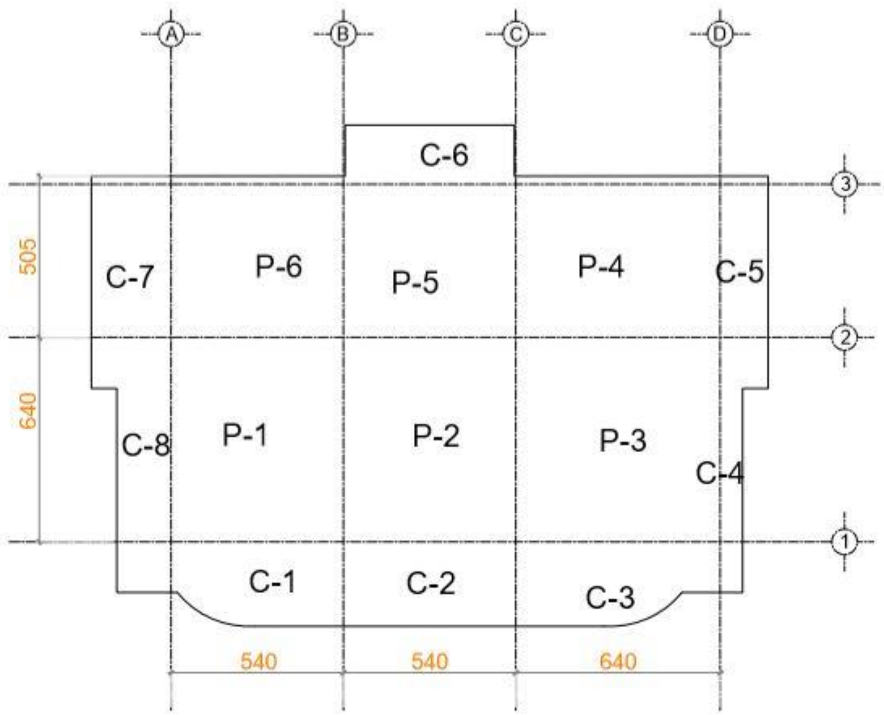


Figure 3.1- Roof plan structural layout

3.1 Wind load analysis

Wind actions fluctuate with time and acts directly pressures on the external surfaces of enclosed structures and directly on the internal surface of open structures. The wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the external effects of turbulent wind. The effect of the on structure depend on the size, shape and dynamic properties of the structure.

The magnitude of the wind load depends on the roof shape, wind direction and location of the building. Appropriate fasteners and holding down bolts or anchors must be used.

Wind pressure, $W = q_p(z) * C_{pe}$ ....Eqn (1)

Where, qp(z)=peak velocity pressure

Cpe, Cpi = coefficient of pressure

Detail introduction of the analysis of wind load based on (ES EN 1991: Part 1-4, 2015) analysis and design of roof. Wind load on the structure depends on many factors such as;

- Wind velocity direction
- The height of the structure, on this structure +18.60 m along
- Topographic location of the structure
- Shape of the structure
- Terrain category
- The roughness of the surrounding

Action of the wind loads on structures is represented either as a wind pressure or a suction force. Wind pressure on the structure may be external wind pressure or internal wind Pressure. External wind pressure (Wex) is the wind pressure acting on the external Surfaces of a structure and internal wind pressure is the wind pressure acting on the internal surfaces of a structure.

Structures deflect or stop the wind, converting the wind's kinetic energy into potential energy of pressure-creating load. The action of wind can be of the type of suction or pressure to our structures both externally or internally. The structural building design of B+G+4 mixed use building located in Addis Ababa, Ethiopia.

The magnitude of the wind load depends on the roof shape, wind direction and location of the building. Appropriate fasteners and holding down bolts or anchors must be used.

W=qp(z)*Cpe

Where; W iswind pressure

qp(z) is peak velocity pressure

Wnet= We-Wi= qp (z)Cpe-qp(z)Cpi..Eqn (2)

Wnet= qp (z)[Cpe-Cpi].

Determination of peak velocity pressure (qp (z)) qp (z)=[1+7lv(z)] [(1/2) ρVm^2(z)]... Eqn (3)

A. Turbulence Intensity lv(z)

Lv (z)=$\frac{\sigma v}{Vm(z)}$=$\frac{k1}{C0(z)*ln\frac{Z}{Zo}}$ for Zmin ≤ Z ≤ Zmax

Lv(z)= =$\frac{k1}{C0(z)*ln\frac{Zmin}{Zo}}$ for Z ≤ Zmin

B. Standard deviation of the turbulence

Wind turbulent intensity lv (z) at height Z is defined as the standard deviation of the turbulence divided by the mean wind velocity. The standard deviation of the turbulence $\sigma_{v,}$ may determine

σv= Kr*Vb*Kl...Eqn (4)

Where;

σv is standard deviation of the turbulence

Kr is terrain factor

Vb is for wind velocity and

Kl is turbulent factor, recommended 1.0

C. Mean Wind

The mean wind velocity Vm (z) at a height Z above the terrain depends on the terrain roughness and orography and on the basic wind velocity Vb, and should be determined using

Vm (z) = Cr (z) *Co(z)* Vb ……………………..............................Eqn (5)

Where;
Cr (z) is the roughness factor

Co (z) is the orography factor, taken as 1.0

Vb=Cdir*Cseason*Vb,o...Eqn (6)

Where;

Cdir is the directional factor, recommended 1.0

Cseason is the season factor, recommended 1.0

Vb is the basic wind velocity, defined as a function of wind direction and time of year at

Vb,o is the fundamental value of the basic wind velocity

D. Roughness factor Cr(z)

On this structure the area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height then terrain category II selected. Terrain roughness factor Cr(z) at height 18.60m(z) is

$$Cr(z) = K_T \ln\left[\frac{Z}{Z_o}\right] \text{ for } Z_{min} \leq Z \leq Z_{max} \text{.. Eqn (7)}$$

Where;

Zo is roughness length is 0.05 m from Table 2.

Kr terrain factor depending on the roughness length Zo calculated using

$$Kr = 0.19*(Zo/Z_{0.11})^{0.07} \text{...Eqn (8)}$$

Where;

$Z_{0.11}$ =0.05m

Zmin is minimum height is 2m

Zmax is to be taken as 200 m,

$Kr=0.19*(0.05/0.05)^{0.07} = 0.19$

$$Cr(z) = K_T \ln\left[\frac{Z}{Z_o}\right] = 0.19*\ln(18.60/0.05)$$

Cr (z) = 1.125

Table-3.1 Terrain categories and terrain parameters

Terrain category	Z_0 m	Z_{min} m
0 Sea or coastal area exposed to the open sea	0.003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle height	0.05	2
III 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)	0.3	5
IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m	1.0	10

Wind turbulent intensity lv (z)at height Z is defined as the standard deviation of the turbulence divided by the mean wind velocity. The standard deviation of the turbulence $\sigma_{v,}$ may determine

σ= Kr*Vb*Kl..(ES EN 1991, 2015) Eqn (9)

Where;

Kr is terrain factor

Vb is for wind velocity and

Kl is turbulent factor, recommended 1.0

$$I_{v(z)} = \frac{\sigma_v}{v_{m(z)}} = \frac{k_l}{c_{o(z)} * \ln(\frac{z}{z_o})} \quad for\ z_{min} \le z \le z_{max}$$

$$I_{v(z)} = I_{v(zmin)} = \frac{k_l}{c_{o(z)} * \ln(\frac{z_{min}}{z_o})} \quad for\ z \le z_{min}$$

Co is orography factor

l_v =0.17

Peak velocity pressure

The peak velocity pressure qp(z) at height z, which includes mean and short-term velocity fluctuations, should be determined (ES EN 1991, 2015)

$$qp\,(z)=[1+7*lv(z)]*1/2*\rho*Vm^2(z)=Ce(z)*qb \qquad \text{Eqn (10)}$$

Where;

ρ is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms it recommended 1.25 kg/m^3.

Ce (z) is the exposure factor

$$Ce\,(z)=\frac{qp(z)}{qb} \qquad \text{Eqn (11)}$$

qb is the basic velocity pressure

$$qb=1/2*\rho*V_b^2 \qquad \text{Eqn (12)}$$

qp(z)=837.76 N/m^2=0.838 KN/m^2

qb=302.5 N/m^2 =0.3025 KN/m^2

Ce(z)=2.77

The wind pressure acting on the external surfaces, we, should be obtained;

$$We= q_p(z)*C_{pe} \qquad \text{Eqn (13)}$$

$$Wi= q_p(Zi)*C_{pi} \qquad \text{Eqn (14)}$$

Where;

qp(ze) is the peak velocity pressure

Cpe, Cpi is the pressure coefficient for the external and internal pressure respectively

External pressure coefficient

Calculate external pressure based on appendix A1

$C_{pe} = C_{pe,1}$	$A \leq 1\ m^2$
$C_{pe} = C_{pe,1} + (C_{pe,10} - C_{pe,1}) \log_{10} A$	$1\ m^2 < A < 10\ m^2$
$C_{pe} = C_{pe,10}$	$A \geq 10\ m^2$

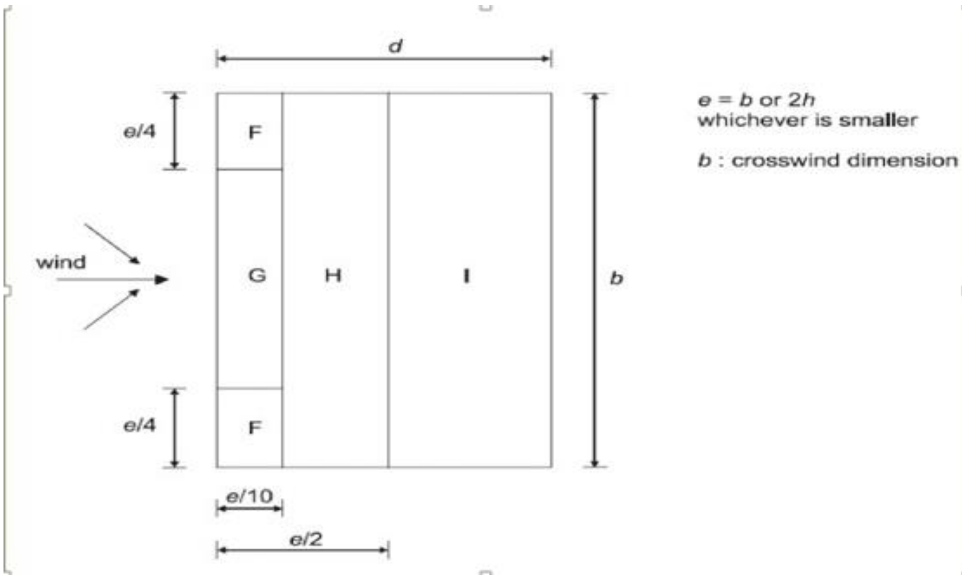


Figure 3.2- key for flat roofs (ES EN 1991, 2015)

b=21.20m

h=18.60m 2h=37.2m

so e=b=21.20m

e/4=21.20/4=5.30m e/2=10.6m e/10=21.20/10=2.12m

Area F---5.30m*2.21m=11.23m^2

Area G—(21.20m-10.6m)*2.12m=22.472 m^2

Area H—(10.6m-2.12m)*21.20m=179.776 m^2

Area I—10.6m*21.20m=224.72 m^2

hp/h=1/17.6=0.05

Table-3.2 external pressure coefficients for flat roof

Zones	F		G		H		I	
Area(m^2)	11.23		22.472		179.776		224.72	
	Cpe,10	Cpe,1	Cpe,10	Cpe,1	Cpe,10	Cpe,1	Cpe,10	Cpe,1

hp/h=0.05	-1.4	-2.0	-0.9	-1.6	-0.7	-1.2	+0.2	-0.2
Cpe	-1.4		-0.9		-0.7		+0.2	

Internal pressure coefficient

When the area of the openings at the dominant face is twice the area of the openings in the remaining faces,

Cpi= 0.75*Cpe.. Eqn (15)

When the area of the openings at the dominant face is at least 3 times the area of the openings in the remaining faces,

Cpi= 0.90*Cpe.. Eqn (16)

Table-3.3 wind pressure

Zone	Cpe	We=0.838*Cpe	Cpi	Wi=0.838*Cpi	Wnet=We-Wi
F	-1.4	-1.1732	-1.1	-0.880	-0.293
G	-0.9	-0.7542	-0.7	-0.566	-0.189
H	-0.7	-0.5866	-0.5	-0.440	-0.147
I	+0.2	0.1676	0.15	0.126	0.042

3.2 Design roof slab (solid slab)

In order to determine the depth of the slab, first it needs to find the concrete cover and effective depth. Consider one meter strip width, b=1000mm.

3.3 Concrete cover

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 concert surface.(Debisa, 2020)

The nominal cover C_{nom} is defined as a minimum cover, C_{min}, plus an allowance in design for deviation, ΔC_{dev}:

$C_{nom} = C_{min} + \Delta C_{dev}$ ....(ES EN 1992-1-1, 2015) Eqn (17)

Where;

The minimum Concrete Cover (C_{min}) should be set to satisfy the requirements safe transmission of bond forces, durability and fire resistance.

ΔCdev is an allowance which should be made in the design for deviations from the minimum cover. It should be taken as 10 mm, unless fabrication (i.e. construction) is subjected to a quality assurance system, in which case it is permitted to reduce ΔCdev to 5mm. (ES EN 1992-1-1, 2015)

$$C_{min} = \max\begin{cases} \text{Cmin, b;} \\ \text{Cmin, dur} + \Delta\text{Cdur}, \gamma - \Delta\text{Cdur, st} - \Delta\text{Cdur, add} \\ 10\ \text{mm} \end{cases} \quad \text{Eqn (18)}$$

Where

- Cmin,b -minimum cover due to bond requirement (ES EN 1992-1-1, 2015)
- Cmin,dur - minimum cover due to environmental conditions, (ES EN 1992-1-1, 2015)
- ΔCdur,γ- additive safety element(ES EN 1992-1-1, 2015)
- ΔCdur,st -reduction of minimum cover for use of stainless steel(ES EN 1992-1-1, 2015)
- ΔCdur,add -reduction of minimum cover for use of additional protection, (ES EN 1992-1-1, 2015)

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero from the ES EN 1992-1-1, Art 4.4.1.2(8).

Cover design for bond-assume Φ10 longitudinal bar and Φ20 nominal maximum aggregate size;

Cover design for Corrosion/ Durability

The building is founded on dry or permanently wet so the condition of exposure is given to be XC1, Member with slab geometry and XC....reduced by 1, Here connect XC1 and member with slab geometry we get reduced class by 1. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths but based on the above table the exposure class is reduce by 1 and the structural class would be S3.(Debisa, 2020)

Therefore the value of minimum cover required for durability of reinforcement steel is determined using (ES EN 1992-1-1, 2015) table 4.4N

Table-3.4 Values of minimum cover, $C_{min,dur}$ requirements

Environmental Requirement for $C_{min,dur}$ (mm)							
Structural Class	Exposure Class						
	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	29	30	35	40	45	50
S6	20	25	35	40	45	50	55

Here connect S3 and XC1 we get 10mm.

Then, -- Cmin= max {Cmin,dur=10mm; Cmin,b=10mm; 10mm}

Therefore Cmin,b=10mm.

ΔCdev (allowance in Design for Variation) recommended 10mm, therefore

$C_{nom} = C_{min} + \Delta C_{dev}$ = 10mm+10mm=20mm the cover is 20mm.

Cover design fire from ES EN 1992 1-2 Table 5.8 minimum dimensions and axis distances for reinforced and pre stressed concrete simply supported one way and two way solid slabs goas to 20.

Governing cover for corrosion, bond/durability and fire Cover=20mm.

3.4 Depth determination

Effective depth determinations; Serviceability requirement by ES EN, 1992; 2015

$$\frac{l}{d}=k[11+1.5\sqrt{fck}\,\frac{\rho o}{\rho}+32\sqrt{fck}(\frac{\rho o}{\rho}-1)^{\frac{3}{2}} \qquad \text{..Eqn (19)}$$

Where;

l/d- is the limit span/depth,

k- Is the factor to take into account the deference structural system,

ρo- is the reference reinforcement ratio = $10^{-3}\sqrt{fck}$, ρ- Is the required tension reinforcement ratio at the mid span to resist the moment due to the design loads (at the support for cantilever) ṕ= is the required comparison reinforcement ratio at the mid span to resist the moment due to design loads (at the support for cantilever) fck in MPa units.(Appendix A, A-2)

Assume that ρ=ρo and equation

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

Where; $N=11+1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho}+3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}-1\right)^{3/2}$

N=11+1.5*20^0.5=17.71,

F1=500/fyk=500/300=1.67, F2=F3=1

Basic ratios of span/effective depth for reinforced concrete members without axial compression table 7.4N (Debisa, 2020) the value of K for end span is 1.3, for interior span is 1.5, and for cantilever is 0.4.

End span (K=1.3)

$\frac{l}{d}$=17.71*1.3*1.67*1*1= 28.78 where l=lx

Interior span (K=1.5)

$\frac{l}{d}$=17.71*1.5*1.67*1*1= 33.21

Cantilever (K=0.4)

$\frac{l}{d}$=17.71*0.4*1.67*1*1= 8.86

From table 6 is describe that d=Lx/(K*N*F1*F2*F3)

D = dmin + Cnom +φl/ 2 where dmin is governing effective depth.

Example 6400/(17.71*1.5*1.67)= 144.26mm

D=144.26mm+20mm+10mm/2=170mm

Provide D = 170mm

Table-3.5 Depth determination on roof slab

PANEL	Support condition	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P2	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P3	Interior	6400	6400	17.71	1.5	1.67	1	144.26	170
P4	Interior	5050	6400	17.71	1.3	1.67	1	131.34	157
P5	Interior	2080	5400	17.71	1.5	1.67	1	46.89	73
P6	Interior	5050	5400	17.71	1.3	1.67	1	131.34	157
C 1	cantilever	700	5400	17.71	0.4	1.67	1	59.17	85
C2	cantilever	700	5400	17.71	0.4	1.67	1	59.17	85
C3	cantilever	1000	5400	17.71	0.4	1.67	1	84.53	111
C4	cantilever	700	6390	17.71	0.4	1.67	1	59.17	85
C5	cantilever	1000	6650	17.71	0.4	1.67	1	84.53	111
C6	cantilever	1000	3050	17.71	0.4	1.67	1	84.53	111
C7	cantilever	1000	6390	17.71	0.4	1.67	1	84.53	111
							Dmax.		170
							Dprovided		170

3.5 Load Determination

Loading on the roof is

Dead load;

Rc slab =0.24m*24KN/m^3=7.5KN/m^2

Cement screed=0.05m*23KN/m^3=1.15KN/m^2

Floor finish marble=0.03m*27 KN/m^3=0.81KN/m^2

Plaster and Painting=0.015m*23 KN/m^3=0.345KN/m^2

Water reservoir=9.81KN/(21.2m*15.67m)= 0.295KN/m^2

Water proofing=0.006m*17 KN/m^3=0.102KN/m^2

Dead load=10.20 KN/m^2

Imposed load 1 KN/m^2 for maintenance

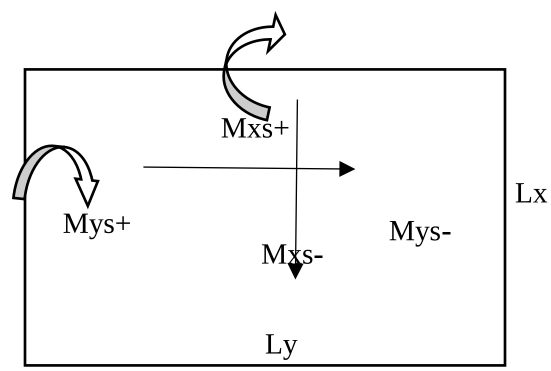
Design load (Pd)=$1.35DL + 1.5LL$+wind load positive

$1.35*10.20\ KN/m^2+1.5*1\ KN/m^2+0.042\ KN/m^2= 15.32\ KN/m^2$

3.6 Design the moment

The precise determination of moments in two way slabs with various conditions of continuity at supported edges is mathematically formidable and not suited to design practice. Various simplified methods have been adopted for determining moments, shears and reactions such slabs. Approximate methods of analysis: coefficient method, yield line and strip method.

The moment is design by using the coefficient method for this building. In slabs where the corners are prevented from lifting, and provision for torsion made, the maximum design moments per unit width are given by the following equations.



$Mxs=\beta sxnLx^2$

$Mys=\beta synLx^2$...Eqn (20)

Where;

Mxs maximum design ultimate moments either over supports or at mid span on strips of unit width and span Lx

Mys maximum design ultimate moments either over supports or at mid span on strips of unit width and span Ly

βsx and βsy moment coefficients find by using value of Ly/Lx, if the exact value is not present to gate the value using interpolation these coefficient is from (Appendix A, A-4)

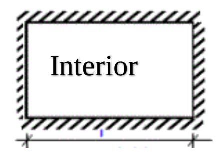
Lx length of shorter side and,

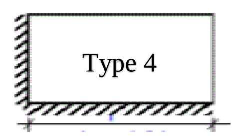
Ly length of longer side

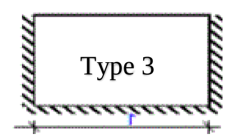
Table-3.6 Roof slab moment analysis

PANEL	TYPE	Lx(m)	Ly(m)	Ly/Lx	n(Pd)) (KN/m^2)	βsx-	βsx+	βsy-	βsy+	Mxs- (KN)	Mxs+ (KN)	Mys- (KN)	Mys+ (KN)
P1	Interior	5.4	7.1	1.31	15.32	0.046	0.032	0.035	0.024	20.55	14.30	15.64	10.72
P2	Interior	5.4	6.4	1.19	15.32	0.042	0.032	0.032	0.024	18.76	14.30	14.30	10.72
P3	Interior	6.4	6.4	1.00	15.32	0.031	0.032	0.024	0.024	19.45	20.08	15.06	15.06
P4	type 3	5.05	6.4	1.27	15.32	0.061	0.037	0.046	0.028	23.83	14.46	17.97	10.94
P5	Interior	5.05	5.4	1.07	15.32	0.035	0.032	0.027	0.024	13.67	12.50	10.55	9.38
P6	type 3	5.05	7.1	1.41	15.32	0.069	0.037	0.051	0.028	26.96	14.46	19.93	10.94

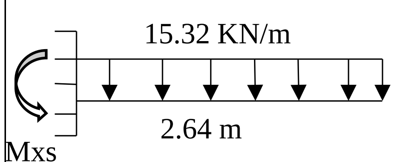
PANEL	TYPE	Lx(m)	Ly(m)	Ly/Lx	n(Pd) (KN/m^2)	Mxs(KN)
C 1	Cantilever	2.64	5.4	2.05	15.32	53.39
C2	Cantilever	2.64	5.4	2.05	15.32	53.39
C3	Cantilever	2.64	6.4	2.42	15.32	53.39
C4	Cantilever	0.7	6.39	9.13	15.32	3.75
C5	Cantilever	1.5	5.05	3.37	15.32	17.24
C6	Cantilever	1.6	5.4	3.38	15.32	19.61
C7	Cantilever	2.5	6.65	2.66	15.32	47.88
C8	Cantilever	1.7	6.39	3.76	15.32	22.14







For Cantilever



$Mxs = WL^2/2$

$= 15.32KN/m * (2.64m^2)/2 = 53.39KN$

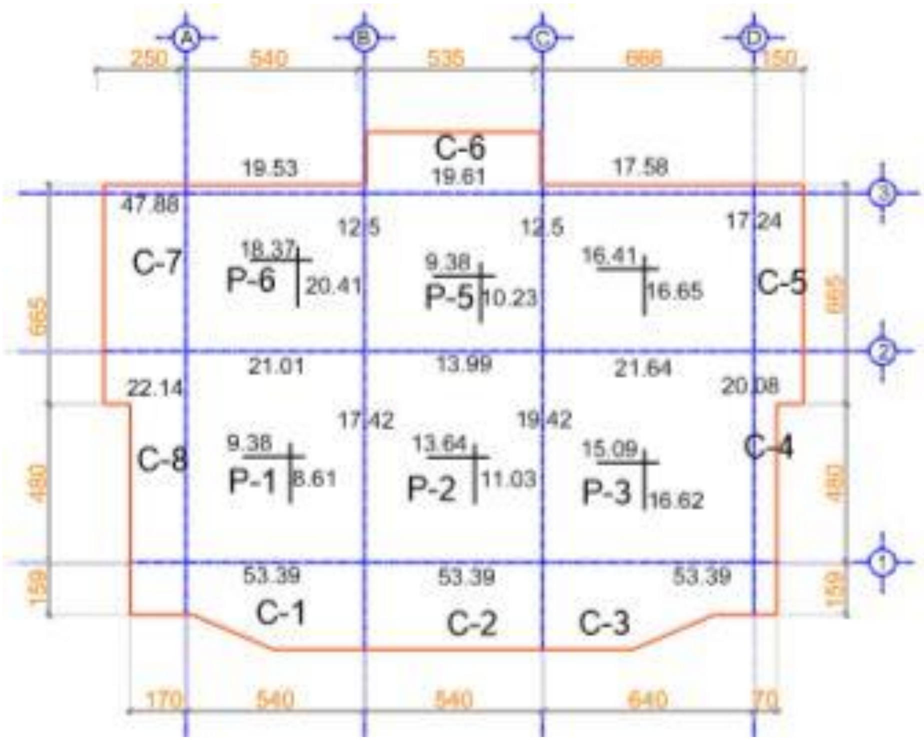


Figure 3.3- Roof moment analysis

3.7 Moment adjustment

a. Support moment adjustment;-

Redistribution of support moments and adjustment of span moments. For each support over which the slab is continuous, there are two different support moments. The differences are distributed between the spans (panels) on either side of the support to equalize the moments by the method given in EBCS-2-1995 A.3.3

- Large moment, the design moment is the average of the two panel. $\Delta M < 0.2$ use average method I
- The unbalance moment is distributed based on their stiffness. $\Delta M > 0.2$ use moment distribution method II

P1 and P2

20.55-18.76=1.79KNm

1.79*100/20.55=8.71% <20%.......use method I

20.55+18.76/2= 17.42KNm

P1 and P6

36.96-14.30=12.66KNm

12.66*100/26.96=46.96% > 20%........use method II

Mx1=-26.96KNm

K1=$(\frac{3}{4*5.05})$=0.149

DF1=Mx1*K1=26.96*0.149=0.487

Madj= -26.96+(0.487*12.66)

= -21.01KNm

Mx2=14.30KNm

K1==$(\frac{1}{6.4})$=0.156

DF1=Mx1*K1=14.30*0.156=0.513

Madj=14.30+(0.513*12.66)

=21.01KNm

If the moment adjusted panel and cantilever is taking the maximum moment.

Table-3.7 Support moment adjustment on Roof

PANEL	Method I	Method II	MAX
P1&P2	17.42	-	-
P1&P6	-	21.01	-
P1&C1	53.39	-	-
P1&C8	-	-	22.14
P2&P3	19.42	-	-
P2&P5	13.99	-	-
P2&C2	-	-	53.39
P3&P4	21.64	-	-
P3&C3	-	-	53.39
P3&C4	-	-	20.08
P4&P5	12.5	-	-
P4&C5	-	-	17.24
P5&P6	12.5	-	-
P5&C6	-	-	19.61
P6&C7	-	-	47.88

b. Span moment adjustment

Adding the span moment and subtract the adjusted moment

For P1 on the Lx direction

16.08KNm+10.72KNm-17.42KNm= 9.38KNm

Ly direction

14.74KNm+10.72KNm-16.85KNm= 8.61KNm

Table-3.8 Span moment adjustment on Roof

PANEL	Msx-	Msx+	Madjust	Msx
P1	16.08	10.72	17.42	9.38
P2	18.76	14.3	19.42	13.64
P3	19.45	15.06	19.42	15.09
P4	23.83	14.46	21.64	16.65
P5	13.67	10.55	13.99	10.23
P6	26.96	14.46	21.01	20.41
PANEL	Msy-	Msy+	Madjust	Msy
P1	14.74	10.72	16.85	8.61
P2	14.3	10.72	13.99	11.03
P3	20.08	15.06	18.52	16.62
P4	17.97	10.94	12.5	16.41
P5	12.5	9.38	12.5	9.38
P6	19.93	10.94	12.5	18.37

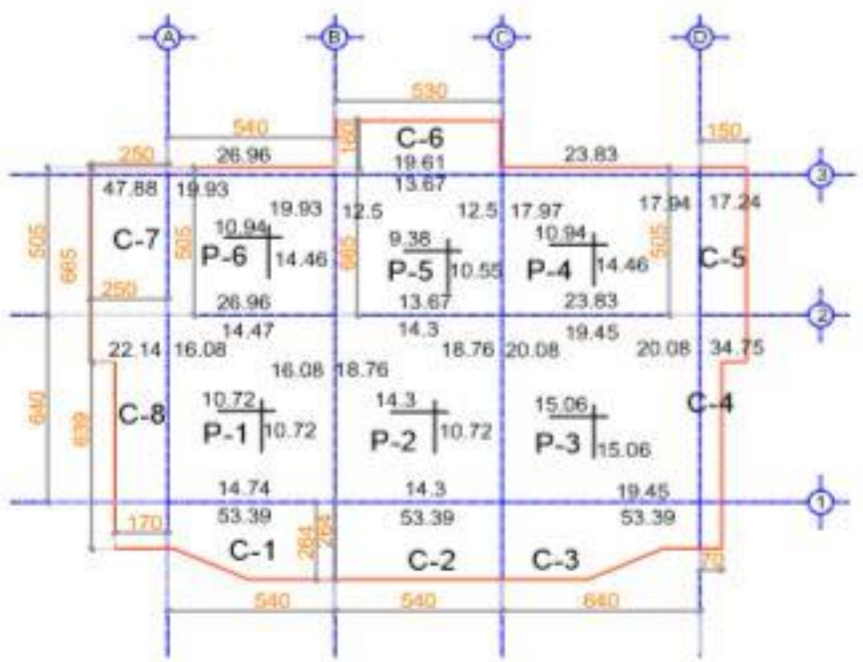


Figure 3.4- Roof moment adjustment

3.8 Design reinforcement bar

Choosing 10ϕ bar the depth in longitudinal and traverse directions are the following.

For the given data;--

Material, C-20, S-400, Depth, D-20-10/2, Width, b=1000mm, fyk=400Mpa, fyd=347.83Mpa, fctm=2.2

$$\mu sd = \frac{msd}{fcd*b*d^2} \text{ .... Eqn (21)}$$

$$Ast = \frac{msd}{Z*fyd} \text{ .... Eqn (22)}$$

$$Ast,min = \max\begin{cases} 0.26\frac{fctm}{fyk}*b*d \\ 0.0013b*d \end{cases} \text{ .... Eqn (23)}$$

$$S = \frac{b*as}{As} \text{ .... Eqn (24)}$$

$$S_{max} = \max\begin{cases} 3D \\ 400 \end{cases} \text{ .... Eqn (25)}$$

Where

Msd is moment

μsd; is relative ultimate moment

As; total area of reinforcement

as; area of single reinforcement

S; spacing

For example for panel 1of roof slab

$$\mu sd = \frac{20.55*10^6}{11.33*1000*225^2} = 0.036$$

$$Ast = \frac{20.55*10^6}{220.89*260.87} = 356.62mm^2$$

$$Ast,min = \max\begin{cases} 0.26\frac{2.2}{300}*1000*225 = 429mm2 \\ 0.0013*1000*225 = 292.5mm2 \end{cases} = 429mm^2$$

$$as = \frac{\Pi*10^2}{4} = 78.5\ mm^2$$

S_{max}= max$\begin{cases} 3 * 250 = 750mm \\ 400mm \end{cases}$ the space between the main bars shall not exceed 750mm.

S=$\frac{1000*78.5}{429}$ =220.1mm~220mm

Therefore ϕ10 cc 220mm.

Table-3.9 Spacing for reinforcement of roof slab

Panel		M(KNm)	Rebar
P-1	Mxs-	20.55	Φ 10 cc 220mm
	Mxs+	14.3	Φ 10 cc 310mm
	Mys-	15.64	Φ 10 cc 290mm
	Mys+	10.72	Φ 10 cc 420mm
P-2	Mxs-	18.76	Φ 10 cc 240mm
	Mxs+	14.3	Φ 10 cc 310mm
	Mys-	14.3	Φ 10 cc 300mm
	Mys+	10.72	Φ 10 cc 420mm
P-3	Mxs-	19.45	Φ 10 cc 230mm
	Mxs+	20.08	Φ 10 cc 220mm
	Mys-	15.06	Φ 10 cc 300mm
	Mys+	15.06	Φ 10 cc 300mm
P-4	Mxs-	23.83	Φ 10 cc 180mm
	Mxs+	14.46	Φ 10 cc 310mm
	Mys-	17.97	Φ 10 cc 250mm
	Mys+	10.94	Φ 10 cc 410mm
P-5	Mxs-	13.67	Φ 10 cc 330mm
	Mxs+	12.5	Φ 10 cc 360mm
	Mys-	10.55	Φ 10 cc 430mm
	Mys+	9.38	Φ 10 cc 480mm
P-6	Mxs-	26.96	Φ 10 cc 160mm
	Mxs+	14.46	Φ 10 cc 310mm
	Mys-	19.93	Φ 10 cc 210mm
	Mys+	10.94	Φ 10 cc 410mm
C-1	Mxs	53.39	Φ 10 cc 80mm
C-2	Mxs	53.39	Φ 10 cc 80mm
C-3	Mxs	53.39	Φ 10 cc 80mm
C-4	Mxs	3.75	Φ 10 cc 750mm
C-5	Mxs	17.24	Φ 10 cc 260mm
C-6	Mxs	19.61	Φ 10 cc 230mm
C-7	Mxs	47.88	Φ 10 cc 90mm
C-8	Mxs	22.14	Φ 10 cc 200mm

3.9 Loads on supporting Beams

The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations:

Vsx=βvx n Lx.. Eqn (26)

Vsy=βvy n Ly.. Eqn (21)

Where:

Vsx Design end sear on strips of unit width and span Ly and considered to act over the middle three-quarters of the edge

Vsy Design end sear on strips of unit width and span Lx and considered to act over the middle three-quarters of the edge

Where design ultimate support moments are used which differ substantially from those that would be assessed from table, adjustment of the values given in table may be necessary (Appendix A, A-5)

Table-3.10 Load transfer from slab to beam

PANEL	TYPE	Lx(m)	Ly(m)	Ly/Lx	n(Pd)(KN/m^2)	βvx	βvy	Vsx(KN/m)	Vsy(KN/m)
P1	Interior	5.4	7.1	1.31	15.32	0.41	0.33	33.92	27.30
P2	Interior	5.4	6.4	1.19	15.32	0.39	0.33	32.26	27.30
P3	Interior	6.4	6.4	1.00	15.32	0.33	0.33	32.36	32.36
P4	Type 3	5.05	6.4	1.27	15.32	0.46	0.36	35.59	27.85
P5	Interior	5.05	5.4	1.07	15.32	0.35	0.33	27.08	25.53
P6	Type 3	5.05	7.1	1.41	15.32	0.49	0.36	37.91	27.85

PANEL	TYPE	Lx(m)	Ly(m)	Ly/Lx	n(Pd)(KN/m^2)	WL(KN)
C 1	cantilever	2.64	5.4	2.05	15.32	40.44
C2	cantilever	2.64	5.4	2.05	15.32	40.44
C3	cantilever	2.64	6.4	2.42	15.32	40.44
C4	cantilever	0.7	6.39	9.13	15.32	10.72
C5	cantilever	1.5	5.05	3.37	15.32	22.98
C6	cantilever	1.6	5.4	3.38	15.32	24.51
C7	cantilever	2.5	6.65	2.66	15.32	38.30
C8	cantilever	1.7	6.39	3.76	15.32	26.04

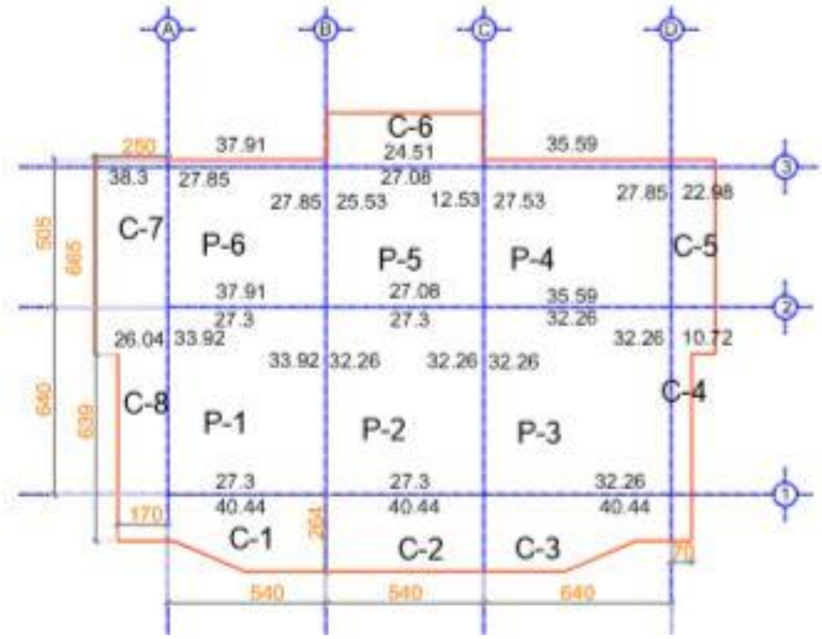


Figure 3.5- Load transfer from roof slab to beam

Slab design

4.1 Introduction

A slab is a broad, flat plate usually horizontal with top and bottom surfaces parallel or nearly. The purpose is supports vertical loads and from an integral portion of the structural frame to resist lateral forces.

There are two types of slabs based on the load transferring mechanisms. These are one way and two way slabs. One-way slabs transmit their load in one direction while two way slabs resist applied two directions.

These types of slabs are composed of rectangular panels supported at all four edges by walls or beams stiff enough to be treated as unyielding. In this project almost of the slabs are two-way and need to be analyzed based on the principle of two way actions. This types slabs are form of construction unique to reinforced concrete among the major structural materials. Two way slab are very efficient, economical, widely used structural system.

Using on this project two way slab by coefficient method, this method provided that the slab is composed of rectangular panels, supported at all four edges by walls of beams, stiff enough to be treated as an unyielding. It is intended for slab subjected to uniformly distributed load. If the slab is subject uniform load or concentrated load, in addition to concentrated load,, this can generally be treated by considering them as equivalent uniform load using approximate rules.

4.2 Design slab (solid slab)

In order to determine the depth of the slab, first it needs to find the concrete cover and effective depth. Consider one meter strip width, b=1000mm.

4.3 Concrete cover

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 concert surface.(Debisa, 2020)

The nominal cover C_{nom} is defined as a minimum cover, C_{min}, plus an allowance in design for deviation, ΔC_{dev}:

$$C_{nom} = C_{min} + \Delta C_{dev} \text{ ....(ES EN 1992-1-1, 2015) Eqn (1)}$$

Where;

The minimum Concrete Cover (C_{min}) should be set to satisfy the requirements safe transmission of bond forces, durability and fire resistance.

ΔCdev is an allowance which should be made in the design for deviations from the minimum cover. It should be taken as 10 mm, unless fabrication (i.e. construction) is subjected to a quality assurance system, in which case it is permitted to reduce ΔCdev to 5mm. (ES EN 1992-1-1, 2015)

$$C_{min} = \max\begin{cases} \text{Cmin, b;} \\ \text{Cmin, dur} + \Delta\text{Cdur}, \gamma - \Delta\text{Cdur, st} - \Delta\text{Cdur, add} \\ 10\text{ mm} \end{cases} \quad \text{Eqn (2)}$$

Where

- Cmin,b -minimum cover due to bond requirement (ES EN 1992-1-1, 2015)
- Cmin,dur - minimum cover due to environmental conditions, (ES EN 1992-1-1, 2015)
- ΔCdur,γ- additive safety element(ES EN 1992-1-1, 2015)
- ΔCdur,st -reduction of minimum cover for use of stainless steel(ES EN 1992-1-1, 2015)
- ΔCdur,add -reduction of minimum cover for use of additional protection, (ES EN 1992-1-1, 2015)

But; the recommended value of ΔCdur,γ , ΔCdur,st , and ΔCdur,add is zero from the ES EN 1992-1-1, Art 4.4.1.2(8).

Cover design for bond-assume Φ10 longitudinal bar and Φ20 nominal maximum aggregate size;

Cover design for Corrosion/ Durability

The building is founded on dry or permanently wet so the condition of exposure is given to be XC1, Member with slab geometry and XC....reduced by 1, Here connect XC1 and member with slab geometry we get reduced class by 1. The recommended Structural Class (design working life of 50 years) is S4 for the indicative concrete strengths but based on the above table the exposure class is reduce by 1 and the structural class would be S3.(Debisa, 2020)

Therefore the value of minimum cover required for durability of reinforcement steel is determined using (ES EN 1992-1-1, 2015) table 4.4N

Table-4.1 Values of minimum cover, $C_{min,dur}$ requirements

Environmental Requirement for $C_{min,dur}$ (mm)							
Structural Class	Exposure Class						
	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	29	30	35	40	45	50
S6	20	25	35	40	45	50	55

Here connect S3 and XC1 we get 10mm.

Then, -- Cmin= max {Cmin,dur=10mm; Cmin,b=10mm; 10mm}

Therefore Cmin,b=10mm.

ΔCdev (allowance in Design for Variation) recommended 10mm, therefore

$C_{nom} = C_{min} + \Delta C_{dev}$ = 10mm+10mm=20mm the cover is 20mm.

Cover design fire from ES EN 1992 1-2 Table 5.8 minimum dimensions and axis distances for reinforced and pre stressed concrete simply supported one way and two way solid slabs goas to 20.

Governing cover for corrosion, bond/durability and fire Cover=20mm.

4.4 Depth determination

Effective depth determinations; Serviceability requirement by ES EN, 1992; 2015

$$\frac{l}{d}=k[11+1.5\sqrt{fck}\,\frac{\rho o}{\rho}+32\sqrt{fck}(\frac{\rho o}{\rho}-1)^{\frac{3}{2}}$$ ....Eqn (3)

Where;

l/d- is the limit span/depth,

k- Is the factor to take into account the deference structural system,

ρo- is the reference reinforcement ratio = $10^{-3}\sqrt{fck}$, ρ- Is the required tension reinforcement ratio at the mid span to resist the moment due to the design loads (at the support for cantilever) ṕ= is the required comparison reinforcement ratio at the mid span to resist the moment due to design loads (at the support for cantilever) fck in MPa units. (Appendix A, A-2)

Assume that ρ=ρo and equation

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

Where; $N=11+1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho}+3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho}-1\right)^{3/2}$

N=11+1.5*20^0.5=17.71,

F1=500/fyk=500/300=1.67, F2=F3=1

Basic ratios of span/effective depth for reinforced concrete members without axial compression table 7.4N (Debisa, 2020) the value of K for end span is 1.3, for interior span is 1.5, and for cantilever is 0.4.

End span (K=1.3)

$\frac{l}{d}$=17.71*1.3*1.67*1*1= 28.78 where l=lx

Interior span (K=1.5)

$\frac{l}{d}$=17.71*1.5*1.67*1*1= 33.21

Cantilever (K=0.4)

$\frac{l}{d}$=17.71*0.4*1.67*1*1= 8.86

From table 6 is describe that d=Lx/(K*N*F1*F2*F3)

D = dmin + Cnom +φl/ 2 where dmin is governing effective depth.

Example 5400/(17.71*1.5*1.67)= 121.72mm

D=121.72mm +20mm+10mm/2=148mm

Provide D = 148mm

Table-4.2 Depth determination on Ground slab

PANEL	Support Condition	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P2	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P3	Interior	6400	6400	17.71	1.5	1.67	1	144.26	170
P4	Interior	5050	6400	17.71	1.3	1.67	1	131.34	157
P5	cantilever	2080	5400	17.71	1.5	1.67	1	46.89	73
P6	Interior	5050	5400	17.71	1.4	1.67	1	121.96	148
P7	End span	2700	5900	17.71	1.3	1.67	1	70.22	96
C 1	cantilever	700	7900	17.71	0.4	1.67	1	59.17	85
C2	cantilever	700	5400	17.71	0.4	1.67	1	59.17	85
C3	cantilever	1000	8900	17.71	0.4	1.67	1	84.53	111
C4	cantilever	700	6400	17.71	0.4	1.67	1	59.17	85
C5	cantilever	1000	5050	17.71	0.4	1.67	1	84.53	111
C6	cantilever	1000	5050	17.71	0.4	1.67	1	84.53	111
C7	cantilever	1000	6400	17.71	0.4	1.67	1	84.53	111
							Dmax.		170
							Dprovided		170

The overall floor depth determination describe on Appendix A, A.6

4.5 Load Determination

Each panels might have different function and floor finishing material so we might have different dead load and live load in the same panel. In this case we take the maximum dead load and live load as a governing.

Pd=1.35 DL+1.5LL

For P 1

RC slab =0.24m*24KN/m^3=5.76KN/m^2

Cement screed=0.025m*23KN/m^3=0.58KN/m^2

Plaster and Painting=0.002m*23 KN/m^3=0.456KN/m^2

Ceramic=0.002m*22 KN/m^3=0.42KN/m^2

Total dead load=5.76KN/m^2+0.58KN/m^2+0.456KN/m^2+0.42KN/m^2=7.22KN/m^2

Table—4.3 Self-weight of slabs with different floor finish on the Ground floor

PANEL	Material	Thickness(m)	Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Dead Load (KN/m^2)
P1	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	7.22
	Ceramic	0.02	21	0.42	
P2	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Ceramic	0.02	21	0.42	10.70
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.38	
	Plastering wall	0.06	23	1.29	
P3	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	9.95
	Ceramic	0.02	21	0.42	
	Partition wall	0.18	14	2.46	
	Plastering wall	0.01	23	0.27	
P4	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.99
	Ceramic	0.02	21	0.42	
	Partition wall	0.16	14	2.28	
	Plastering wall	0.07	23	1.50	
P5	Rc Slab	0.24	24	5.76	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	8.03
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
P6	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	9.33
	Ceramic	0.02	21	0.42	
	Partition wall	0.09	14	1.28	
	Plastering wall	0.04	23	0.84	
C1	Rc Slab	0.24	24	5.76	

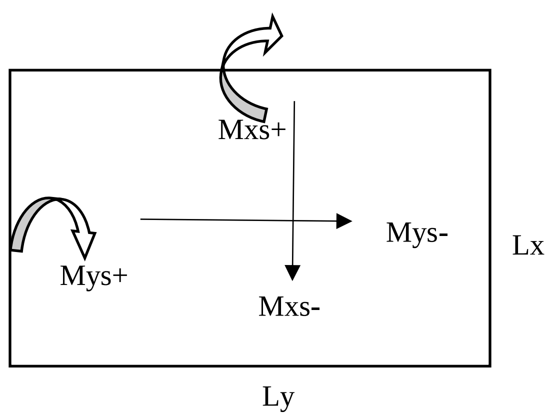
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	9.26
	Marble	0.03	27	0.81	
	Partition wall	0.06	14	0.85	
	Plastering wall	0.03	23	0.80	
C2	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	7.61
	Marble	0.03	27	0.81	
C3	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	9.34
	Partition wall	0.06	14	0.90	
	Plastering wall	0.04	23	0.84	
C4	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	7.61
	Marble	0.03	27	0.81	
C5	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Ceramic	0.02	21	0.42	8.48
	Partition wall	0.05	14	0.77	
	Plastering wall	0.02	23	0.50	
C6	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	7.22
	Plastering	0.02	23	0.46	
	Ceramic	0.02	21	0.42	
C7	Rc Slab	0.24	24	5.76	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	7.61
	Marble	0.03	27	0.81	

PANEL	Dead Load (KN/m^2)	Live Load (KN/m^2)	Total Load (KN/m^2)
P1	7.22	4.5	16.50
P2	10.7	3.5	19.70
P3	9.95	2.5	17.18
P4	10.99	2	17.84
P5	8.03	3.5	16.09
P6	9.33	2.5	16.35
C1	9.26	3.5	17.75
C2	7.61	3.5	15.52
C3	9.34	3.5	17.86
C4	7.61	3.5	15.52
C5	8.48	4	17.45
C6	7.22	4.5	16.50
C7	7.61	3.5	15.52

4.6 Design the Moment

The precise determination of moments in two way slabs with various conditions of continuity at supported edges is mathematically formidable and not suited to design practice. Various simplified methods have been adopted for determining moments, shears and reactions such slabs. Approximate methods of analysis: coefficient method, yield line and strip method.

The moment is design by using the coefficient method for this building. In slabs where the corners are prevented from lifting, and provision for torsion made, the maximum design moments per unit width are given by the following equations.



$M_{xs}=\beta_{sxn}L_x^2$

$M_{ys}=\beta_{syn}L_x^2$..Eqn (4)

Where;

Mxs maximum design ultimate moments either over supports or at mid span on strips of unit width and span Lx

Mys maximum design ultimate moments either over supports or at mid span on strips of unit width and span Ly

βsx and βsy moment coefficients find by using value of Ly/Lx, if the exact value is not present to gate the value using interpolation. (Appendix A, A-4)

Lx length of shorter side and,

Ly length of longer side

Table-4.4 Ground slab moment analysis

PANEL		Lx	Ly	Ly/Lx	n(pd)	βsx-	βsx+	βsy-	βsy+	Msx-	Msx+	Msy-	Msy+
P1	Interior	5.4	6.4	1.19	7.22	0.042	0.032	0.032	0.024	8.84	6.74	6.74	5.05
P2	Interior	5.4	6.4	1.19	10.7	0.042	0.032	0.032	0.024	13.10	9.98	9.98	7.49
P3	Interior	6.4	6.4	1.00	9.95	0.031	0.024	0.032	0.024	12.63	9.78	13.04	9.78
P4	type 3	5.05	6.4	1.27	10.99	0.061	0.037	0.046	0.028	17.10	10.37	12.89	7.85
P6	type 3	5.05	5.4	1.07	9.33	0.046	0.037	0.034	0.028	10.95	8.80	8.09	6.66

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P5	interior	2.08	5.4	2.60	8.03	17.37
C 1	cantilever	2.36	7.9	3.35	9.26	25.79
C2	cantilever	2.36	5.4	2.29	7.61	21.19
C3	cantilever	2.36	8.9	3.77	9.34	26.01
C4	cantilever	2.5	6.4	2.56	7.61	23.78
C5	cantilever	2.5	5.05	2.02	8.48	26.50
C6	cantilever	2.5	5.05	2.02	7.22	22.56
C7	cantilever	2.5	6.4	2.56	7.61	23.78

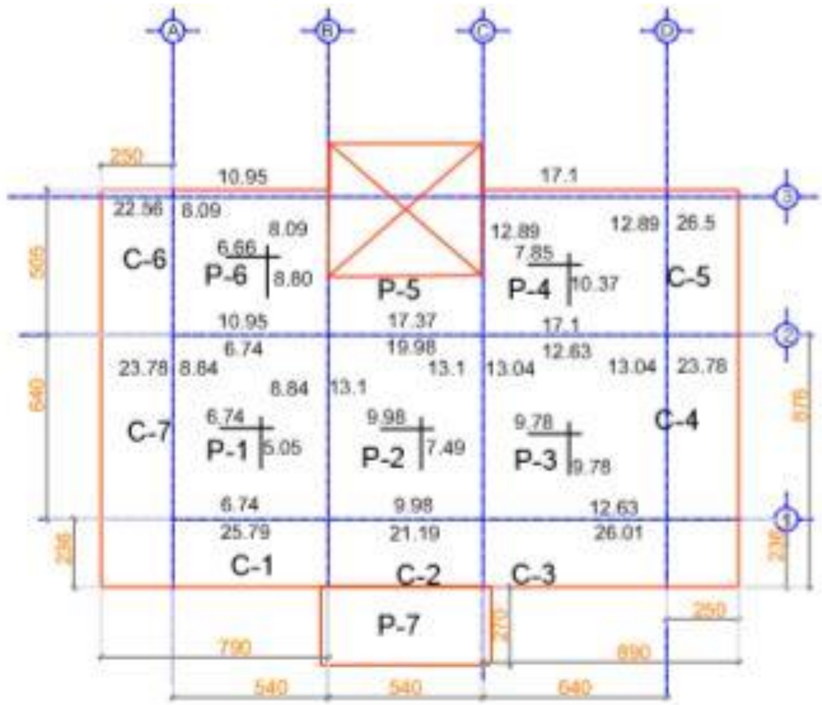


Figure 4.1- Ground moment analysis

4.7 Moment adjustment

a. Support moment adjustment;-

Redistribution of support moments and adjustment of span moments. For each support over which the slab is continuous, there are two different support moments. The differences are distributed between the spans (panels) on either side of the support to equalize the moments by the method given in EBCS-2-1995 A.3.3

- Large moment, the design moment is the average of the two panel. ΔM< 0.2 use average method I
- The unbalance moment is distributed based on their stiffness. ΔM>0.2 use moment distribution method II

For P2 &P6

8.84-8.09=0.75KNm

0.75*100/8.84=8.48% <20%.......use method I

8.84+8.09/2= 8.6KNm

b. Span or Field moment adjustment;-

Span moment is adjusted in the direction where maximum support moments exist. To adjust span moment, we add the maximum adjusted support moment and span moment in that direction then deduct the adjusted support moment.

Such span moment panel (P-1) on the ground floor

Mxs= 8.84+6.74-10.97=4.61

Msy=6.74+5.05-8.6=3.19

Table-4.5 support moment adjustment on Ground floor

PANEL	MTD 1	MTD 2	MAX
P1&P2		10.97	
P1&P6		8.6	
P1&C1			25.79
P1&C7			23.78
P2&P3	13.07		
P2&P5			17.37
P2&C2			21.19
P3&P4		14.92	
P3&C3			26.01
P3&C4			23.78
P4&P5			12.89
P4&C5			26.5
P5&P6			8.09
P6&C7			22.56

Table-4.6 span moment adjustment on Ground floor

PANEL	Msx-	Msx+	Madjust	Msx
P1	8.84	6.74	10.97	4.61
P2	13.1	9.98	13.07	10.01
P3	12.63	9.78	14.92	7.49
P4	17.1	10.37	14.92	12.55
P6	10.95	8.8	8.6	11.15
PANEL	Msy-	Msy+	Madjust	Msy
P1	6.74	5.05	8.6	3.19
P2	9.98	7.49	9.98	7.49
P3	13.04	9.78	13.07	9.75
P4	12.89	7.85	14.92	5.82
P6	8.09	6.66	8.09	6.66

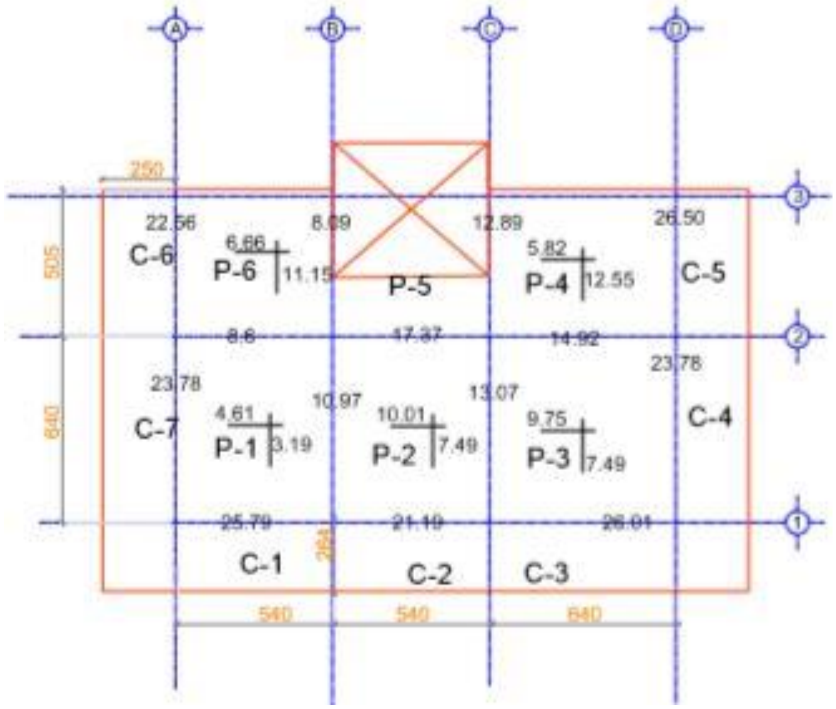


Figure 4.2- Ground moment adjustment

Flexural reinforcement

Material data; -- concrete C-25 and steel grade S-400,

Width b=1000

Total depth of slab, D=280

Check depth for flexure—

Maximum moment, M_{max} from all floor is 26.94 KNm and take k=0.15

$$d_{req}= \sqrt{\frac{Mdmax}{k*fck*1000}} \text{..Eqn (5)}$$

$$d= \sqrt{\frac{26.94*10^6}{0.15*25*1000}} = 84.75mm<280mm....ok!!$$

Therefore, depth of 210mm satisfy flexural requirement for all floors.

4.8 Reinforcement design

Area of reinforcement Ast, min =0.26($^{fctm}/_{fyk}$)bd> As, max=0.013bd

K=Msd/d2*b*fck

As=Msd/fyd*z

as=Πd^2/4

S=b*as/As

Maximum spacing is maximum of 400 & 3h

Smax=400

The value of reinforcement for each slab are tabulated as follow:

Table-4.7 reinforcement for ground floor slab

Panel		M(KNm)	Rebar
P-1	Mxs-	8.84	Φ 10 cc 490mm
	Mxs+	6.74	Φ 10 cc 640mm
	Mys-	6.74	Φ 10 cc 610mm
	Mys+	5.05	Φ 10 cc 720mm
P-2	Mxs-	13.1	Φ 10 cc 330mm
	Mxs+	9.98	Φ 10 cc 430mm
	Mys-	9.98	Φ 10 cc 410mm
	Mys+	7.49	Φ 10 cc 550mm
P-3	Mxs-	12.63	Φ 10 cc 340mm
	Mxs+	9.78	Φ 10 cc 440mm
	Mys-	13.04	Φ 10 cc 310mm
	Mys+	9.78	Φ 10 cc 420mm
P-4	Mxs-	17.1	Φ 10 cc 250mm
	Mxs+	10.37	Φ 10 cc 420mm
	Mys-	12.89	Φ 10 cc 320mm
	Mys+	7.85	Φ 10 cc 530mm
P-5	Mxs	17.37	Φ 10 cc 240mm
P-6	Mxs-	10.95	Φ 10 cc 390mm
	Mxs+	8.8	Φ 10 cc 490mm
	Mys-	8.09	Φ 10 cc 510mm
	Mys+	6.66	Φ 10 cc 620mm
C-1	Mxs	25.79	Φ 10 cc 160mm

C-2	Mxs	21.19	Φ 10 cc 200mm
C-3	Mxs	26.01	Φ 10 cc 160mm
C-4	Mxs	23.73	Φ 10 cc 180mm
C-5	Mxs	26.5	Φ 10 cc 160mm
C-6	Mxs	22.56	Φ 10 cc 190mm
C-7	Mxs	23.78	Φ 10 cc 180mm

4.9 Load transfer to supporting beams

The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations

Vsx=βvx*Pd*Lx..Enq (6)

Vsy=βvy*Pd*Lx

Where;

Vsy; Design end sear on strips of unit width and span y l and considered to act over the middle three-quarters of the edge

Vsx; Design end sear on strips of unit width and span x l and considered to act over the middle three-quarters of the edge

Where design ultimate support moments are used which differ substantially from those that would be assessed from Table, adjustment of the values given in Table may be necessary from EBCS. (Appendix A, A-5)

For P1

Vsx=βvx*Pd*Lx =0.39*16.5*5.4=34.75KN

Vsy=βvy*Pd*Lx =0.33*16.5*5.4=29.40KN

Table-4.8 Load transfer from Ground floor slab to beam

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
P1	Interior	5.4	6.4	1.19	16.5	0.39	0.33	34.75	29.40
P2	Interior	5.4	6.4	1.19	19.7	0.39	0.33	41.49	35.11
P3	Interior	6.4	6.4	1.00	17.18	0.33	0.33	36.28	36.28
P4	type 3	5.05	6.4	1.27	17.84	0.46	0.36	41.44	32.43
P6	type 3	5.05	5.4	1.07	16.35	0.39	0.36	32.20	29.72

PANEL	TYPE	Lx(m)	Ly(m)	Ly/Lx	n(Pd)(KN/m^2)	WL(KN)

P5	interior	2.08	5.4	2.60	16.09	33.47
C 1	cantilever	2.36	7.9	3.35	17.35	40.95
C2	cantilever	2.36	5.4	2.29	15.52	36.63
C3	cantilever	2.36	8.9	3.77	17.86	42.15
C4	cantilever	2.5	6.4	2.56	15.52	38.80
C5	cantilever	2.5	5.05	2.02	17.45	43.63
C6	cantilever	2.5	5.05	2.02	16.5	41.25
C7	cantilever	2.5	6.4	2.56	15.52	38.80

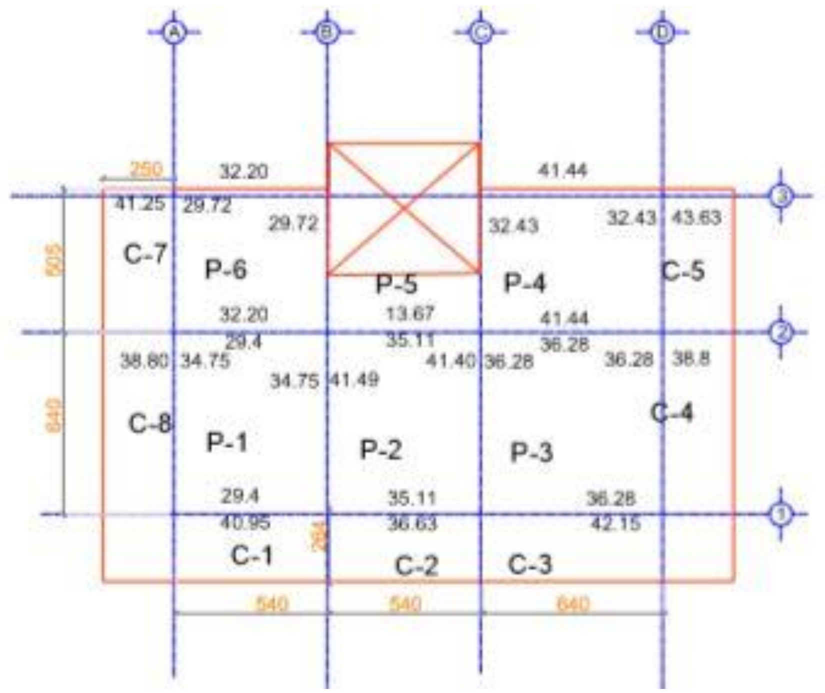


Figure 4.3 Load transfer from ground slab to beam

4.10 Check depth for Shear

The design value of shear resistance for members not requiring design shear reinforcement.

$V_{rd,c}=[C_{rd,c}*K(100\rho_1 f_{ck})^{1/3}]b_w d$…........Eqn (7) EBCS 1992-1-1:2014 equation 6.2a

$C_{rd,c}=0.18/\gamma_c=0.18/1.5=0.12$

$\rho_1=\rho_{min}=\frac{0.26*0.3*(f_{ck}^{\frac{2}{3}})}{f_{yk}}=0.001538<0.02$

$V_{rd,c}=[0.12*2(100*0.001538*20)^{1/3}]*1000*240$

Stair design

Stair is an important element of a structure which connects places of different levels. Stair case analysis and design is similar to slabs. It involves the analysis steps followed for slabs. The inclined configuration is analyzed by projecting the loads on a horizontal plane. The stair contains two flights with the same configuration. In our case the stair type is Dog legged.

5.1 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.

Given data;--

Number of riser=11

Number of tread= 10

Width of tread= 30cm

Width of riser= 15cm

EBCS 2015 recommended unit weight which are used in stair design are given bellow,

-Unit weight of marble=27KN/m^3 - Unit weight of concrete=25KN/m^3

-Unit weight of cement screed =23KN/m^3 - Unit weight of plastering=23KN/m^3

-Thickness of cement screed=2cm -Thickness of plastering=2cm

-Thickness of marble=3cm

-Take 1m width strip

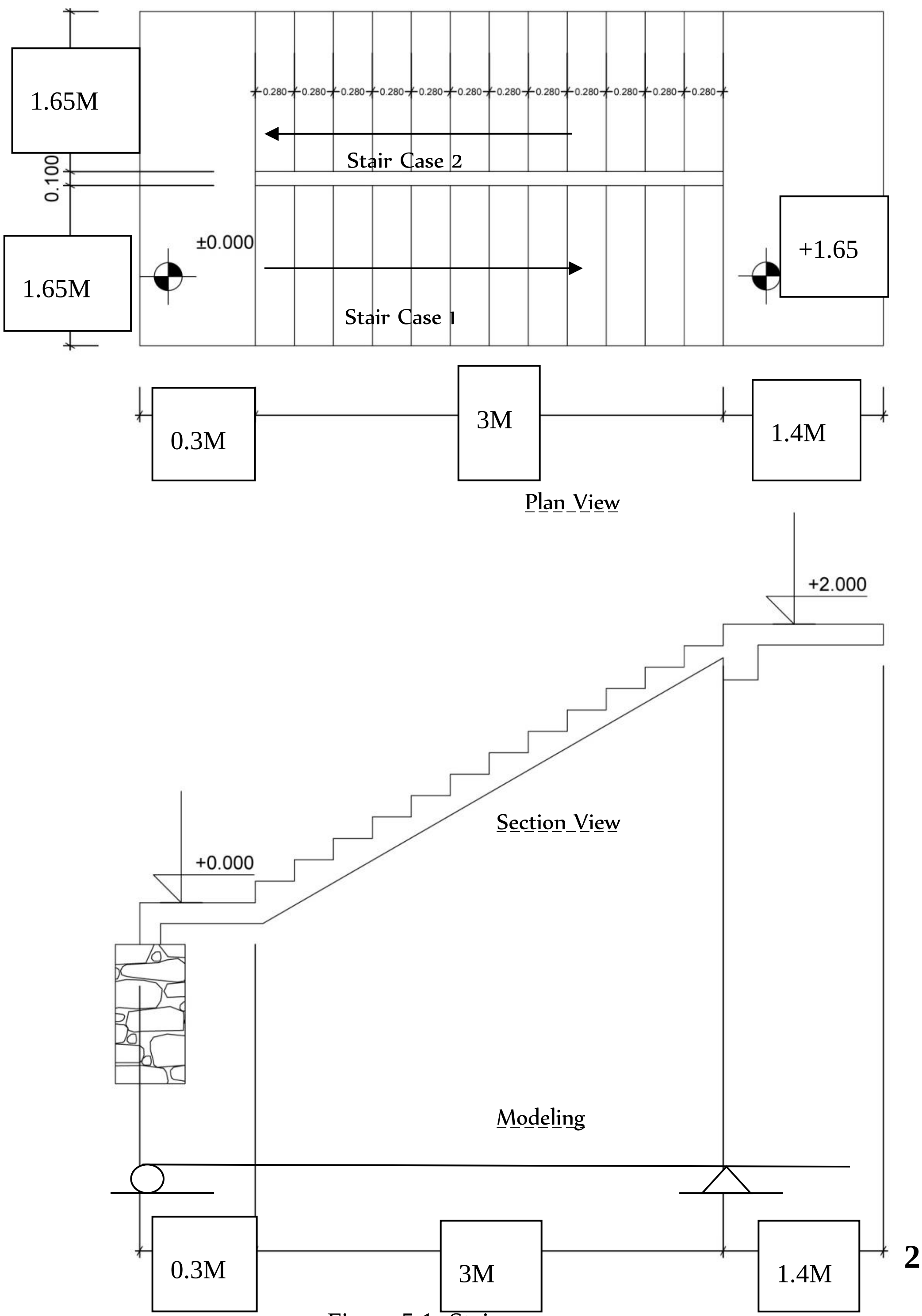


Figure 5.1- Stair case

➢ Height of Riser$=\frac{\text{Height of Stair}}{\text{No of Stair}}=\frac{1.65\text{m}-0\text{m}}{11}=15\text{cm}$

Therefor use riser height 15cm for the design.

Material C-25, S-400

5.2 Determination of minimum depth for deflection

The limiting span to depth ratio may be estimated using the following expressions and multiplied with by correction factors to allow for the type of reinforcement used and other variables.no allowance has been made fort any pre-camber in the derivation of these expressions.

$$\frac{l}{d}=k[11+1.5\sqrt{fck}\,\frac{\rho o}{\rho}+32\sqrt{fck}(\frac{\rho o}{\rho}-1)^{\frac{3}{2}}$$ ....Eqn (1)

Where;

L/d is the limit span depth,

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

ρo= is the reference reinforcement ratio ,10-3√fck

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 the design loads at support for cantilevers fck =characteristics cylindrical compressive strength of concrete in Mpa.

Basic ratios of span/effective depth for reinforced concrete members without axial compression table 7.4N (Debisa, 2020) the value of K for end span is 1.3, for interior span is 1.5, and for cantilever is 0.4.

a. Depth for deflection for the inclined slab

End span (K=1.3)(ES EN 1992-1-1, 2015)

$$\frac{l}{d}=k(11+1.5\sqrt{fck}*\frac{\rho_0}{\rho-\rho'}+\frac{1}{12}\sqrt{fck}*\left(\sqrt{\frac{\rho'}{\rho_0}}\right)^{3/2})$$

$$\frac{l}{d}=k(11+1.5\sqrt{20}*\frac{0.4472}{0.5-\rho'}+\frac{1}{12}\sqrt{fck}*\left(\sqrt{\frac{\rho'}{0.4472}}\right)^{3/2})$$

$$\frac{l}{d}=\ k(11+1.5\sqrt{20}*\frac{0.4472}{0.5}+3.2\sqrt{20}*\left(\frac{0.4472}{0.5}-1\right)^{\frac{3}{2}})$$

$\frac{l}{d}$ = 20.52*k

$\frac{5000}{d}$ = 20.52*1.3

d = $\frac{3000}{26.67}$ = 112.46 mm

D = dmin + Cnom +φl/ 2 where dmin is governing effective depth.

D= 112.46+20+20/2=142.46mm

D=150 mm

b. Depth for deflection for landing

Interior span (K=1.5) (ES EN 1992-1-1, 2015)

$$\frac{l}{d} = k(11 + 1.5\sqrt{fck} * \frac{\rho_0}{\rho-\rho'} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho'}{\rho_0}}\right)^{3/2})$$

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5-\rho'} + \frac{1}{12}\sqrt{fck} * \left(\sqrt{\frac{\rho'}{0.4472}}\right)^{3/2})$$

$$\frac{l}{d} = k(11 + 1.5\sqrt{20} * \frac{0.4472}{0.5} + 3.2\sqrt{20} * \left(\frac{0.4472}{0.5} - 1\right)^{\frac{3}{2}})$$

$\frac{l}{d}$ = 20.52*k

$\frac{5000}{d}$ = 20.52*1.5

d = $\frac{1400}{30.78}$ = 45.5mm

D = dmin + Cnom +φl/ 2 where dmin is governing effective depth.

D= 45.5+20+20/2=75.5mm

D=80 mm

Overall depth

D=d+cover+Ø/2, assume Ø14

dmax= max (112.46,75.5)= d=112.46mm

D=112.46mm+20+14/2=142.46mm

Use D=150mm

5.3 Load Determination

Table-5.1 Material date

Material	Unit weight(KN/m^3)	Thickness(cm)
Marble	27	3
Cement screed	23	2
Plastering	23	2
Concrete	24	-

Take 1m width strip

Step dead load

Table-5.2 Self-weight of stair with different floor finish on the Ground floor

PANEL	Material	Thickness(m)	Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Load (KN/m^2)
Step	Rc Slab	0.08	24	1.92	3.77
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	

Riser dead load

- D.L of cement screed= $\dfrac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (15*30 cm)}}$..............................Eqn(2)

$\dfrac{11(0.15m*0.02m*23KN/m^3}{4.5}$=0.169 KN/m

- D.L of Finishing = $\dfrac{\text{No of riser (hcs*tcs*}\gamma\text{sc)}}{\text{Projected length (15*30 cm)}}$

$\dfrac{11(0.16m*0.03m*27KN/m^3}{4.5}$=0.317 KN/m

Therefore D.L of riser (15cm)= 0.169 KN/m + 0.317 KN/m

=0.486 KN/m

Waist Dead Load

tanθ=1.65/3

$\theta = \tan^{-1}(\frac{1.65}{3})$

=28.81°

Sin θ=1.65/L_{inc}

$L_{inc\backslash}$=1.65/sin θ =1.65m/sin 28.81°

L_{inc} =3.5m

- D.L of concrete $= \frac{D*L\ inc*\gamma conc}{Lprojected} = \frac{0.15m*3.5m*24\ KN/m^3}{3\ m}$

=3.6 KN/m

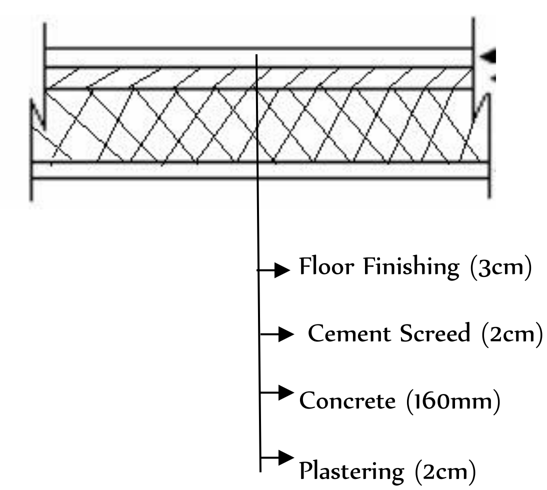
- D.L of Plastering $= \frac{tpl*Linc*\gamma pl}{Lprojected} = \frac{0.02m*3.5m*23\ KN/m^3}{3\ m}$

=0.536 KN/m

Therefore D.L of waist = 3.6 KN/m + 0.536 KN/m

=4.136KN/m

Landing Dead Load



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

=0.03m*27 KN/m^3+0.02*23 KN/m^3 + 0.15m*24 KN/m^3 +0.02m*23 KN/m^3

=5.33 KN/m

Therefore D.L of Landing=5.33 KN/m

Total Dead load and design load for incline slab

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

=3.6 KN/m+0.536 KN/m+4.136 KN/m

=8.272KN/m

Live load= 5 KN/m^2*1m=5 KN/m

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

=1.35*8.27 KN/m + 1.5*5 KN/m

=18.67 KN/m

For landing

D.L=5.33 KN/m

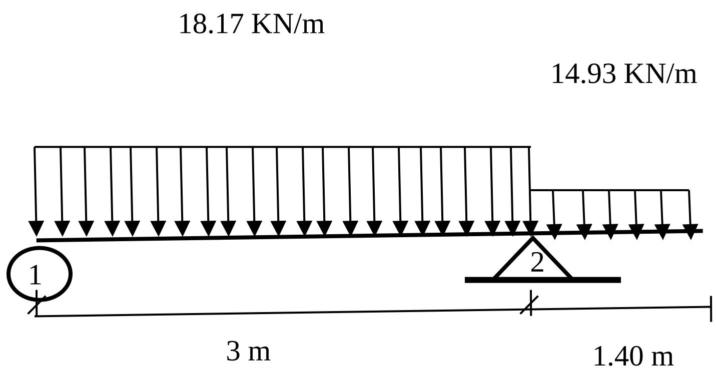
L.L=5 KN/m^2 * 1m=5KN/m

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

=1.35*5.33 KN/m + 1.5*5 KN/m

=14.93 KN/m

Loading



5.4 Moment analysis

+ΣM at 1=0

$18.17*3m*(3/2)+14.93*1.4*(3+1.4/2)-R_2*3m=0$

$R_2*3m=0$

$R_2*3m=159.10KNm$

$R_2=53.03KNm$

ΣFy=0 (↓+)

$R_1+R_2=18.17\ KN/m*3\ m+14.93\ KN/m*1.4m=75.41\ KN$

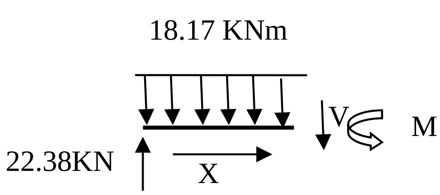
$R_1+53.03KN=75.41\ KN$

$R_1=75.41\ KN-53.03\ KN$

$R_1=22.38KN$

Analyzing using the method of section

For Z=0, to Z=0.83m (Z in measured from support 1)



$M(x)+18.17\ KN/m * x^2/2=22.38x$

$M(x)=22.38x-9.085\ x^2$

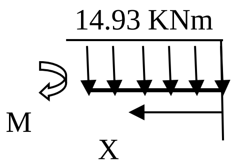
- At Z=0, X=0

$M(0)=0$

- At Z=3, X=3

$M(3)=-14.625KNm$

For Z=3m to Z=4.4m



$M(x)=-14.93\ x^2/2$

$=-7.465\ x^2$

At Z=4.4m, x=0

At Z=4.4m, x=1.4m

Mmax (+ve) is b/n Z=3 to Z=4.4m

M is max at V=0, dm(x)/dx=0

$dM(x)/dx=d/dx(22.38x-9.085\ x^2)=0$

$=-22.38+18.17x=0$, $x=1.467m$

$M(1.467m)=49.03KNm$

Shear force

$V(x)=dM(x)/dx$

For Z=0 to Z=3m

$M(x)=22.38x-9.085\ x^2$

$dM(x)/dx=V(x)=-18.17x+22.38$

$V(0)=22.38$ KN

- $V(3)=-32.13$ KN

  $M(x) =-7.465\, x^2$

  $dM(x)/dx= -14.93x$

  at x=4.4m,
- $V(1.15)= -32.84$KN

5.5 Check depth for flexure

M is M_{max} of all the stair moment

$$\geq \frac{\sqrt{M/b}}{K_m}$$

$M_{max}=49.03$ KNm

b=1m

Km=57.83 (without moment distribution)

d=150mm-cover-∅/2, assuming ∅=14mm

=150mm-20mm-14mm/2

=123mm

$$d\geq \frac{\sqrt{49.03KN.m/1m}}{57.83}=121.08mm$$

123mm≥121.08 mm OK!

5.6 Reinforcement Design

For the given

1. Material Data, C-25, S-400
2. Effective depth, d=123mm
3. Width, b=1000mm
4. Moments calculated for each panels using charts

As=Ks*M/d

5 To calculate spacing by selecting diameter of bar as

S=1000as/A whereas=area of single bar

As=calculated area of steel

S=spacing

6 Compare the above result with minimum provision given by our code.

$As_{min}=\rho_{min}*b*d=0.5/fyk*b*d=0.5*1000*128/300=213.33$

$S_{max} \leq \begin{cases} 3D \\ 400mm \end{cases} = \begin{cases} 3*150mm \\ 400mm \end{cases} = 450mm$

$S_{max}=450mm$

Development Length

$Lbnet=al_bAs_{cal}/As_{prov}$ a=0.7 for hook, a=1 for straight

$l_{b.}=Ø/4*fyd/fbd$

$fbd=2f_{ctd}$,for deformed bar.

$l_b=Ø/8*fyd/fbd$

$fyd=fyk/\gamma_s=400Mpa/1.15=347.8\ MPa$

$fctd=0.21(fck)^{2/3}/\gamma_c=0.21(30/1.25)^{2/3}/1.5$

=1.165 Mpa

$l_b=Ø/8*347.8\ Mpa/1.165\ MPa=Ø*298.54$

Table-5.3 reinforcement for stair

Panel	moment	Km	Ks	As	Asmin	Scalculated	Sprovided	Remark
Stair case-1	49.03	46.23	4.31	1197.38	213.33	94.46	90.00	ϕ12C/C90
	7.465	24.20	3.98	302.97	213.33	373.30	370	ϕ 12C/C370
Stair case-2	42.72	50.28	4.41	1449.19	213.33	78.04	70	ϕ 12C/C70
	23.45	37.25	4.12	743.18	213.33	152.18	150	ϕ 12C/C150
	10.44	24.85	3.98	319.62	213.33	353.85	300	ϕ 12C/C300

Lateral Load

Building structure are exposed to lateral loads of the earth quake and wind loads. The occurrence of these loads simultaneously on the structure is very rare and therefore, we will design the structure for the governing load among the two. Thus, the design process involves the determination of the two loads separately and designing for the maximum effect.

6.1 Earthquake analysis

Using, Equivalent static (building code) analysis method.

This type of analysis is applied to buildings whose response is not significantly affected by contribution from higher modes vibration. These requirements are claimed to be satisfied by buildings which is meets the criteria for regularity in plan and elevation and Have fundamental periods of vibration(ES EN 8, 2015)

$$T1 \leq \begin{cases} 4Tc \\ 2\ Sec \end{cases}$$

a) Center of mass

The horizontal forces at each floor level, Fi, are distributed to lateral load resistive structural elements in proportion to their rigidities assuming rigid floor diaphragms. Center of mass (X_m, Y_m): it is a point on a floor level where the whole floor mass and its inertial effects can be replaced using a lumped equivalent mass.

$$\mathrm{Xm} = \frac{\sum \mathrm{WiXi}}{\sum \mathrm{Wi}}, \ldots \text{ Eqn (1)}$$

$$\mathrm{Ym} = \frac{\sum \mathrm{WiYi}}{\sum \mathrm{Wi}}, \ldots \text{ Eqn (2)}$$

Where:

X_m, and Y_m are the coordinate of the point of application of Fi when the seismic action is parallel to the Y-direction and X-direction respectively.

For Axis-A2 W=0.6m, D=0.6m, H=2m and weight=0.6*0.6*2*25=18KN,

Moment on x direction multiply weight with x-direction moment army the same for y-direction after gating the result taking the summation on x and y direction.

Table-6.1 center of mass calculation for foundation

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.6	0.6	2	18	0	0	0	0
Axis-A3	0.6	0.6	2	18	0	6.4	115.2	0
Axis-A5	0.6	0.6	2	18	0	11.45	206.1	0
Axis-B2	0.6	0.6	2	18	5.4	0	0	97.2
Axis-B3	0.6	0.6	2	18	5.4	6.4	115.2	97.2
Axis-B5	0.6	0.6	2	18	5.4	11.45	206.1	97.2
Axis-C2	0.6	0.6	2	18	10.8	0	0	194.4
Axis-C3	0.6	0.6	2	18	10.8	6.4	115.2	194.4
Axis-C5	0.6	0.6	2	18	10.8	11.45	206.1	194.4
Axis-D2	0.6	0.6	2	18	17.2	0	0	309.6
Axis-D3	0.6	0.6	2	18	17.2	6.4	115.2	309.6
Axis-D5	0.6	0.6	2	18	17.2	111.45	2006.1	309.6
Total weight of Footing column				216			3085.2	1803.6
					Xm	8.35		
					Ym	14.28		

The center of mass calculation from ground floor up-to fourth floor is presented on the Appendix D, D-1

Table-6.2 center of mass calculation for beam

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.35	0.4	11.45	40.08	0	11.45	458.86	0
Axis B	2--5	0.35	0.4	11.45	40.08	5.4	11.45	458.86	216.4
Axis C	2--5	0.35	0.4	11.45	40.08	10.8	11.45	458.86	432.8
Axis D	2--5	0.35	0.4	11.45	40.08	17.2	11.45	458.86	689.3
Axis 2	A-D	0.35	0.4	17.2	60.20	17.2	0	0.00	1035.4
Axis 3	A-D	0.35	0.4	17.2	60.20	17.2	6.4	385.28	1035.4
Axis 5	A-D	0.35	0.4	17.2	60.20	17.2	11.45	689.29	1035.4
					340.9			2910.01	4444.83
						Xm	13.04		
						Ym	8.54		

The center of mass calculation of the other beam presented on Appendix D, D-1

Center of mass for slab

Table-6.3 center of mass for ground floor

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	5.4	6.4	7.22	249.5232	5.4	6.4	1596.948	1347.425
P2	5.4	6.4	10.7	369.792	10.8	6.4	2366.669	3993.754
P3	6.4	6.4	9.95	407.552	17.2	6.4	2608.333	7009.894
P4	5.05	6.4	10.99	355.1968	17.2	11.45	4067.003	6109.385
P5	2.05	5.4	8.03	88.8921	10.8	8.45	751.1382	960.0347
P6	5.05	5.4	9.33	254.4291	5.4	11.45	2913.213	1373.917
C1	2.5	5.4	9.26	125.01	5.4	-2.5	-312.525	675.054
C2	2.5	5.4	7.61	102.735	10.8	-2.5	-256.838	1109.538
C3	2.5	8.9	9.34	207.815	19.7	-2.5	-519.538	4093.956
C4	2.5	6.4	7.61	121.76	19.7	6.4	779.264	2398.672
C5	2.5	5.05	8.48	107.06	19.7	11.45	1225.837	2109.082
C6	2.5	5.4	7.22	97.47	10.4	14	1364.58	1013.688
C7	2.5	13.95	7.61	265.3988	-2.5	11.45	3038.816	-663.497
				2752.634			19622.9	31530.9
			Xm	11.45481				
			Ym	7.128773				

The overall center of mass is presented on Appendix D, D-3

Center of mass for wall

Table -6.4 center of mass ground floor wall

partition Wall						moment arm		moment	
wall on	b/n Axis	width	depth	length	weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	3&2	0.2	3	5.4	45.36	5.4	11.45	519.372	244.944
	2&1	0.2	3	1.9	15.96	5.4	6.4	102.144	86.184
Axis C	3&2	0.2	3	5.4	45.36	10.8	11.45	519.372	489.888
	1&2	0.2	3	1.9	15.96	10.8	6.4	102.144	172.368
Axis D	3&2	0.15	3	3	18.9	17.2	11.45	216.405	325.08
Axis A-B	3&2	0.15	3	2.1	13.23	2.5	11.45	151.4835	33.075
Axis C-D	3&2	0.15	3	3.3	20.79	13.6	11.45	238.0455	282.744
Axis 2-3	C&D	0.2	3	5.02	42.168	13.8	8.6	362.6448	581.9184
	C&D	0.2	3	3	25.2	13.4	8.6	216.72	337.68
	C&D	0.15	3	2.26	14.238	12.6	8.4	119.5992	179.3988
	A&B	0.15	3	3.05	19.215	3.05	7.85	150.8378	58.60575
Axis B	Cantilever	0.2	3	1.6	13.44	5.4	14	188.16	72.576

	Cantilever	0.2	3	1.6	13.44	5.4	-2.5	-33.6	72.576
Axis C	Cantilever	0.2	3	1.6	13.44	10.8	14	188.16	145.152
Axis 2	Cantilever	0.2	3	2.5	21	-2.5	6.4	134.4	-52.5
	Cantilever	0.2	3	2.4	20.16	19.7	6.4	129.024	397.152
Axis 2-3	Cantilever	0.2	3	2.5	21	-2.5	11.45	240.45	-52.5
Axis 3	Cantilever	0.2	3	2.5	21	19.7	11.45	240.45	413.7
	Cantilever	0.2	3	2.4	20.16	-2.5	11.45	230.832	-50.4
Axis A*	Cantilever	0.2	3	5.4	45.36	-2.5	11.45	519.372	-113.4
Axis D*	Cantilever	0.2	3	5.4	45.36	19.7	11.45	519.372	893.592
					510.741			5055.388	4517.834

The overall wall on all floor presented on Appendix D, D-4

Base shear determination

$F_b=S_d(T1)*m*\lambda$.................................Eqn (3)

Where

Sd (T1)- is the ordinate of the design spectrum at period T1

m- The total mass of the building, above the foundation

λ -is the correction factor the value of which is equal to: $\lambda=0.85$ if $T1\leq 2Tc$ and the building has more than two stores or $\lambda=1.0$ otherwise $T1 = Ct*H^{3/4}$

For concrete moment resisting frame Ct=0.075, H=14.65m

$T1=0.075*18.6^{0.75}=0.67sec$ (Appendix D, D-2)

Table-6.5 Total weight

Story	Remark	Floors	weight(KN)
GR	W1	Foundation + basement	441
1	W2	ground floor	3994.715
2	W3	1st floor	4128.532
3	W4	2nd floor	4333.191
4	W5	3rd floor	4320.029
5	W6	4th floor	4184.96
6	W7	Roof floor	3148.485
7	W8	WATER TANK	283.783
		sum of weight	24834.695

For the horizontal component of the seismic action the design spectrum, Sd(T) Shall be define by the following expressions

$0 \leq T \leq T_B$:Sd(T)=ag*S*$\frac{2}{3}+\frac{T}{TB}*(\frac{2.5}{q}-\frac{2}{3})$............................Eqn (4)

$T_B \leq T \leq T_c$: Sd(T)=ag*S*$\frac{2.5}{q}$

$T_D \leq T$:Sd(T)=$\begin{cases} ag*S*\frac{2.5}{q}*\frac{TcTd}{T2} \\ \geq \beta * ag \end{cases}$

Where;

q-is the behavior factor

β- is the lower bound factor for the horizontal design

For frame system and DCM, qo =3 α u/α1

For buildings which are not regular in elevation, the value of qo should be reduced by 20%

qo = 3 α u/α1 - 0.2*3 α u/α1 = 0.8 * 3 α u/α1

α1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

αu is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor αu may be obtained from a nonlinear static (pushover) global analysis.

When the multiplication factor αu/α1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate value of αu/α1 may be used.

Frames or frame-equivalent dual systems.

i. One-story buildings: αu/α1=1,1;
ii. Multistory, one-bay frames: αu/α1=1,2;
iii. Multistory, multi-bay frames or frame-equivalent dual structures: αu/α1=1,3

In our case is multistory so using αu/α1=1,3.

For buildings which are not regular in plan (see 4.2.3.2), the approximate value of αu/α1 that may be not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in (5) of this subclasses.

The factor kw reflecting the prevailing failure mode in structural systems with walls shall be 1.0 for frame and frame equivalent dual system.(ES EN 8, 2015)

$q=q_oK_w \geq 1.5$

$K_w = 1$ $\quad\quad\quad\quad q=0.8*3*1.2*1=2.88$

$$Sd(T) = \max\begin{cases} 0.981 * 1 * \left(\frac{2.5}{2.88}\right) = 0.3177 \\ 0.2 * 0.981 = 0.1963 \end{cases}$$

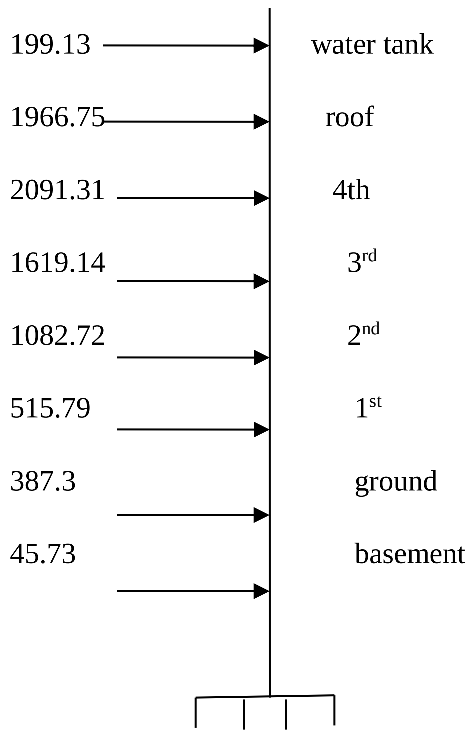
Sd(T)=0.3177

$F_b=S_b(T1)*W_{tot}=0.3177*24834.783=7890.01KN$

Computation of story shear force (Fi)

Table-6.6 Weight summary according to story level

Level	Remark	Wi	hi	Wi*hi	Fb	Ft	Fb-Ft	Fi
Basement	W1	441	2.7	1190.7	7890.01	370.04	7519.97	45.078151
Ground floor	W2	3994.715	0	0	7890.01	370.04	7519.97	0
First floor	W3	4128.532	3.3	13624.16	7890.01	370.04	7519.97	515.7905
Second floor	W4	4333.191	6.6	28599.06	7890.01	370.04	7519.97	1082.7184
Third floor	W5	4320.029	9.9	42768.29	7890.01	370.04	7519.97	1619.1445
Fourth floor	W6	4184.96	13.2	55241.47	7890.01	370.04	7519.97	2091.3609
Roof floor	W7	3148.485	16.5	51950	7890.01	370.04	7519.97	1966.7507
water tank	W8	282.783	18.6	5259.764	7890.01	370.04	7519.97	199.12692
				198633.4				7519.97


199.13
water tank
1966.75
roof
2091.31
4th
1619.14
3rd
1082.72
2nd
515.79
1st
387.3
ground
45.73
basement

Frame analysis

Our fame was analyzed using ETABS 2016. The procedure for modeling the frame will be as the following;

Step 1- Plot grid coordinates that represent the given structural layout.

Step 2- Our building is made of reinforced concrete structures. Therefore, the materials which we define are concrete & steel rebar.

Step 3- define load and frame section

Step 4- draw the structure

Step 5- assigning loads; Then we assigned Fi (story shear) as a joint load on each frame joint and we analyze the 3D frame separately for each EQ coming from x and y direction. Finally, we have assigned all the transferred loads which are the dead and live load from slab and wall to the beams.

Step 6- Analysis; finally run the model and check for any errors.

Table 7.1 for comb from ETABS.

load combination					
comb 1	1.35DL	1.5LL	linear add		linear add
comb 2	DL	LL			linear add
comb 3	DL	0.6LL	EQXL	0.3EQYL	linear add
comb 4	DL	0.6LL	EQXR	0.3EQYL	linear add
comb 5	DL	0.6LL	-EQXL	0.3EQYL	linear add
comb 6	DL	0.6LL	-EQXR	0.3EQYL	linear add
comb 7	DL	0.6LL	EQXL	0.3EQYR	linear add
comb 8	DL	0.6LL	EQXR	0.3EQYR	linear add
comb 9	DL	0.6LL	-EQXL	0.3EQYR	linear add
comb 10	DL	0.6LL	-EQXR	0.3EQYR	linear add
comb 11	DL	0.6LL	EQXL	-0.3EQYL	linear add
comb 12	DL	0.6LL	EQXR	-0.3EQYL	linear add
comb 13	DL	0.6LL	-EQXL	-0.3EQYL	linear add
comb 14	DL	0.6LL	-EQXR	-0.3EQYL	linear add
comb 15	DL	0.6LL	EQXL	-0.3EQYR	linear add
comb 16	DL	0.6LL	EQXR	-0.3EQYR	linear add
comb 17	DL	0.6LL	-EQXL	-0.3EQYR	linear add

comb 18	DL	0.6LL	-EQXR	-0.3EQYR	linear add
comb 19	DL	0.6LL	EQYL	0.3EQXL	linear add
comb 20	DL	0.6LL	EQYR	0.3EQXL	linear add
comb 21	DL	0.6LL	-EQYL	0.3EQXL	linear add
comb 22	DL	0.6LL	-EQYR	0.3EQXL	linear add
comb 23	DL	0.6LL	EQYL	0.3EQXR	linear add
comb 24	DL	0.6LL	EQYR	0.3EQXR	linear add
comb 25	DL	0.6LL	-EQYL	0.3EQXR	linear add
comb 26	DL	0.6LL	-EQYR	0.3EQXR	linear add
comb 27	DL	0.6LL	EQYL	-0.3EQXL	linear add
comb 28	DL	0.6LL	EQYR	-0.3EQXL	linear add
comb 29	DL	0.6LL	-EQYL	-0.3EQXL	linear add
comb 30	DL	0.6LL	-EQYR	-0.3EQXL	linear add
comb 31	DL	0.6LL	EQYL	-0.3EQXR	linear add
comb 32	DL	0.6LL	EQYR	-0.3EQXR	linear add
comb 33	DL	0.6LL	-EQYL	-0.3EQXR	linear add
comb 34	DL	0.6LL	-EQYR	-0.3EQXR	linear add
comb 35	1.35DL	1.5LL	0.9WL		linear add
comb 36	1.35DL	1.5LL	-0.9WL		linear add
comb 37	1.35DL	1.5WL			linear add
comb 38	comb 3-18				envelope
comb 39	comb 19-34				envelope
comb 40	comb 1-39				envelope

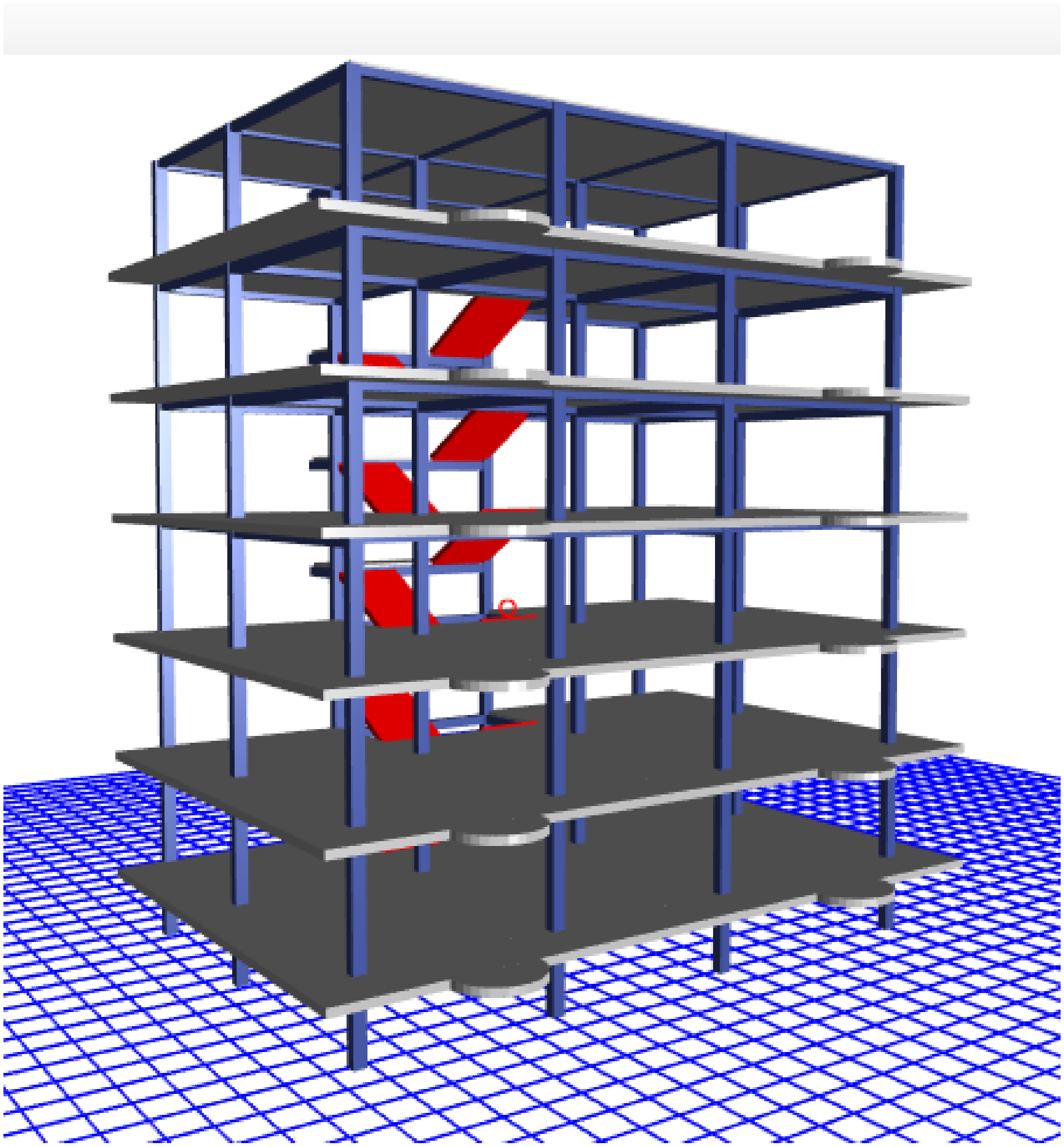


Figure-7.1 the model of the structure from ETABs out put

Design of beam

8.1 General

Beam is a horizontal structural member which offers resistance to 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.

Table-8.1 Types of beams

Types of Beam		
According to support condition	According to amount of reinforcement	According to location of reinforcement
Simply supported	Balanced section	Singly reinforced section
Cantilever	Under reinforced section	Doubly reinforced section
Propped	Over reinforced section	
Overhanging		
continuous		

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

If a section of a beam is limited in depth, it cannot develop the compressive force required to resist the applied bending moment. If it’s a small increase in moment, over reinforced beam section can be used (not recommended in design).

In such case, providing 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.

8.2 Flexural Design

Step 1: Design Data

Dimension: 300x400mm

Material used

A. Concrete

Grade of concrete C-25

Concrete cube strength, f_{cu} =25MPa

Concrete characteristic strength, $f_{cj}=0.8*f_{cu}=0.8*25=20MPa$

B. Steel S-400

Steel tensile strength,, f_{yk} =400MPa

Steel tensile design strength, $f_{yd}=f_{yk}/\gamma s=400/1.15= 347.83MPa$

C. Concrete cover

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

$C_{nom}=C_{min} + \Delta C_{dev}$.. Eqn (1)

Where, c_{min}—minimum concrete cover

ΔC_{dev}—allowance in design for deviation

Minimum concrete cover shall be provided in order to ensure:

i. The safe transmission of bond forces
ii. An adequate fire resistance
iii. The protection of the steel against corrosion (durability)

The greater value for Cmin satisfying the requirements for both bond and environmental conditions shall be used.

C_{min}= max {Cmin,b ;$C_{min,dur}+ \Delta C_{dur,\gamma}- \Delta C_{dur,st}- \Delta C_{dur,add}$;10mm}

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

Table-8.2 Minimum cover due to bond requirement

Bond Requirement	
Arrangement of bars	Minimum cover $C_{min,b}$
Separated	Diameter of bar
Bundled	Equivalent diameter
If the nominal maximum aggregate size is greater than 32 mm, $C_{min,b,}$ should be increased by 5mm	

Therefore, As the bars are arranged separately, the minimum cover due to bond requirement, C_{min}, b is 20mm (diameter of the bar).

Cmin,dur minimum cover due to environmental conditions

Table-8.3 Minimum bond due to environmental condition

Environmental Requirement for Cmin,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

Therefore, As the structural class is S1 & Exposure class is X0, the minimum cover due to environmental requirement, $C_{min,dur}$ is 10mm.

- ΔCdur, γ additive safety element

The concrete cover should be increased by the additive safety element ΔCdur, γ

Note: The value of ΔCdur, γ for use in a Country may be found in its National Annex. The recommended value is 0mm.

Therefore, the concrete cover for safety element, ΔCdur,γ is 0mm.

- ΔCdur,st reduction of minimum cover for use of stainless steel

Where stainless steel is used or where other special measures have been taken, the minimum cover may be reduced by $\Delta C_{dur,\gamma}$. For such situations the effects on all relevant material properties should be considered, including bond.

Note: The recommended value $\Delta C_{dur,st.}$, without further specification, is 0mm.

Therefore, the reduction of minimum cover for use of stainless steel, $\Delta C_{dur,st}$ is 0mm

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

For concrete with additional protection (e.g. coating) the minimum cover may be reduced by $\Delta C_{dur,add}$

Note: The recommended value, for $\Delta C_{dur,add}$ without further specification, is 0 mm.

Therefore, the reduction of minimum cover for use of additional protection, $\Delta C_{dur,add}$ is 0mm.

Finally, $C_{min}=\max\{C_{min,b};\ C_{min,dur}+\Delta C_{dur,\gamma}-\Delta c_{dur,st}-\Delta c_{dur,add};10mm\}$

$C_{min}=\max\{20;\ 10+0-0-0;10mm\}=20mm$

8.3 Allowance in design for deviation

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

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

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

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

- Where fabrication is subjected to a quality assurance system, in which the monitoring includes measurements of the concrete cover, the allowance in design for deviation ΔC_{dev} may be reduced

  $10mm \geq \Delta C_{dev} \geq 5mm$

- ✦ Where it can be assured that a very accurate measurement device is used for monitoring and nonconforming members are rejected (e.g. precast elements), the allowance in design for deviation ΔC_{dev} may be reduced:

  $10mm \geq \Delta C_{dev} \geq 0mm$

Therefore, we have taken $\Delta C_{dev} = 5mm$

$C_{nom}=C_{min}+ \Delta C_{dev}=20mm+5mm=25mm$

Step 2: Check depth for deflection

- ✓ $\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}(\frac{\rho_o}{\rho} - 1)^{3/2}\right]$ $if\ \rho \leq \rho_o$
- ✓ $\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_o}}\right]$ $if\ \rho > \rho_o$

Taking L/d=26 for end span L/d = 30 for interior span from ES EN 1992:2015 9 table 7.4N

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

Therefore,

End span, 26*1.25=32.5

Interior span, 30*1.25=37.5

Cantilever, 8*1.25=10

End span,d=L/32.5=6000mm/32.5=184.61mm

Interior span,d=L/37.5=6000mm/37.5=160mm

$D_{provided}=D_{provided}$+ stirrup+ diameter of bar/2 =184.61mm+8mm+20/2 =202.61mm

Therefore $D_{provided}$=400mm≥ $D_{required}$=202.61mm……Ok!

Step 3: Check whether the beam is singly or doubly reinforced.

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

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

Where K=$\frac{Msd}{bd^{2(fck)}}$

Step 4: Provide reinforcement

Z =$\frac{d}{2}\left(1+\sqrt{1-3.53K}\right) \leq 0.95d$

As_{calc}=$\frac{Msd}{f_{yd}\,Z}$=

Step 5: Check for minimum & maximum reinforcement area

Asmin= 0.0013bd

As, max=0.04bd

Number of steel, n

$a_s = \frac{\pi D^2}{4}$ = 3.14*20^2/4 =314mm^2

n= $\frac{As,calc}{as}$

Bending moment diagram (M2-3 on axis A)

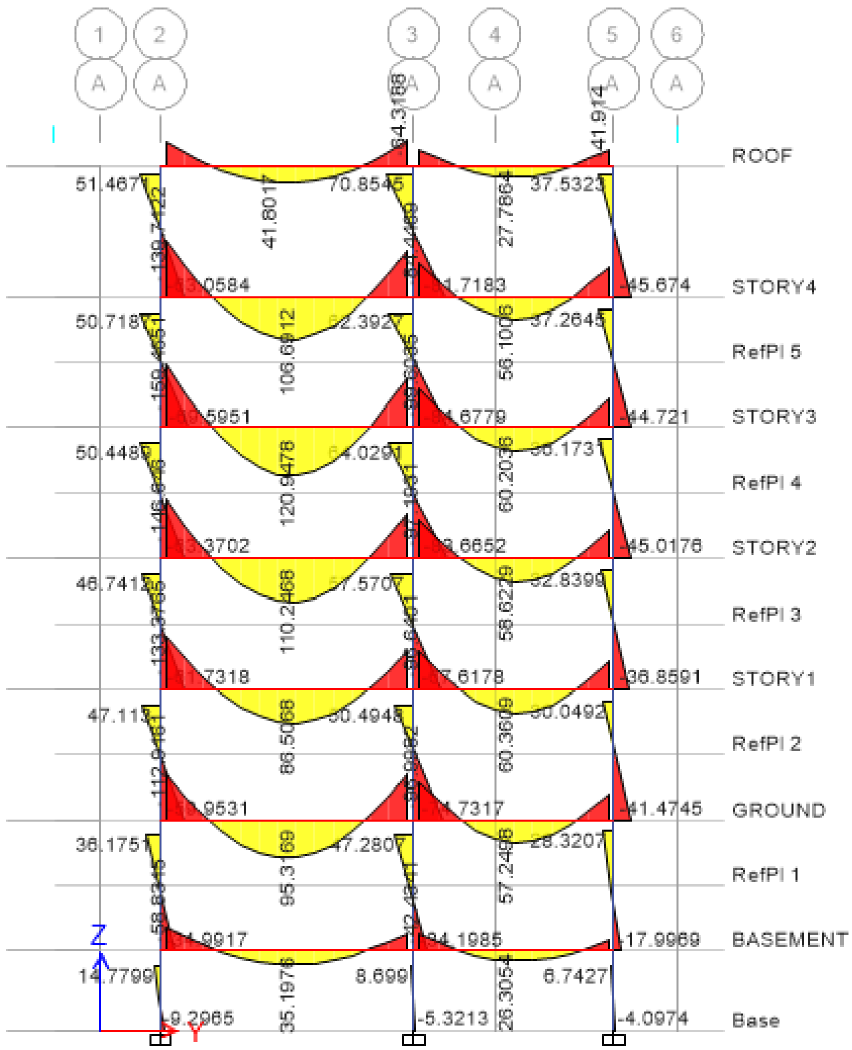


Figure-8.1 bending moment diagram of beam in axis A

Table-8.4 reinforcement of beam on axis A

Axis A		Roof											
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	Asmin (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	51.48	300	375	0.061	353.5855	Single	146.25	4500	418.579	418.5786	1.333053	2Φ 20
Span	2--3	41.802	300	375	0.050	357.817	Single	146.25	4500	335.868	335.8684	1.069644	2Φ 20
Support	3	64.31	300	375	0.076	347.8038	Single	146.25	4500	531.59	531.5904	1.692963	2Φ 20
Span	3--5	27.786	300	375	0.033	363.7651	Single	146.25	4500	219.603	219.6028	0.699372	2Φ 20
Support	5	41.914	300	375	0.050	357.7686	Single	146.25	4500	336.814	336.8138	1.072655	2Φ 20

Axis A	STORY 4												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	139.74	300	375	0.166	308.3421	Single	146.25	4500	1302.93	1302.929	4.149456	5Φ 20
Span	2--3	106.69	300	375	0.126	327.0132	Single	146.25	4500	937.975	937.9753	2.987183	3Φ 20
Support	3	64.44	300	375	0.076	347.7442	Single	146.25	4500	532.756	532.7563	1.696676	2Φ 20
Span	3--5	56.1	300	375	0.066	351.5271	Single	146.25	4500	458.814	458.8144	1.461192	2Φ 20
Support	5	45.67	300	375	0.054	356.1385	Single	146.25	4500	368.676	368.6761	1.174128	2Φ 20

Axis A	STORY 3												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	159.67	300	375	0.189	295.5345	Double	146.25	4500	1553.27	1553.274	4.946733	5Φ 20
Span	2--3	120.94	300	375	0.143	319.2877	Single	146.25	4500	1088.98	1088.982	3.468096	4Φ 20
Support	3	99.6	300	375	0.118	330.7018	Single	146.25	4500	865.876	865.8761	2.757567	3Φ 20
Span	3--5	60.2	300	375	0.071	349.6784	Single	146.25	4500	494.949	494.9492	1.576271	2Φ 20
Support	5	44.72	300	375	0.053	356.5523	Single	146.25	4500	360.588	360.5882	1.14837	2Φ 20

Axis A	STORY 2												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov(mm2)	No.of bar	Remark
Support	2	146.67	300	375	0.174	304.0484	Double	146.25	4500	1386.86	1386.856	4.41674	5Φ 20
Span	2--3	110.24	300	375	0.131	325.1292	Single	146.25	4500	974.802	974.8017	3.104464	4Φ 20
Support	3	99.6	300	375	0.118	330.7018	Single	146.25	4500	865.876	865.8761	2.757567	3Φ 20
Span	3--5	58.623	300	375	0.069	350.3919	Single	146.25	4500	481.002	481.002	1.531854	2Φ 20
Support	5	45.018	300	375	0.053	356.4226	Single	146.25	4500	363.123	363.1231	1.156443	2Φ 20

Axis A	STORY 1												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	133.37	300	375	0.158	312.1585	Single	146.25	4500	1228.33	1228.332	3.911887	4Φ 20
Span	2--3	86.51	300	375	0.103	337.2734	Single	146.25	4500	737.424	737.4239	2.348484	3Φ 20
Support	3	99.6	300	375	0.118	330.7018	Single	146.25	4500	865.876	865.8761	2.757567	3Φ 20
Span	3--5	60.36	300	375	0.072	349.6058	Single	146.25	4500	496.368	496.3677	1.580789	2Φ 20
Support	5	36.86	300	375	0.044	359.9377	Single	146.25	4500	294.416	294.4157	0.93763	2Φ 20

Axis A	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	112.95	300	375	0.134	323.6734	Single	146.25	4500	1003.26	1003.257	3.195086	4Φ 20

Span	2--3	95.32	300	375	0.113	332.8832	Single	146.25	4500	823.238	823.2375	2.621776	3Φ 20
Support	3	96.64	300	375	0.115	332.2139	Single	146.25	4500	836.319	836.3192	2.663437	3Φ 20
Span	3--5	57.24	300	375	0.068	351.0151	Single	146.25	4500	468.821	468.8207	1.493059	2Φ 20
Support	5	41.47	300	375	0.049	357.9603	Single	146.25	4500	333.067	333.0674	1.060724	2Φ 20

Axis A	BASEMENT												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	2	58.83	300	375	0.070	350.2985	Single	146.25	4500	482.829	482.8293	1.537673	2Φ 20
Span	2--3	35.198	300	375	0.042	360.6451	Single	146.25	4500	280.589	280.5892	0.893596	2Φ 20
Support	3	42.43	300	375	0.050	357.5456	Single	146.25	4500	341.173	341.1729	1.086538	2Φ 20
Span	3--5	26.31	300	375	0.031	364.3799	Single	146.25	4500	207.587	207.5867	0.661104	2Φ 20
Support	5	17.997	300	375	0.021	367.8031	Single	146.25	4500	140.675	146.25	0.465764	2Φ 20

8.4 Shear Design

The concrete itself can resist shear by a combination of un-cracked concrete in the compression zone. The dowelling action of the bending reinforcement and aggregate interlocking across tension crack but, because concrete is weak in tension, the shear reinforcement is designed to resist all the tensile stress caused by the shear force.

Even where the shear force is small near the Centre span of beam a minimum amount of reinforcement in the form of links must be provided in order to form a cage supporting the longitudinal reinforcement and to resist any tensile stresses due to factors such as thermal movement and shrinkage of concrete.

Behavior of beams without web reinforcement is that the forces transferring shear across an inclined crack in a beam without web reinforcements are illustrated in Figure below.

Shear is transferred across line A-B-C by Vcy, the shear in the compression zone, by Vay, the vertical component of the shear transferred across the crack by interlock of the aggregate articles on the two faces of the crack, and by Vd, the dowel action of the longitudinal reinforcement. Immediately after inclined cracking, as much as 40 to 60 percent of the total shear is carried by Vd and Vay together.

Behavior of beams with web reinforcement

Inclined cracking causes the shear strength of beams to drop below the flexural capacity. The purpose of web reinforcement is to ensure that the full flexural capacity can be developed. Prior to incline cracking, the strain in the web reinforcement is equal to the corresponding strain of the concrete. Because concrete cracks at a very small strain, the stress in the web reinforcements prior to inclined cracking will not exceed 3 to 6 ksi. Thus, web reinforcements do not prevent inclined cracks from forming; they come into play after the cracks have formed.

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

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

Regions of the member where VEd ≤ VRd,c, no calculated shear reinforcement is necessary. VEd is the design shear force in the section considered resulting from external loading and pre-stressing (bonded or un-bonded). When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should be provided according to 9.2.2.

In regions where VEd> VRd,c, sufficient shear reinforcement should be provided in order that VEd ≤ VRd

Concrete capacity

The design value for the shear resistance VRd, c is given by:

$$V_{Rd,c} = \left[C_{Rd,c}K(100\rho 1 fck)^{1/3} + K1\delta_{cp}\right]bd$$

With minimum of VRd,c=(Vmin+K1δcp)bd

Where;

fck = 20MPa

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

ρ1=$\frac{As1}{bd}$

$\delta cp = \frac{NEd}{Ac}$

Ned: is the axial force in the cross-section due to loading or pre-stressing [in N] (NEd> 0 for compression). The influence of imposed deformations on NEd may be ignored.

Ac: is the area of concrete cross section [mm2].

VRd,c is [N]

$$C_{Rd,c} = \frac{0.18}{\gamma c}$$

K1=0.15

VRd,c=[CRd,c K(100ρ1fck)^(1⁄3)+K1 δcp]bd

Diagonal compression check of concrete

Where v is a strength reduction factor for concrete cracked in shear.

V=0.6 for fck ≤60MPa from ESEN 1992-1:2015

VRd,max=αcwbwzv1fcd$(\frac{1}{\cot\theta+\tan\theta}) =$

Where:

αcw=1 and cot θ =2.5

bw=

z=0.9d

fcd=11.33MPa

v1=0.6(1-$\frac{fck}{250}$)=

VRd,max= α_{cw}bwZv1fcd$(\frac{1}{\cot\theta+\tan\theta})$=

Calculate the required shear reinforcement

The ratio of shear reinforcement is given by;

ρw= $Asw/(S * bw - sin\,\alpha)$

Where:

ρw is the shear reinforcement ratio ρw should not be less than ρw,min

Asw 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

bw is the breadth of the web of the member

α is the angle between shear reinforcement and the longitudinal axis.

ρw,min =$\frac{0.08\sqrt{fck}}{fyk}$

The maximum longitudinal spacing between shear assemblies should not exceed Sl,max.

Slmax=0.75d (1+cot α)

Where

α is the inclination of the shear reinforcement to the longitudinal axis of the beam.

.The maximum longitudinal spacing of bent-up bars should not exceed Sb,max.

Sb,max=0.6d(1+cot α)

The transverse spacing of the legs in a series of shear links should not exceed St,max

St,max=0.75d

Smin=$\frac{Asw}{\rho wbwsin\alpha}$

Required shear reinforcement

S,calc=$\frac{Asw*0.78*d*fyk*cot\theta}{VEd}$

8.5 Anchorage of longitudinal reinforcement

Reinforcing bars, wires or welded mesh fabrics shall be so anchored that the bond forces are safely transmitted to the concrete avoiding longitudinal cracking or spelling. Transverse reinforcement shall be provided if necessary.

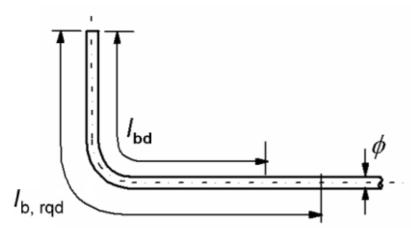


Figure 8.2 Basic tension anchorage length, lb, rqd, for any shape measured along the centerline

lb,rqd = (Ø/ 4) (σsd / fbd)

For bent bars the basic required anchorage length, lb,rqd, and the design length, lbd, should be measured along the center-line of the bar. The calculation of the required anchorage length shall take into consideration the type of steel and bond properties of the bars. The ultimate bond strength shall be sufficient to prevent bond failure.

The design value of the ultimate bond stress, fbd, may be taken as:

fbd = 2.25 η1 η 2 fctd

where:

fctd is the design value of concrete tensile strength,

fctd=fctk/1.5=1.0315

η 1 is a coefficient related to the quality of the bond condition and the position of the bar during concreting.

η1=1 when “good” condition is obtained and

η1= 0.7 for all other cases and for bars in structural elements built with slip forms, unless i b h w h ‘g ’ b conditions exist.

Therefore, η1= 1.0

η_2 is related to the bar diameter:

$\eta 2 = 1.0$ for $Ø<30mm$

$\eta 2 = (132 - Ø)/100$ for $Ø > 32$ mm

Therefore, $\eta 1 = 1.0$ since $Ø = 20mm \leq 32mm$

$fbd = 2.25 \eta 1 \eta 2 fctd = 2.25x1.0x1.0x1.0315 = 2.32MPa$

$lb,rqd = (Ø / 4) (\sigma sd / fbd) = (20/4)(347.83/2.32) = 749.6mm$

The design anchorage length, lbd, is:

$lbd = \alpha 1 \alpha 2 \alpha 3 \alpha 4 \alpha 5 lb,rqd \geq lb,min$

where

α 1, α 2 , α 3 , α 4 and α 5 are coefficients given in table

α 1 is for the effect of the form of the bars assuming adequate cover.

α 2 is for the effect of concrete minimum cover.

α 3 is for the effect of confinement by transverse reinforcement.

α 4 is for the influence of one or more welded transverse bars ($Ø t > 0.6 Ø$) along the design anchorage length lbd.

α 5 is for the effect of the pressure transverse to the plane of splitting along the design anchorage length.

The product $(\alpha 2 \alpha 3 \alpha 5) \geq 0.7$

lb,rqd is basic anchorage length, obtained earlier.

lb,min is the minimum anchorage length if no other limitation is applied:

for anchorages in tension: $lb,min \geq max\{0.3lb,rqd ; 10 Ø; 100 mm\}$

for anchorages in compression: $lb,min \geq max\{0.6lb,rqd ; 10 Ø; 100 mm\}$

Table-8.5 value of α 1, α 2, α 3, α 4 and 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	-

lbd = α 1 α 2 α 3 α 4 α 5 lb,rqd = 1x0.7x0.7x0.7x0.7x749.6 = 180mm

But, lbd ≥ lb,min

For anchorages in tension:

lb,min ≥ max{0.3lb,rqd ; 10 Ø; 100 mm}

lb,min ≥ max{0.3x749.6=224.89mm ; 10 =200mm; 100 mm}

lb,min ≥ max{225mm; 200mm; 100 mm}

lb,min ≥ 225mm

For anchorages in compression:

lb,min ≥ max{0.3lb,rqd ; 10 Ø; 100 mm}

lb,min ≥ max{0.3x749.6=224.89mm ; 10 Ø =200mm; 100 mm}

lb,min ≥ max{225mm; 200mm; 100 mm}

lb,min ≥ 225mm

The design anchorage length, lbd, is:

For tension, lbd =230mm

For compression, lbd =230mm

Anchorage of bottom reinforcement at intermediate supports. The anchorage length should not be less than 10 Ø (for straight bars) or not less than the diameter of the mandrel (for hooks and bends with bar diameters at least equal to 16 mm) or twice the diameter of the mandrel. This reinforcement should be continuous which may be achieved by means of lapped bars.

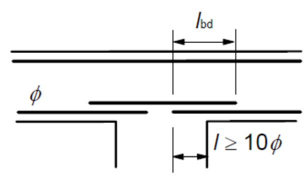


Figure-8.3 Anchorage at intermediate supports

Forces are transmitted from one bar to another by: Lapping of bars, with or without bends or hooks; Welding; Mechanical devices assuring load transfer in tension-compression or in compression only.

The detailing of laps between bars shall be such that: The transmission of the forces from one bar to the next is assured; Spelling 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 Figure

- The clear distance between lapped bars should not be greater than 4 Ø or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4 Ø or 50 mm;

- The longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length,l0;
- In case of adjacent laps, the clear distance between adjacent bars should not be less than 2Ø or 20 mm.

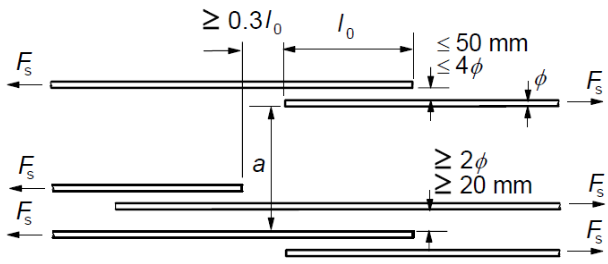


Figure 8.4 Adjacent laps

Lap Length;- The design lap length is:

lo = α 1 α 2 α 3 α 5 α 6 lb,rqd ≥ l_o,min

Where:

lb,rqd is basic anchorage length

lo,min ≥ max {0.3 α 6 lb,rqd; 15 Ø; 200 mm}

Values of α 1 α 2 α 3 and α 5 may be taken from table;

α 6 = Values of α 6 are given in Table 8.3.

Table-8.6 Values of the coefficient α 6

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	> 50%
α 6	1	1.15	1.4	1.5
Note: Intermediate values may be determined by interpolation.				

l_o = α 1 α 2 α 3 α 5 α 6 lb,rqd = 1x0.7x0.7x0.7x1.4x749.6 = 359.86mm ≥ l0,min

$l_{o,min} \geq$ max {0.3 α 6 lb,rqd; 15 Ø; 200 mm}

lo,min ≥ max {0.3x1.4x749.6=314.83mm; 15x20=300mm; 200 mm}

Therefore, Take l0 = 360mm

The beam for all axis done on (Appendix B)

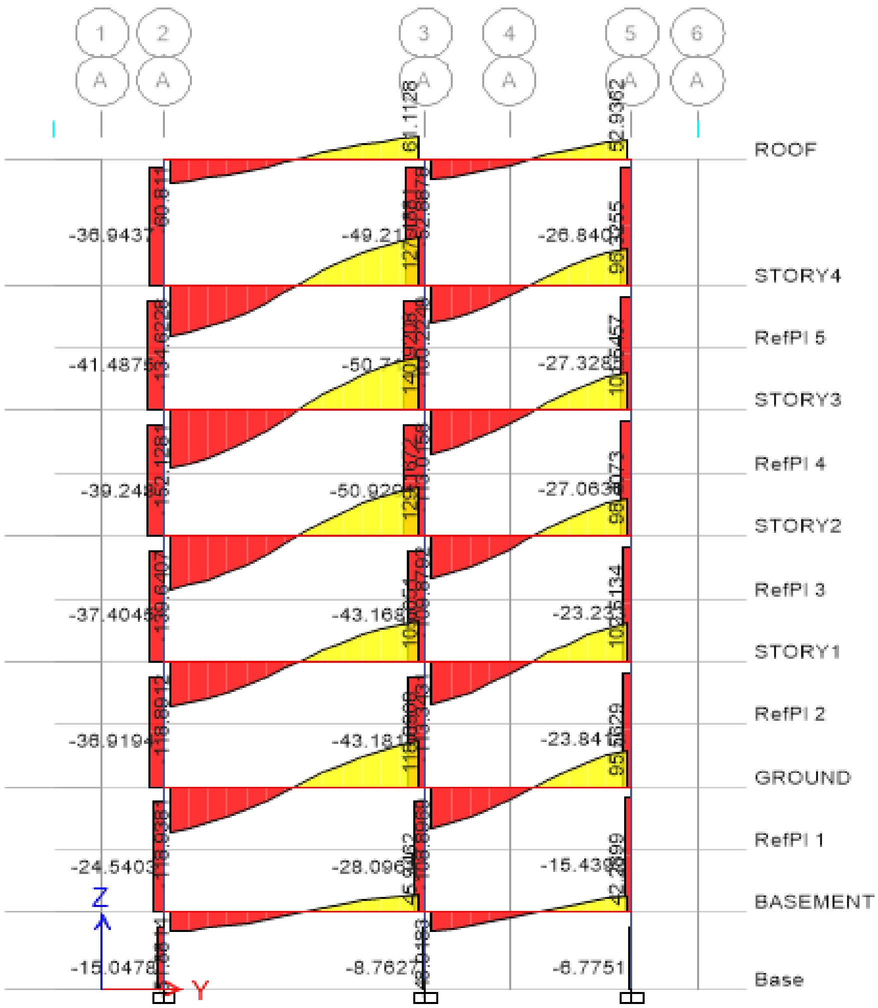


Figure 8.5 shear force diagram of beam in axis A

Table-8.7 shear reinforcement of a beam on axis A

			ROOF						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	60.811	54.03	281.25	225	258.93	188.16	225	Φ 8 cc 210mm
	Near 3	61.113	54.3	281.25	225	257.64	188.16	225	Φ 8 cc 210mm
3--5	Near 3	52.89	45.41	281.25	225	308.08	188.16	225	Φ 8 cc 210mm
	Near 5	52.94	45.46	281.25	225	307.74	188.16	225	Φ 8 cc 210mm

			STORY 4						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	134.62	119.6	281.25	225	116.97	188.16	116.9718	Φ 8 cc 110mm
	Near 3	127.01	112.84	281.25	225	123.98	188.16	123.9794	Φ 8 cc 120mm
3--5	Near 3	100.22	86.05	281.25	225	162.58	188.16	162.5779	Φ 8 cc 150mm
	Near 5	96.335	82.71	281.25	225	169.14	188.16	169.1432	Φ 8 cc 150mm

			STORY 3						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	128.13	113.84	281.25	225	122.89	188.16	122.8903	Φ 8 cc 120mm
	Near 3	140.92	125.2	281.25	225	111.74	188.16	111.7399	Φ 8 cc 110mm
3--5	Near 3	113.02	97.04	281.25	225	144.17	188.16	144.1656	Φ 8 cc 130mm
	Near 5	100.65	86.42	281.25	225	161.88	188.16	161.8819	Φ 8 cc 150mm

			STORY 1						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	118.89	105.63	281.25	225	132.44	188.16	132.4418	Φ 8 cc 130mm
	Near 3	105.05	93.33	281.25	225	149.90	188.16	149.8964	Φ 8 cc 130mm
3--5	Near 3	113.34	97.32	281.25	225	143.75	188.16	143.7508	Φ 8 cc 130mm
	Near 5	102.51	88.02	281.25	225	158.94	188.16	158.9392	Φ 8 cc 150mm

			GROUND						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	118.89	105.63	281.25	225	132.44	188.16	132.4418	Φ 8 cc 130mm
	Near 3	116.09	103.14	281.25	225	135.64	188.16	135.6392	Φ 8 cc 130mm
3--5	Near 3	108.89	93.49	281.25	225	149.64	188.16	149.6399	Φ 8 cc 130mm
	Near 5	95.53	82.02	281.25	225	170.57	188.16	170.5661	Φ 8 cc 150mm

			BESEMENT						
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark
2--3	Near 2	51.88	46.09	281.25	225	303.53	188.16	225	Φ 8 cc 210mm
	Near 3	45.84	40.73	281.25	225	343.48	188.16	225	Φ 8 cc 210mm
3--5	Near 3	48.01	41.22	281.25	225	339.39	188.16	225	Φ 8 cc 210mm
	Near 5	42.28	36.3	281.25	225	385.39	188.16	225	Φ 8 cc 210mm

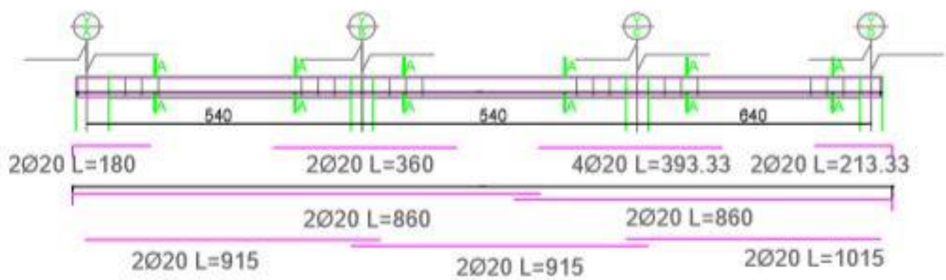


Figure 8.6 Roof floor beam A-B-C & D on axis 2

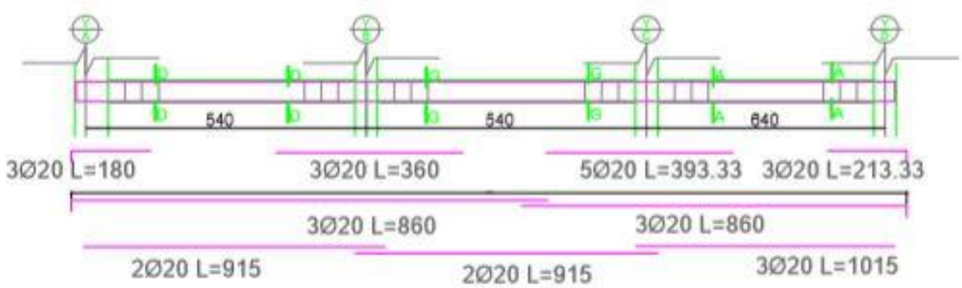


Figure 8.7 Roof floor beam A-B-C &D on axis 2

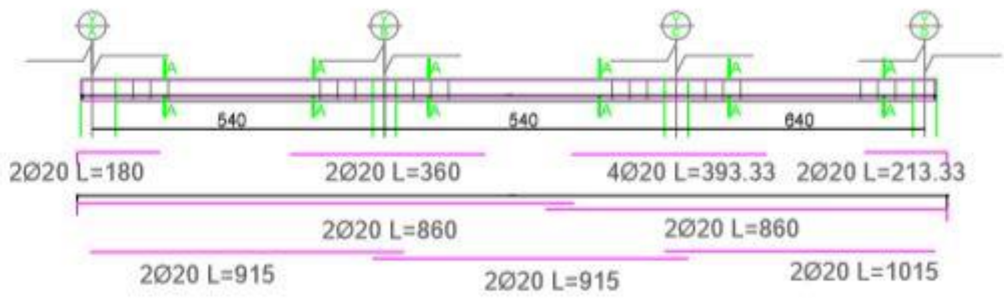


Figure 8.8 Roof floor beam A-B-C& D on axis 2

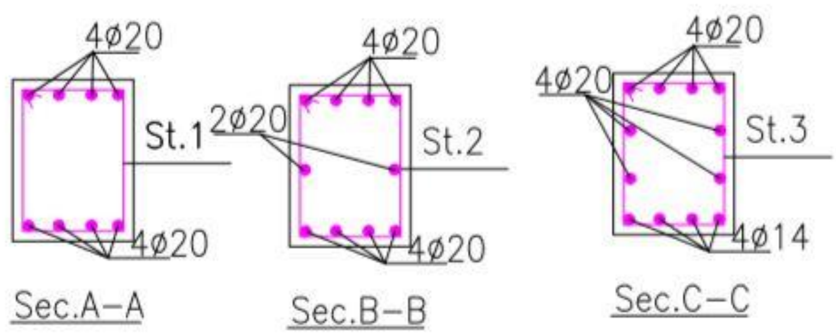


Figure 8.9 section reinforcement bar

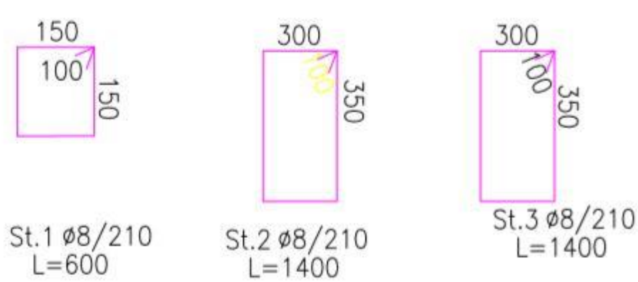


Figure 8.10 stirrup of beam reinforcement

Column

A column is a vertical structural member supporting axial compressive loads, with or without moments. The cross sectional dimensions of a column are generally considerably less than its height. Columns support vertical loads from the floors and roof and transmit these loads to the foundations.

Columns are axially loaded vertical members, which carry their load primarily in compression. The majority of compression or tension members carry a portion of the load in bending which may arise due to the unbalanced moments in the members connected to their ends. The result of such bending moments in axially loaded members into reduces the range of axial force that the member can carry. For this reason, it is essential to note that the effect of bending in axially loaded members should be considered in designing these columns.

Columns are defined as members that carry loads mainly in compression, even though the bending action may produce tensile forces over part of their cross section. Columns are designed to transfer the load from beams and slabs (in flat slab) down to the foundation without buckling or crashing.

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 compression member that is vertical. Almost all compression members in RC structure subjected to moment in addition to axial loads these may be due to misalignment of load on column, may result from column resisting portion of the unbalanced moment at the ends of the beams supported by the columns/ unbalanced moments from beams.

The bending moments can be converted in to un-equivalent axial load applied at eccentricity. Therefore, columns should be designed for the design moment obtained from the total eccentricity.

Columns may be classified based on the following criteria;

- Basis of geometry; depending on the structural or architectural requirements such as, rectangular, square, circular, L-shaped, T-shaped, etc.
- Basis of composition; composite columns, in filled columns, etc.
- Basis of manner by which lateral reinforcement; tied columns, spiral columns.
- Basis of manner by which lateral stability is provided to the structure as a whole; braced columns, un-braced columns
- Basis of sensitivity to second order effect due to lateral displacements; sway columns, non-sway columns.

- Basis of degree of slenderness; short column, slender column.
- Basis of loading; axially loads column, columns under un-axial bending, columns under biaxial bending.

Axial column; column subjected to axial loads accompanied by bending about one axis whereas biaxial, for column support axial force and bending about two perpendicular axes. A braced structure is one which contain bracing elements. These are vertical elements usually walls, which are so stiff relative to other vertical elements that may be assumed to be attract all horizontal forces. Braced structure may be defined as one where the bracing elements attract and transmits to the foundations, at least 90% of all horizontal forces applied to the structure.

9.1 Determination of Effective length

The effective height (length) of a column is the distance between the two consecutive points of contra flexure or zero bending moments. The effective length, lo of member is defined as the length of a pin ended strut with constant normal forces having the same cross-section and buckling load. It depend on the deflected shape and the end restraints.

Table-9.1 C2 story 4th floor column

Top beam	Depth(m)	Width(m)	Length(m)
b1(x,x)	0.2	0.2	5.05
b2(x,x)	0.2	0.2	6.4
b1(y,y)	0.2	0.2	5.4
b1(y,y)	0.2	0.2	5.4
bottom beam	Depth(m)	Width(m)	Length(m)
b1(x,x)	0.3	0.25	5.05
b2(x,x)	0.3	0.25	6.4
b1(y,y)	0.3	0.25	5.4
b1(y,y)	0.3	0.25	5.4
Column	0.35	0.3	3.00

Mtopx-x=36.22KNm

Mtopy-y=36.22KNm

Mbotx-x=27.5KNm

Mboty-y=15.76KNm

NED=406KN

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 (lo) for braced member (ES EN 1992-1-1, 2015)

$$l_0=0.5l\sqrt{\left[1+\frac{k1}{0.5+k1}\right]\left[1+\frac{k2}{0.5+k2}\right]} \text{..............................Eqn (1)}$$

$$k1=\frac{column\ stiffness}{\Sigma beam\ stiffness} \text{..Eqn (2)}$$

$$Ki=\frac{{}^{EI}/_{l}column}{(\Sigma 2^{EI}/_{l})beam}$$

Where,

K1, K2-relative flexibilities of rotational restraint at both ends 1, 2

l- Length of column

K1-stiffness at end 1

K2-stiffness at end 2

Stiffness at each end (K) =column stiffness/∑beam stiffness

$$I_{column}=\frac{bh^3}{12} \text{...Eqn (3)}$$

$$I_{beam}=\frac{bh^3}{12}$$

$$I_{column}=\frac{300*350^3}{12}=1071875000mm^4$$

$$I_{beam}=\frac{200*200^3}{12}=133333333.3\ mm^4$$

$$K_{1x\text{-}x}=\frac{1071875000/_{3000}}{\left(\frac{2*260416667}{5050}+\frac{2*260416667}{6400}\right)}=1.936$$

$K_{2x\text{-}x} = \frac{1071875000/3000}{\left(\frac{2*562500000}{5050} + \frac{2*5625000000}{6400}\right)} = 0.896$

$K_{1y\text{-}y} = \frac{1071875000/3000}{\left(\frac{2*260416667}{5400} + \frac{2*260416667}{5400}\right)} = 1.852$

$K_{2y\text{-}y} = \frac{1071875000/3000}{\left(\frac{2*562500000}{5400} + \frac{2*5625000000}{5400}\right)} = 0.8575$

$l_0x\text{-}x = 0.5l\sqrt{\left[1+\frac{k1x-x}{0.5+k1x-x}\right]\left[1+\frac{k2x-x}{0.5+k2x-x}\right]}$..................................Eqn (4)

$l_0x\text{-}x = 0.5*3000\sqrt{\left[1+\frac{1.936}{0.45+1.936}\right]\left[1+\frac{0.896}{0.45+0.896}\right]}$

$l_0x\text{-}x$ = 2605.6mm

$l_0y\text{-}y = 0.5l\sqrt{\left[1+\frac{k1y-y}{0.5+k1y-y}\right]\left[1+\frac{k2y-y}{0.5+k2y-y}\right]}$

$l_0y\text{-}y = 0.5l\sqrt{\left[1+\frac{1.852}{0.5+1.852}\right]\left[1+\frac{0.858}{0.5+0.858}\right]}$

$l_0y\text{-}y$ =2592.9mm

9.2 Maximum and minimum Reinforcement calculation

For reinforced concrete section to resist shear force ties are used as transverse reinforcement. The diameter of the transverse reinforcement should not be less than 6mm or one quarter of the maximum diameter of the longitudinal bars. The diameter of transverse reinforcement should not be less than 6mm or one quarter of the longitudinal bars:

The total amount of longitudinal reinforcement for column should not be less than As,min and should not exceed As,max. The recommended value of those maximum and minimum reinforcement calculation:-

$$\text{Asmin=max}\begin{cases} 0.1 * \frac{NED}{fyd} \\ 0.002 * Ac \end{cases}$$ ....Eqn(5) (ES EN 1992-1-1, 2015)

$$\text{Asmin=max}\begin{cases} 0.1 * \frac{NED}{fyd} = 0.1 * \frac{406}{347.83} = 117mm2 \\ 0.002 * Ac = 0.002 * 300 * 350 = 210mm2 \end{cases}$$

Asmin=210mm^2

As, max =0.04Ac=0.04*350*300= 4200mm^2

9.3 Determination of eccentricity

Determination of accidental eccentricity, equivalent first order eccentricity and second order effect.

Accidental eccentricity if column is slender column (λ> λlim) ea is calculated as ea=max (lo/400) and 20mm which is the maximum. The effect of imperfection taken from according to (ES EN 1992-1-1, 2015)

$$\text{eix-x= max}\begin{cases} \frac{L0}{400} \\ \frac{h}{20} \\ 20mm \end{cases}$$ ....Eqn (6)

$$\text{eix-x= max}\begin{cases} \frac{2605}{400} = 6.5mm \\ \frac{300}{30} = 10mm \\ 20mm \end{cases}$$

eix-x =20mm

$$\text{eiy-y= max}\begin{cases} \frac{L0}{400} \\ \frac{h}{20} \\ 20mm \end{cases}$$ ....Eqn (7)

$$\text{eiy-y = max}\begin{cases} \frac{2593}{400} = 6.5mm \\ \frac{300}{10} = 10mm \\ 20mm \end{cases}$$

eiy-y =20mm

Note:

The first order effect includes the geometric imperfection, but not consideration of slenderness.

The second order effect is considered the slenderness of structural members or deformation of the structure

First order iteration

MED=max {MO2, Moe+M2, Mo1+0.5M2}.......................................Eqn (8)

M_{01}=min {|Mtop|, |Mbottom| +ei*NED}

M_{02}=max {|Mtop|, |Mbottom| +ei*NED}

Moe=0.4*M_{01}+0.6*M_{02}≥0.4*M_{02}

For first order moment M_2=0

In X direction

M_{01}x-x=min {|Mtopx-x|, |Mbottomx-x| +ei*NED}..................................Eqn (9)

M_{01}x-x=min {|36.22|, |-27.5| +0.02*426}

M_{01}x-x=36.02KNm

M_{02}x-x=max {|Mtopx-x|, |Mbottomx-x| +ei*NED

M_{02}x-x=max {|36.22|, |-27.5| +0.02*426}

M_{02}x-x=44.74KNm

Moex-x=0.4*M_{01}x-x+0.6*M_{02}x-x≥0.4*M_{02}x-x...Eqn (10)

Moex-x=0.4*36.02+0.6*44.74≥0.4*44.74

Moex-x=41.25KNm

MEDx-x=44.74KNm

In Y direction

M_{01}y-y=min {|Mtopy-y|, |Mbottomy-y| +ei*NED}..................................Eqn (11)

M_{01}y-y=min {|15.76|, |27.5|+0.02*426}

M_{01}y-y=24.28 KNm

M_{02}y-y=max {|Mtopy-y|, |Mbottomy-y| +ei*NED}

M_{02}y-y= max {|15.76|, |27.5|+0.02*426}

M_{02}y-y=36.02KNm

Moey-y=0.4*M01y-y+0.6*M02y-y≥0.4*M02y-y..................................Eqn (12)

Moey-y=0.4*15.76 +0.6*44.74≥0.4*44.74

Moey-y=31.32KNm

MEDy-y= 36.02KNm

9.4 Check slenderness

Columns are broadly categorized in to two as short and slender columns. Short columns are columns for which the strength is governed by the strength of the materials and the geometry of the cross section. In short columns, Second-order effects are negligible. In these cases, it is not necessary to consider slenderness effects and compression members can be designed based on forces determined from first-order analyses. When the unsupported length of the column is long, lateral deflections shall be so high that the moments shall increase and weaken the column. (Betelhem, 2020)

$\lambda=\frac{lo}{i}$..Eqn (13)

Where

λ--Slenderness ratio

lo--Effective length of column

i-is radius of gyration of the un cracked concrete i=$\sqrt{\frac{I}{A}}$

λx-x=$\frac{lox-x}{i}$=$\frac{2605}{101.04}$=25.8

λy-y =$\frac{loy-y}{i} = \frac{2593}{101.04}$=25.7

The minimum limiting value of slenderness is;--

λlim=$20 * A * B * C/_{\sqrt{n}}$...Eqn (14)

Where, A – 1/ (1 + 0.2φef), φef-effective creep ratio

B - √1 + 2ω,ω-Asfyd/Acfcd, mechanical reinforcement ratio

C = 1.7—ɼm

n=N_{ED}/Acfcd, relative normal force

Mo1, Mo2 are the first order end moments, |Mo2|≥|Mo1|

If φef is not known, A=0.7

If ω is not known, B=1.1

Take A=0.7 B=1.1 C=1.7—ɼm

Where; ɼm=$\frac{mo1x-x}{mo2x-x}$=$\frac{36.02}{44.74}$=0.81 Cx-x=1.7-0.81=0.89

n=$\frac{Ned}{Acfcd}$=$\frac{406*10^3}{350*300*11.33} = 0.34$

Therefore, slenderness limit (λlimx-x) =$20 * A * B * C/_{\sqrt{n}}$

=$20 * 0.7 * 1.1 * 0.89/_{\sqrt{0.34}}$=

λlimx-x =23.51mm

ƛlimy-y=$20*A*B*C/\sqrt{n}$

Where; ɼm=$\frac{mo1y-y}{mo2y-y}$=$\frac{24.38}{36.02}$=0.68 Cx-x=1.7-0.68=1.02

24.28 36.02

ƛlimy-y =20*0.7*1.1*1.02/√0.8=

ƛlimy-y = 26.94mm

Check slenderness

a. λx-x >λlimx-x, slender column otherwise short column
b. λy-y > λ limy-y, slender column otherwise short column

λx-x > λlimx-x

25.8> 23.51, slender column

λy-y < λ limy-y

25.7< 26.94 Short column

Vsd=$Nsd/(Ac*fcd)$= $406*10\text{^}3/(350*300*11.33)$=0.34

μsd= $MED/(fc*Ac*h)$..Eqn (15)

μsdx-x= $MEDx-x/(fc*Ac*h)$ = $44.74*10\text{^}6/(350*300*300*11.33)$=0.125

μsdy-y= $MEDy-y/(fc*Ac*h)$ = $36.02*10\text{^}6/(350*300*300*11.33)$=0.101

9.5 Reinforcement Calculation

- Longitudinal reinforcement

As,tot=ω*Ac*fcd/fyd..Eqn (16)

Where biaxial rectangular chart number ω=0.2

As,tot=0.2*350*300*11.33/347.8

As,tot=684.10mm^2

Asmin=210mm^2 and As, max =4200mm^2

As min <As<As max

210mm^2 < 684.10mm^2 <4200mm^2..............................OK!

Take $\emptyset 20mm$, as=3.14*(20)^2/4=314mm^2

As=$\frac{Msd}{fyd*h}$

Number of bar=As/as=3890.79mm^2/314mm^2=12.4 ≈13bar

Provide 13Ø20 mm

- For shear reinforcement

Diameter of bar= max $\begin{cases} \frac{1}{4} * longtidunal\ bar\ diameter = \frac{20}{4} = 5 \\ 6mm \\ Diameter\ of\ stirrup = 8mm \end{cases}$

From above take Ø8mm

Diameter of bar= min $\begin{cases} 20 * \phi\ long, = 20 * 20mm = 400mm \\ lesser\ dimention\ of\ column = 300mm \\ 400mm \end{cases}$

Finally, provide $\emptyset 8C/C$ 300mm

The detailing of column presented on Appendix C, C-2

9.6 Determination of whether being sway or Non-sway

For the purpose of design calculations, structures or structural members may be classified as sway or non- sway depending on their sensitivity to second-order effects due to lateral displacements. In a sway frame, additional internal forces or moments due to the effects of the horizontal displacements of its nodes shall be taken into account for design. Additional internal forces or moments are neglected in a non-sway frame since its response to in-plane horizontal forces is sufficiently stiff.(Betelhem, 2020)

For a non – sway frame $N_{sd}/N_{cr}<0.1$

Where: N_{sd} – the design value of the total vertical load.

N_{cr} – critical vertical load for failure in a sway mode

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 it's 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.

Using the spread or isolated footing. It are used to support individual column. Isolated footings are stepped type, simple type or slope type, having projections in the base concrete. To support heavy loads, reinforcement is also provided at the base. The reinforcement provided is in the form of steel bars and is placed in both directions. According to EBCS-7 minimum depth of footing should be 50 cm, for footings on sloping sites, minimum depth of footing should be 60cm and 90 cm below ground surface on rocky and formations, respectively.

The foundation should be design such that soil below doesn't fail in shear and the settlement is the safe limit

Table-10.1 the values of axial load, Mx, My are obtained from Etabs

Column	point	FZ	MX	MY
C1	1	1764.654	1.3717	5.4827
C2	2	2769.743	11.7316	1.4313
C3	3	3046.011	2.2307	6.7755
C4	4	1891.137	2.6084	-9.4865
C5	5	3908.196	-6.1721	5.3552
C6	6	5189.027	-7.9922	-1.3082
C7	7	5241.385	-5.6849	2.7655
C8	8	4131.321	-1.2713	-12.4153
C9	9	2804.355	-4.5965	16.7681
C10	10	4384.796	-12.8066	-3.0116
C11	11	4496.277	-9.8054	4.0996
C12	12	2768.263	-15.7588	-23.4217

Sample design of isolated footing

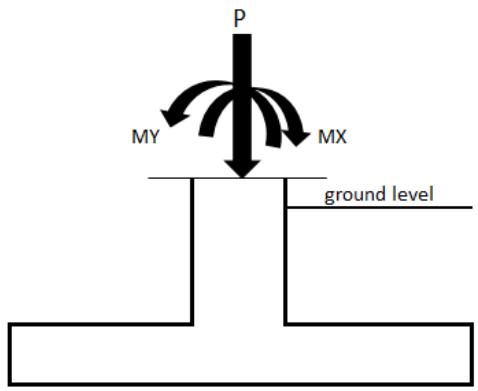


Figure-10.1 sample of isolated footing

Material Data

Concrete data

fcu (Mpa)= C20/25

Fck= 20

Fcd (Mpa)= 11.33

fctm (Mpa)= 2.2104

gmc = 1.5

Ecm (Gpa)= 30

cover (mm)= 60

Reinforcement data

fyk(Mpa)= 400

fyd(Mpa)= 347.8

gms = 1.15

Es(Gpa)= 200

main bar (mm)=ϕ 16

Table-10.2 Etabs out put

Story	Joint label	Load case/combo	Fz	Mx	My
Base	3	Comb1	3046.0112	2.2307	6.7755

Area proportioning

First let us assume that the footing pad is square

Area (A)=B*L=B*B

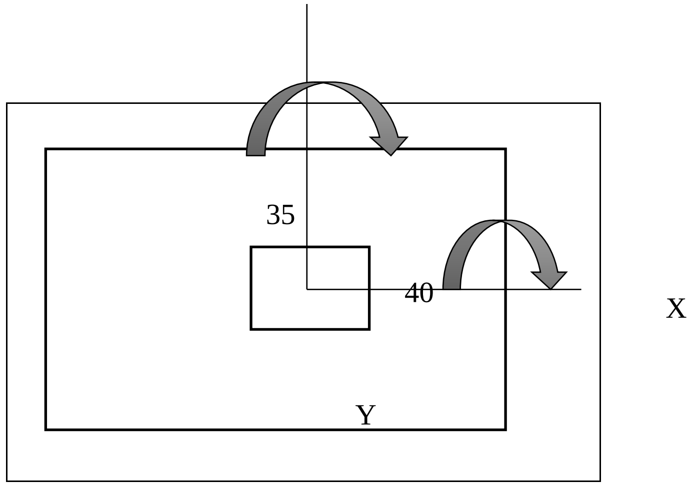


Figure-10.2 footing pad

10.1 Bearing capacity

The soil must be capable of carrying the loads from the structure placed up on it without a shear failure and with the resulting settlements being tolerable for the structure. The design bearing resistance can be taken from the presumed design bearing pressure for different soils according to EBCS-7, 1995 ART 6.10.2 Table 6.3 or can be calculated analytically according to EBCS-7, 1995 ART 6.5.22 for drained as well as un-drained conditions. We have assumed type of soil to be cohesive clay stiff soil and take the corresponding bearing capacity from the presumed design bearing pressure from EBCS-7, 1995 ART 6.10.2 Table 6.3 page 805 which is 300 Kpa

Dimension of column is 500x500mm

From bearing capacity of the soil using the theoretical method

$$\sigma Max \leq Qmax$$

$$\sigma Max/min = p/A * (1 \pm 6ey/B \pm 6ex/L) \text{............Eqn 1}$$

Now finding the eccentricity of each loadings

Eccentricity for service load

$ex = MY/FZ$=6.7755/3046.0112=0.002224384

$ey = MX/FZ$=2.2307/3046.0112=0.000732335

Since our column is square assume square foundation B=L

Substituting these values in equation1, the dimensions of the footing is calculated as fallows

$$\sigma All = pA*(1\pm 6eyB \pm 6exB) \ldots\ldots\ldots\ldots \text{ Eqn } 2$$

At limiting condition σMax =Qmax=350

350=3046.0112/B∗B∗(1+6∗0.0022/B+6∗0.00073/B)

By trial and error the value of B=2.96m

A=B^2=2.96*2.96=8.76m^2

σmax=349.74

σmin=345.57

Since σmax=349.74< σ allowable(350)

So the footing dimension is accepted

The stress at four corner of the footing

σmin=(1−6∗ex/B−6∗ey/B),= 345.57

$\sigma int1$=(1+6∗ex/B−6∗ey/B)= 348.71

$\sigma int2$=(1−6∗ex/B+6∗ey/B)= 346.6

σmax=(1−6∗ex/B+6∗ey/B)= 349.74

$\sigma aveg$=(σmin+σ1+σ2+σmax)/4= 347.65

10.2 Depth determination

The critical section used for footing depth determination is similar to flat slab section.

These sections are

- Wide beam shear section
- Punching shear section
- Bending moment section

a. Wide beam shear section

Critical section for shear is at distance d from the face of supports

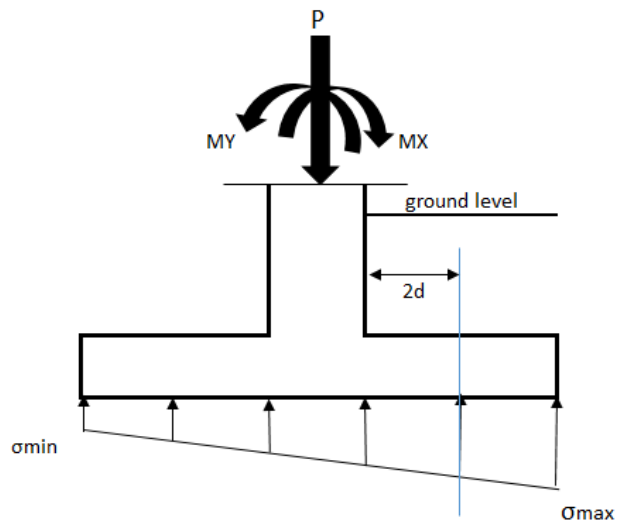


Figure-10.3 wide beam

First assume the depth of footing pad to be D=900M

Cover=60mm

Min φ=16

Depth for vrdc

The design shear at section Y-Y

$Vyy = \sigma avge * B * (B/2 - col(H)/2 - dvrc/1000)$..............................Eqn 4

$dvr = D - cover - min\phi/2$,=900-60-16/2 =832mm

$crdc = 0.18/_{\gamma}$ =018/1.5 =0.12

$K = \min of \begin{cases} 1 + +\frac{\sqrt{200}}{\text{dvrc}} = 1 + \sqrt{200/832} = 1.49 \\ \quad\quad\quad 2 \end{cases}$

K= 1.49

$\rho min = min$ of $\begin{cases} 0.26 * \frac{\text{ftcm}}{400} = 0.26 * 2.6/400 = 0.0014 \\ \quad\quad\quad 0.02 \end{cases}$

ρmin=0.0014

$Vyy=\sigma avge*B*$ $(B/2-col(H)/2-dvrc/1000)$= 409.57

The design shear at section X-X

Vxx$=\sigma avge*L*(L/2-col(H)/2-dvrc/1000)$= 1793.14

Vsd=max of. Vyy=409.57, Vxx=1793.14

Vsd=1793.14

Vrdc=$Crdc*K*$[100$*\rho min*$fck]$^{1/3}$$*B*dvrmin$=2935.56

Vrdc>Vsd--------the depth is adequate for wide beam shear

The design shear at section Y-Y

Assume D=700

$Vyy=\sigma avge*B*(B/2-col(H)/2-dvrc/1000)$=

$dvrmin=D-cover-min\phi/2$=700-60-16/2=632

$crdc$=0.18/$_{\gamma}$ =018/1.5 =0.12

$$K=\text{min } of \begin{cases} 1++\frac{\sqrt{200}}{\text{dvrc}}=1+\sqrt{200/632}=1.5 \\ 2 \end{cases}$$

K=1.5

$$\rho min = min \text{ of} \begin{cases} 0.26*\frac{\text{ftcm}}{400}=0.26*2.6/400=0.0014 \\ 0.02 \end{cases}$$

ρmin=0.0014

$Vyy=\sigma avge*B*(B/2-col(H)/2-dvrc/1000)$= 615.377

The design shear at section X-X

Vxx$=\sigma avge*L*(L/2-col(H)/2-dvrc/1000)$=1113.07

Vsd=max of Vyy=615.377,Vxx=1113.07

Vsd=1113.07

Vrdc=$Crdc*K*$[100$*\rho min*$fck]1/3$*B*dvrmin$= 2229.9

Vrdc= 2229.9

Vrdc>Vsd--------the depth is adequate for wide beam shear

The depth of footing for wide beam shear

D= max of Dvrdc-cover-ϕ/2= 832

Dvmin=632

Therefore=832m

b. Punching shear

Critical section for shear is at distance 2d from the face of column check punching shear at critical perimeter (column face)

Ved,=$Fz-\sigma avg*$(H+B)= 3046.01-347.654*(0.5+0.5)= 2698.36

Ө=22*=0.383972435

tan Ө=0.404

cot Ө= 2.475

V=0.6$*$[1$-FCK$/250]=0.6*(1-20/250)=0.54

Z=0.9*d=0.9*832=748.8

U0=perimeter of column

U0 =2*((3*d/1000+H))+((3*d/1000+0.5))

= 2*((3*832/1000+0.5))+((3*832/1000+0.5))

=11.984

Vrd,=cw* U0*1000*z*Ө*fcd)/(cotӨ+tanӨ)/1000

= (1*11.984*1000*748.8**22*11.33/ (2.475+0.404)

=3254.2203

$Vmin$=0.035$*k^{1.5}*$f$ck^{0.5}*Uo*d$= 0.035$*$1.56$^{1.5}*$20$^{0.5}*$15.321$*$0.832

$Vmin$= 3627.811

Min of Vmin, Vrdc>>Ved, red----3239.02>1664.088----- punching shear at the face of the column is safe

The depth of footing required for both wide beam and punching shear

D=d+cover/2=832+60+8=900

c. Reinforcement Calculation parallel to longer direction

$Msdx$=$\sigma avg*B*[(L/2-col(H)/2)2]/B$= 636.98

$Ascalc$=$msdx*$106/(0.87 fyk z)= 2484.287

K=Msdx/fck bd2= =0.001228

Z=d(0.25-(k/1.134))= 736.794

$ASmin$=0.26$*fctm/fyk$)$*$1000$*d$=4252.81

$ASmax$=0.04$*$1000$*d$=118400

AS, provided = AS calculated <= Asmin <=Asmax

AS, provided =4252.81

Spacing

$$\text{Spacing= min of } \begin{cases} \mathrm{b}\left(\frac{\mathrm{as}}{\mathrm{As}}\right) = 1000 * 3.14 * \frac{2.02}{4} * 4252.81 == 73.83\mathrm{mm} \\ 3 * \mathrm{d} = 3 * 832 = 2496 \\ 400mm \end{cases}$$

=70mm

So we take the minimum spacing =70mm

Provide φ20 C/C 70mm

Conclusion and recommendation

Conclusion

- This final year project aims to provide a complete earthquake, structural analysis and design of B+G+4 mixed use building. The building is located in Addis Ababa. Solid slab, beam, column, stair in this design project.
- This structural design concerned to ensure economical, safe and adequate design that able to stand safely and function without excessive deflection in having excellent serviceability.to achieve this carful design consideration and proper method of case analysis are used.
- The design is carried out based on building standard code, and published reference book. Deferent software analysis is also used to simplify and precise design results. Microsoft Excel sheet, template and ETABS software programs are incorporated.
- Generally; the project express full and step wise procedures that provide an excellent structural design of mixed used building

Recommendation

- The material used during construction should be satisfied the quality of material used in design
- During construction every stage of construction should be controlled by skilled person due to the reason of construction complexity
- If any confusion will happen, anyone can check or refer the new Ethiopian building and amend the design.
- For faculty before delivering the new building code each faculty staff should be understand obviously.

Reference

Betelhem, A. (2020). *Reinforced Concrete Structures 2, Column, and Presentation Outline*.

Debisa, G. (2020). *Design of Reinforced Concrete Slab according to EBCS 2015, Info- session for BSc Thesis/project, Degree of Bachelor Science in Civil Engineering*.

ES EN 8. (2015). *Ethiopian Standard- Based on European Norm; ES EN 1998 Part 1, Design of Structures for Earthquake Resistance- Part 1: General Rules- Seismic Actions and Rules for Buildings, Ethiopian Standard Revision Committee on "'Design of Structures for Earthquake Resistance.'"*

ES EN 1990. (2015). *Ethiopian Standards based on Euro Norms; ES EN 1990:2015, Basis of Structural Design and Action on Structures*.

ES EN 1991. (2015). *ES EN 1991:2015, Actions on Structures-Part 1-4: General actions-Wind actions*.

ES EN 1991: Part 1-4. (2015). *Ethiopian Standards based on Euro Norms; ES EN 1991:2015, Actions on Structures-Part 1-4: General actions-Wind actions, Wind force*.

ES EN 1992-1-1. (2015). *Ethiopian Standards based on Euro Norms; ES EN 1992-1-; 2015, Design of Concrete Structures- Part 1-1: General rules and rules for Buildings*.

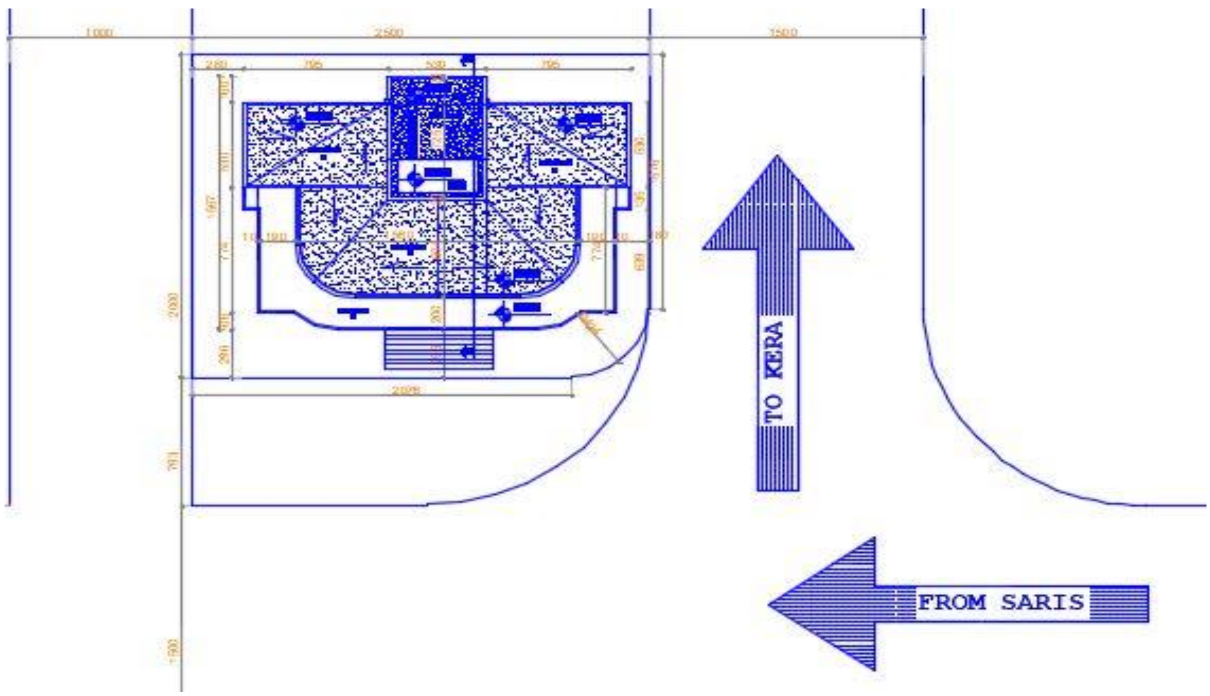
ES EN 1997, Part 1. (2015). *Ethiopian standards Based on Euro Norms; ES EN 1997-1:2015, Geotechnical Design- Part 1: General Rules, Ministry of Construction*.

APPENDIX

Appendix- A

A-1 Architectural drawing of the project

Location on the Study area of the Project is on Addis Ababa



A-2 Basic ratios of effective depth

Table Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	k	Concrete highly stressed $\rho = 1.5\%$	Concrete highly 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
	11.5	20	30

Interior span of beam or one-way or two-way spanning slab Slab supported on columns without beams (flat slab) (based on longer span) Cantilever	11.2 0.4	17 6	24 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			

A-3 Terrain category

To calculate the external pressure coefficient based on(ES EN 1991, 2015)

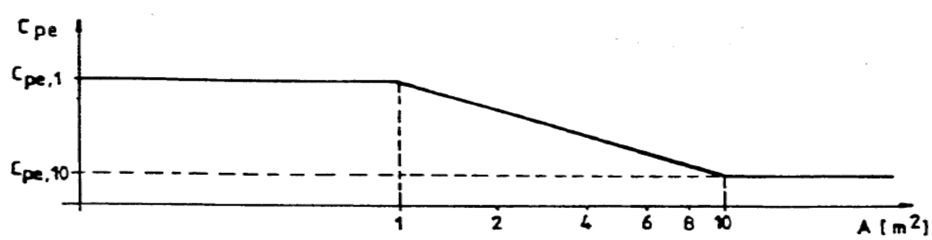


Figure A.1 Variation of External Pressure Coefficient for Buildings with Size of the Loaded Area *A*.

Note: The Figure is based on the following:

$c_{pe} = c_{pe,1}$ $\quad A \leq 1m^2$

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

$A \geq 10m^2$ $\quad c_{pe} = c_{pe,10}$

A-4 bending moment coefficients for rectangular panels

Table A-2 Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners

Type of panel and moments considered	Short span coefficients,βsx								Long span coefficients, βsy for all values of Ly/Lx
	Values of Ly/Lx								
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	

Interior panels Negative moment at continuous edge	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
Positive moment at mid -span	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
One short edge dis continuous Negative moment at continuous edge	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
Positive moment at mid -span	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.028
One long edge dis continuous Negative moment at continuous edge	0.039	0.049	0.056	0.063	0.068	0.073	0.082	0.089	0.037
Positive moment at mid -span	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous Negative moment at continuous edge	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.045
Positive moment at mid -span	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.034
Two short edges discontinuous Negative moment at continuous edge	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	-
Positive moment at mid -span	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
Two long edges discontinuous Negative moment at continuous edge	-	-	-	-	-	-	-	-	0.045
Positive moment at mid -span	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
Three edges discontinuous(one long edge continuous) Negative moment at continuous edge	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	-
Positive moment at mid -span	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0.044
Three edges discontinuous(one short edge continuous) Negative moment at continuous edge	-	-	-	-	-	-	-	-	0.058

Positive moment at mid -span	0.042	0.054	0.063	0.071	0.078	0.084	0.096	0.105	0.044
Four edges discontinuous	0.055	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056
Positive moment at mid -span									

A-5 Shear force coefficients

Table A-3 Shear force coefficients for uniformly loaded rectangular panels supported on four sides with provision for torsion at corners

Type of Panel and location	β_{vx} for values of l_y/l_x								β_{vy}
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Four edges continuous									
Continuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
One short edge discontinuous									
Continuous edge	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
Discontinuous edge	-	-	-	-	-	-	-	-	0.24
One long edge discontinuous									
Continuous edge	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
Discontinuous edge	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
Two adjacent edges discontinuous									
Continuous edge	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
Discontinuous edge	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
Two short edges discontinuous									
Continuous edge	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
Discontinuous edge	-	-	-	-	-	-	-	-	0.26
Two long edges discontinuous									
Continuous edge	-	-	-	-	-	-	-	-	0.40
Discontinuous edge	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
Three edges discontinuous (one long edge discontinuous)									
Continuous edge	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
Discontinuous edge	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.29
Three edges discontinuous (one short edge discontinuous)									
Continuous edge	-	-	-	-	-	-	-	-	0.45
Discontinuous edge	0.29	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
Four edges discontinuous									
Discontinuous edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

A-6 Depth determination for slab from 1st floor up to 4th floor

Table- A-6.1 Depth determination on First floor slab

PANEL	Support Condition	Lx	Ly	N	K	F1	F2&F3	d (mm)	D (mm)
P1	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P2	Interior	5400	6400	17.71	1.5	1.67	1	121.72	148
P3	Interior	6400	6400	17.71	1.5	1.67	1	144.26	170
P4	type3	5050	6400	17.71	1.3	1.67	1	131.34	157
P5	cantilever	2080	5400	17.71	1.5	1.67	1	46.89	73
P6	type3	5050	5400	17.71	1.3	1.67	1	131.34	157
C 1	cantilever	700	7900	17.71	0.4	1.67	1	59.17	85
C2	cantilever	700	5400	17.71	0.4	1.67	1	59.17	85
C3	cantilever	1000	8900	17.71	0.4	1.67	1	84.53	111
C4	cantilever	700	6400	17.71	0.4	1.67	1	59.17	85
C5	cantilever	1000	5050	17.71	0.4	1.67	1	84.53	111
C6	cantilever	1000	5050	17.71	0.4	1.67	1	84.53	111
C7	cantilever	1000	6400	17.71	0.4	1.67	1	84.53	111
							Dmax.		170
							Dprovided(mm)		170

A-7 Load determination for all slabs

Table A-7.1 Self-weight of slabs with different floor finish on the First floor

PANEL	Material	Thickness(m)	Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Load (KN/m^2)
P1	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	8.08
	Pvc	0.02	16	0.32	
P2	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	10.23
	Marble	0.03	27	0.81	
	Partition wall	0.05	14	0.69	
	Plastering wall	0.03	23	0.65	
P3	Rc Slab	0.28	24	6.72	

	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	8.08
	Pvc	0.02	16	0.32	
P4	Rc Slab	0.28	24	6.72	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	13.96
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.20	14	2.81	
	Plastering wall	0.08	23	1.84	
P5	Rc Slab	0.28	24	6.72	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	8.99
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
P6	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	8.08
	Pvc	0.02	16	0.32	
C1	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.05
	Marble	0.03	27	0.81	
	Partition wall	0.05	14	0.77	
	Plastering wall	0.03	23	0.72	
C2	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	9.97
	Marble	0.03	27	0.81	
	Partition wall	0.05	14	0.73	
	Plastering wall	0.03	23	0.68	
C3	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.20
	Marble	0.03	27	0.81	
	Partition wall	0.06	14	0.85	
	Plastering wall	0.03	23	0.79	
C4	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.85

	Marble	0.03	27	0.81	
	Partition wall	0.08	14	1.18	
	Plastering wall	0.05	23	1.10	
C5	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	8.08
	Pvc	0.02	16	0.32	
C6	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	8.08
	Pvc	0.02	16	0.32	
C7	Rc Slab	0.28	24	6.72	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.85
	Marble	0.03	27	0.81	
	Partition wall	0.08	14	1.18	
	Plastering wall	0.05	23	1.10	

PANEL	Dead Load (KN/m^2)	Live Load (KN/m^2)	Total Load (KN/m^2)
P1	8.08	4.5	17.66
P2	10.23	4.5	20.56
P3	8.08	4.5	17.66
P4	13.96	2	21.85
P5	8.99	3.5	17.39
P6	8.08	4.5	17.66
C1	10.05	3.5	18.82
C2	9.97	3.5	18.71
C3	10.2	3.5	19.02
C4	10.85	3.5	19.90
C5	8.08	4.5	17.66
C6	8.08	4.5	17.66
C7	10.85	3.5	19.90

Table A-7.2 Self-weight of slabs with different floor finish on the Second floor

PANEL	Material	Thickness(m)			

			Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Load (KN/m^2)
P1	Rc Slab	0.26	24	6.24	10.94
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.47	
	Plastering wall	0.03	23	0.64	
P2	Rc Slab	0.26	24	6.24	12.80
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.21	14	2.99	
	Plastering wall	0.04	23	0.98	
P3	Rc Slab	0.26	24	6.24	10.88
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.43	
	Plastering wall	0.03	23	0.63	
P4	Rc Slab	0.26	24	6.24	10.67
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.13	14	1.86	
	Plastering wall	0.03	23	0.79	
P5	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	

	Ceramic Screed	0.025	23	0.58	8.51
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
P6	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.68
	Pvc	0.02	16	0.32	
	Partition wall	0.14	14	1.92	
	Plastering wall	0.03	23	0.75	
C1	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.68
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.37	
	Plastering wall	0.03	23	0.80	
C2	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	10.08
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.11	14	1.56	
	Plastering wall	0.02	23	0.51	
C1	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.68
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.37	
	Plastering wall	0.03	23	0.80	
C4	Rc Slab	0.26	24	6.24	
	Ceramic	0.02	21	0.42	

	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	16.53
	Marble	0.03	27	0.81	
	Partition wall	0.43	14	6.04	
	Plastering wall	0.09	23	1.98	
C5	Rc Slab	0.26	24	6.24	8.02
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
C6	Rc Slab	0.26	24	6.24	8.51
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C7	Rc Slab	0.26	24	6.24	8.51
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C8	Rc Slab	0.26	24	6.24	8.02
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
C9	Rc Slab	0.26	24	6.24	16.53
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
	Partition wall	0.43	14	6.04	
	Plastering wall	0.09	23	1.98	

PANEL	Dead Load (KN/m^2)	Live Load (KN/m^2)	Total Load (KN/m^2)
P1	10.94	3.5	20.02
P2	12.8	3.5	22.53
P3	10.88	3.5	19.94
P4	10.67	2	17.40
P5	8.51	3.5	16.74
P6	10.68	2	17.42
C1	10.68	3.5	19.67
C2	10.08	2	16.61
C3	10.68	3.5	19.67
C4	16.53	3.5	27.57
C5	8.02	3.5	16.08
C6	8.51	3.5	16.74
C7	8.51	3.5	16.74
C8	8.02	2	13.83
C9	16.53	3.5	27.57

Table A-7.3 Self-weight of slabs with different floor finish on the Third floor

PANEL	Material	Thickness(m)	Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Load (KN/m^2)
P1	Rc Slab	0.25	24	6.00	12.76
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.27	14	3.75	
	Plastering wall	0.05	23	1.23	
P2	Rc Slab	0.25	24	6.00	12.25
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.22	14	3.08	
	Plastering wall	0.04	23	1.01	
P3	Rc Slab	0.25	24	6.00	

	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	9.86
	Marble	0.03	27	0.81	
	Partition wall	0.07	14	0.92	
	Plastering wall	0.03	23	0.78	
P4	Rc Slab	0.25	24	6.00	10.71
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.14	14	2.03	
	Plastering wall	0.04	23	0.90	
P5	Rc Slab	0.25	24	6.00	8.27
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
P6	Rc Slab	0.25	24	6.00	10.92
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.18	14	2.50	
	Plastering wall	0.03	23	0.64	
C1	Rc Slab	0.25	24	6.00	10.36
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.11	14	1.56	
	Plastering wall	0.04	23	1.03	
C2	Rc Slab	0.25	24	6.00	

	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.32
	Marble	0.03	27	0.81	
	Pvc	0.02	16	0.32	
	Partition wall	0.08	14	1.11	
	Plastering wall	0.05	23	1.05	
C3	Rc Slab	0.25	24	6.00	10.74
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.41	
	Plastering wall	0.05	23	1.17	
C4	Rc Slab	0.25	24	6.00	9.82
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
	Partition wall	0.07	14	0.94	
	Plastering wall	0.03	23	0.62	
C5	Rc Slab	0.25	24	6.00	8.77
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Partition wall	0.06	14	0.83	
	Plastering wall	0.02	23	0.48	
C6	Rc Slab	0.25	24	6.00	8.27
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C7	Rc Slab	0.25	24	6.00	
	Ceramic	0.02	21	0.42	

	Ceramic Screed	0.025	23	0.58	8.27
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C8	Rc Slab	0.25	24	6.00	8.68
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.04	14	0.55	
	Plastering wall	0.02	23	0.36	
C9	Rc Slab	0.25	24	6.00	12.65
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
	Partition wall	0.24	14	3.30	
	Plastering wall	0.05	23	1.08	

PANEL	Dead Load (KN/m^2)	Live Load (KN/m^2)	Total Load (KN/m^2)
P1	13	2	20.55
P2	12.49	3.5	22.11
P3	10.1	3.5	18.89
P4	10.95	2	17.78
P5	8.51	3.5	16.74
P6	11.16	2	18.07
C1	10.6	3.5	19.56
C2	10.56	3.5	19.51
C3	10.98	3.5	20.07
C4	10.06	3.5	18.83
C5	9.01	2	15.16
C6	8.51	3.5	16.74
C7	8.51	3.5	16.74
C8	8.92	2	15.04
C9	12.89	3.5	22.65

Table A-7.4 Self-weight of slabs with different floor finish on the Fourth floor

PANEL	Material	Thickness(m)	Unit Weight(KN/m^3)	Dead Load (KN/m^2)	Total Load (KN/m^2)
P1	Rc Slab	0.25	24	6.00	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.85
	Marble	0.03	27	0.81	
	Partition wall	0.12	14	1.69	
	Plastering wall	0.04	23	0.89	
P2	Rc Slab	0.25	24	6.00	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	11.00
	Partition wall	0.14	14	1.97	
	Plastering wall	0.04	23	0.86	
P3	Rc Slab	0.25	24	6.00	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.34
	Pvc	0.02	16	0.32	
	Marble	0.03	27	0.81	
	Partition wall	0.10	14	1.43	
	Plastering wall	0.03	23	0.75	
P4	Rc Slab	0.25	24	6.00	
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	10.37
	Pvc	0.02	16	0.32	
	Partition wall	0.13	14	1.83	
	Plastering wall	0.03	23	0.77	
P5	Rc Slab	0.25	24	6.00	
	Ceramic Screed	0.025	23	0.58	7.85
	Plastering	0.02	23	0.46	

	Marble	0.03	27	0.81	
P6	Rc Slab	0.25	24	6.00	11.30
	Ceramic	0.02	21	0.42	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
	Partition wall	0.17	14	2.38	
	Plastering wall	0.05	23	1.14	
C1	Rc Slab	0.25	24	6.00	7.85
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C2	Rc Slab	0.25	24	6.00	7.85
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C3	Rc Slab	0.25	24	6.00	7.85
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C4	Rc Slab	0.25	24	6.00	7.85
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Marble	0.03	27	0.81	
C5	Rc Slab	0.25	24	6.00	7.36
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	
C6	Rc Slab	0.25	24	6.00	7.36
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	
	Pvc	0.02	16	0.32	

C7	Rc Slab	0.25	24	6.00	
	Ceramic Screed	0.025	23	0.58	
	Plastering	0.02	23	0.46	7.85
	Marble	0.03	27	0.81	

PANEL	Dead Load (KN/m^2)	Live Load (KN/m^2)	Total Load (KN/m^2)
P1	11.09	3.5	20.22
P2	11.24	3.5	20.42
P3	10.58	3.5	19.53
P4	10.61	2	17.32
P5	8.09	3.5	16.17
P6	11.54	2	18.58
C1	8.09	3.5	16.17
C2	8.09	3.5	16.17
C3	8.09	3.5	16.17
C4	8.09	2	13.92
C5	7.6	3.5	15.51
C6	7.6	2	13.26
C7	8.09	3.5	16.17

A-8 Moment analysis for all slabs

Table A-8.1 First floor slab moment analysis

PANEL		Lx	Ly	Ly/Lx	n(pd)	βsx-	βsx+	βsy-	βsy+	Msx-	Msx+	Msy-	Msy+
P1	Interior	5.4	7.1	1.31	17.66	0.046	0.032	0.035	0.024	23.69	16.48	18.02	12.36
P2	Interior	5.4	6.4	1.19	20.56	0.042	0.032	0.032	0.024	25.18	19.18	19.18	14.39
P3	Interior	6.4	6.4	1.00	17.66	0.031	0.032	0.024	0.024	22.42	23.15	17.36	17.36
P4	type 3	5.05	6.4	1.27	21.85	0.061	0.037	0.046	0.028	33.99	20.62	25.63	15.60
P6	type 3	5.05	5.4	1.07	17.66	0.046	0.037	0.034	0.028	20.72	16.66	15.31	12.61

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P5	interior	2.08	5.4	2.60	17.39	37.62
C 1	cantilever	2.5	7.9	3.16	18.82	58.81
C2	cantilever	3	5.4	1.80	18.71	84.20
C3	cantilever	2.5	8.9	3.56	19.02	59.44
C4	cantilever	2.5	6.4	2.56	19.9	62.19
C5	cantilever	2.5	5.05	2.02	17.66	55.19
C6	Cantilever	2.5	5.05	2.02	17.66	55.19
C7	cantilever	2.5	6.4	2.56	19.9	62.19

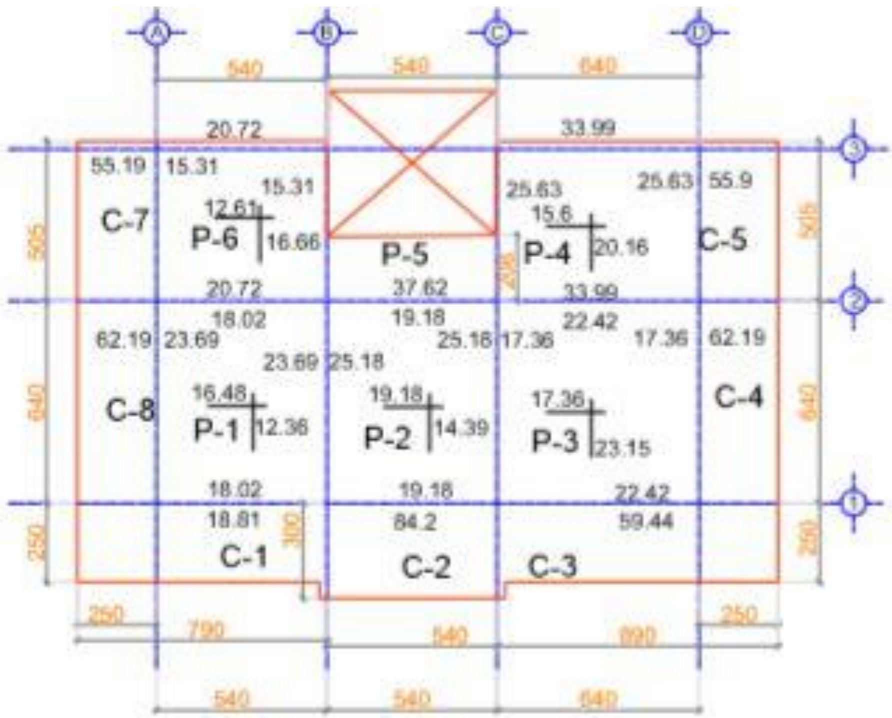


Figure A-8.1 1st floor moment analysis

Table A-8.2 Second floor slab moment analysis

PANEL		Lx	Ly	Ly/Lx	n(pd)	βsx-	βsx+	βsy-	βsy+	Msx-	Msx+	Msy-	Msy+
P1	Interior	5.4	6.4	1.19	20.02	0.042	0.032	0.032	0.024	24.52	18.68	18.68	14.01
P2	Interior	5.4	6.4	1.19	22.53	0.042	0.032	0.032	0.024	27.59	21.02	21.02	15.77
P3	Interior	6.4	6.4	1.00	19.94	0.031	0.032	0.024	0.024	25.32	26.14	19.60	19.60
P4	type 3	5.05	6.4	1.27	17.4	0.061	0.037	0.046	0.028	27.07	16.42	20.41	12.42
P6	type 3	5.05	5.4	1.07	17.42	0.046	0.037	0.034	0.028	20.44	16.44	15.10	12.44

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P5	cantilever	2.08	5.4	2.60	16.74	36.21
C 1	cantilever	2.7	7.1	2.63	19.67	71.70
C2	cantilever	1.59	5.4	3.40	16.61	21.00
C3	cantilever	2.7	7.1	2.63	19.67	71.70
C4	cantilever	1.5	6.4	4.27	27.57	31.02
C5	cantilever	1.5	5.05	3.37	16.08	18.09
C6	cantilever	0.8	3.05	3.81	16.74	5.36
C7	cantilever	0.8	3.05	3.81	16.74	5.36
C8	cantilever	2.5	5.05	2.02	13.83	43.22

C9	cantilever	2.5	6.4	2.56	27.57	86.16

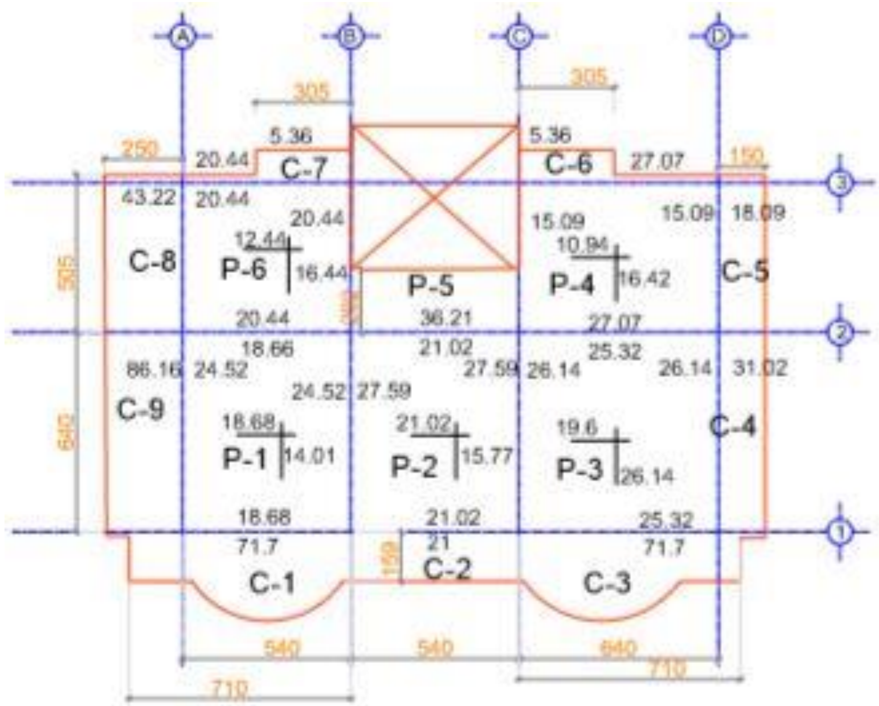


Figure A-8.2. 2nd floor moment analysis

Table- A-8.3 Third floor slab moment analysis

PANEL		Lx	Ly	Ly/Lx	n(pd)	βsx-	βsx+	βsy-	βsy+	Msx-	Msx+	Msy-	Msy+
P1	Interior	5.4	6.4	1.19	13	0.042	0.032	0.032	0.024	15.92	12.13	12.13	9.10
P2	Interior	5.4	6.4	1.19	12.49	0.042	0.032	0.032	0.024	15.30	11.65	11.65	8.74
P3	Interior	6.4	6.4	1.00	10.1	0.031	0.032	0.024	0.024	12.82	13.24	9.93	9.93
P4	type 3	5.05	6.4	1.27	10.95	0.061	0.037	0.046	0.028	17.03	10.33	12.85	7.82
P6	type 3	5.05	5.4	1.07	11.16	0.046	0.037	0.034	0.028	13.09	10.53	9.68	7.97

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P5	cantilever	2.08	5.4	2.60	8.51	18.41
C 1	cantilever	2.64	7.1	2.69	10.6	36.94
C2	cantilever	2.64	5.4	2.05	10.56	36.80
C3	cantilever	2.64	7.1	2.69	10.98	38.26
C4	cantilever	1.5	6.4	4.27	10.06	11.32

C5	cantilever	1.5	5.05	3.37	9.01	10.14
C6	cantilever	0.8	3.05	3.81	8.51	2.72
C7	cantilever	0.8	3.05	3.81	8.51	2.72
C8	cantilever	2.5	5.05	2.02	8.92	27.88
C9	cantilever	2.1	6.4	3.05	12.89	28.42

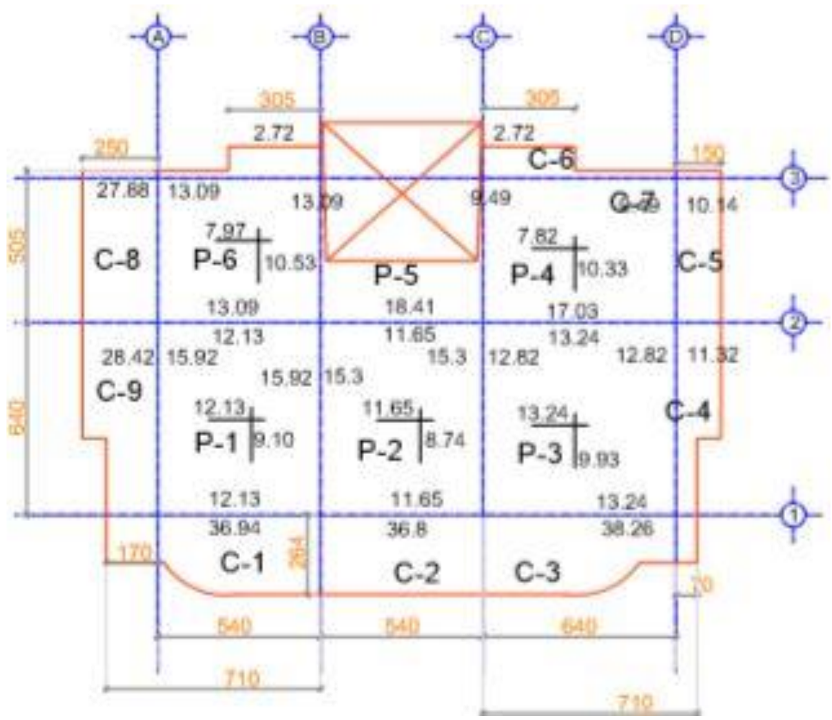


Figure A-8.3 3rd floor moment analysis

Table A-8.4 Fourth floor slab moment analysis

PANEL		Lx	Ly	Ly/Lx	n(pd)	βsx-	βsx+	βsy-	βsy+	Msx-	Msx+	Msy-	Msy+
P1	Interior	5.4	6.4	1.19	20.22	0.042	0.032	0.032	0.024	24.76	18.87	18.87	14.15
P2	Interior	5.4	6.4	1.19	20.42	0.042	0.032	0.032	0.024	25.01	19.05	19.05	14.29
P3	Interior	6.4	6.4	1.00	19.53	0.031	0.032	0.024	0.024	24.80	25.60	19.20	19.20
P4	type 3	5.05	6.4	1.27	17.32	0.061	0.037	0.046	0.028	26.94	16.34	20.32	12.37
P6	type 4	5.05	5.4	1.07	18.58	0.046	0.037	0.034	0.028	21.80	17.53	16.11	13.27

PANEL		Lx	Ly	Ly/Lx	n(pd)	Mxs(KN)
P5	cantilever	2.08	5.4	2.60	16.17	34.98
C 1	cantilever	2.64	5.4	2.05	16.17	56.35
C2	cantilever	2.64	5.4	2.05	16.17	56.35
C3	cantilever	2.64	6.4	2.42	16.17	56.35
C4	cantilever	1.5	6.4	4.27	13.92	15.66
C5	cantilever	1.5	5.05	3.37	15.51	17.45
C6	cantilever	2.5	5.05	2.02	13.26	41.44
C7	cantilever	2.5	6.4	2.56	16.17	50.53

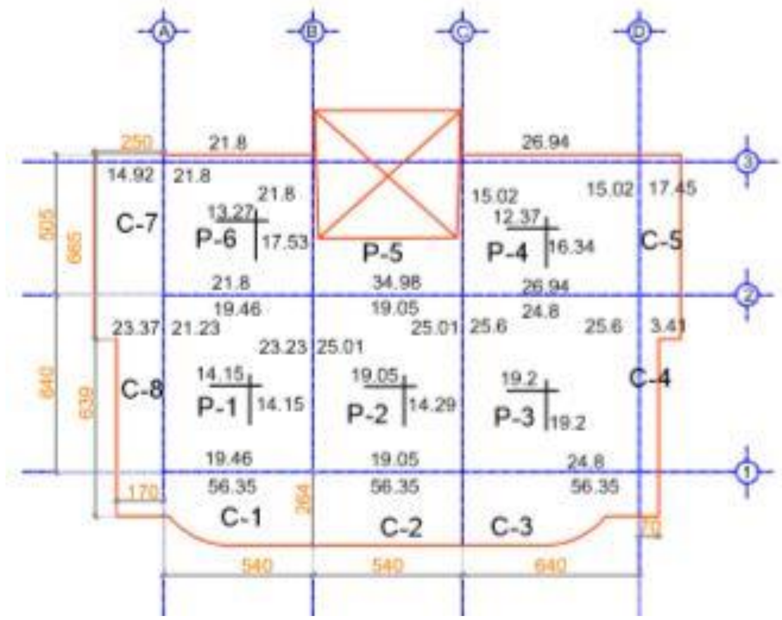


Figure A-8.4 4th floor moment analysis

A-9 Moment adjustment for all slabs

Table A-9.1 support moment adjustment on First floor

PANEL	MTD 1	MTD 2	MAX
P1&P2	24.44		
P1&P6	19.37		
P1&C1			58.81
P1&C7			62.19
P2&P3		20.94	
P2&P5			37.62
P2&C2			84.2
P3&P4		28.35	
P3&C3			59.44
P3&C4			62.19

P4&P5			25.63
P4&C5			55.19
P5&P6			15.31
P6&C7			55.19

Table A-9.2 span moment adjustment on First floor

PANEL	Msx-	Msx+	Madjust	Msx
P1	23.69	16.48	24.44	15.73
P2	25.18	19.18	24.44	19.92
P3	22.42	23.15	28.35	17.22
P4	33.99	20.62	33.99	20.62
P6	20.72	16.66	19.37	18.01
PANEL	Msy-	Msy+	Madjust	Msy
P1	18.02	12.36	19.37	11.01
P2	19.18	14.39	19.18	14.39
P3	17.36	17.36	17.36	17.36
P4	25.63	15.6	25.63	15.60
P6	15.31	12.61	15.31	12.61

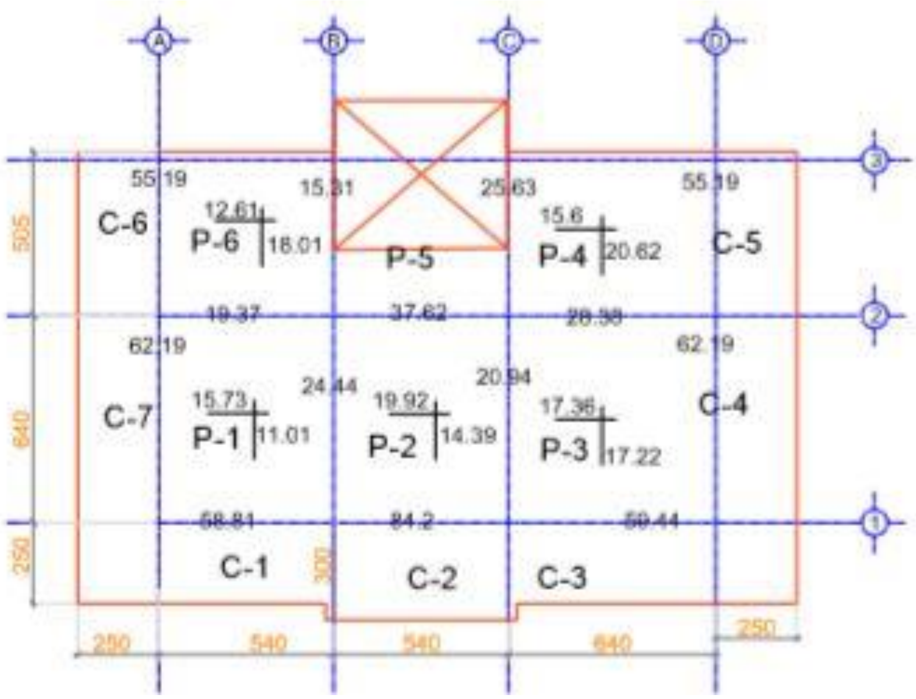


Figure A-9.3 1st floor moment adjustment

Table- A-9.4 support moment adjustment on Second floor

PANEL	MTD 1	MTD 2	MAX
P1&P2	26.06		
P1&P6	19.56		
P1&C1			86.16
P2&P3	26.87		
P2&P5			36.21
P2&C2			21.02
P3&P4	26.2		
P3&C3			71.7
P3&C4			31.02
P4&P5			15.09
P4&C5			18.09
P4&C6			19.97
P5&P6			11.99
P6&C7			15.55
P6&C8			43.22

Table- A-9.5 span moment adjustment on Second floor

PANEL	Msx-	Msx+	Madjust	Msx
P1	24.52	18.68	26.06	17.14
P2	27.59	21.02	26.87	21.74
P3	25.32	26.14	26.87	24.59
P4	27.07	16.42	26.2	17.29
P6	20.44	16.44	19.56	17.32
PANEL	Msy-	Msy+	Madjust	Msy
P1	18.68	14.01	17.12	15.57
P2	21.02	15.77	21.02	15.77
P3	26.14	19.6	22.96	22.78
P4	15.09	12.42	15.09	12.42
P6	20.44	12.44	20.44	12.44

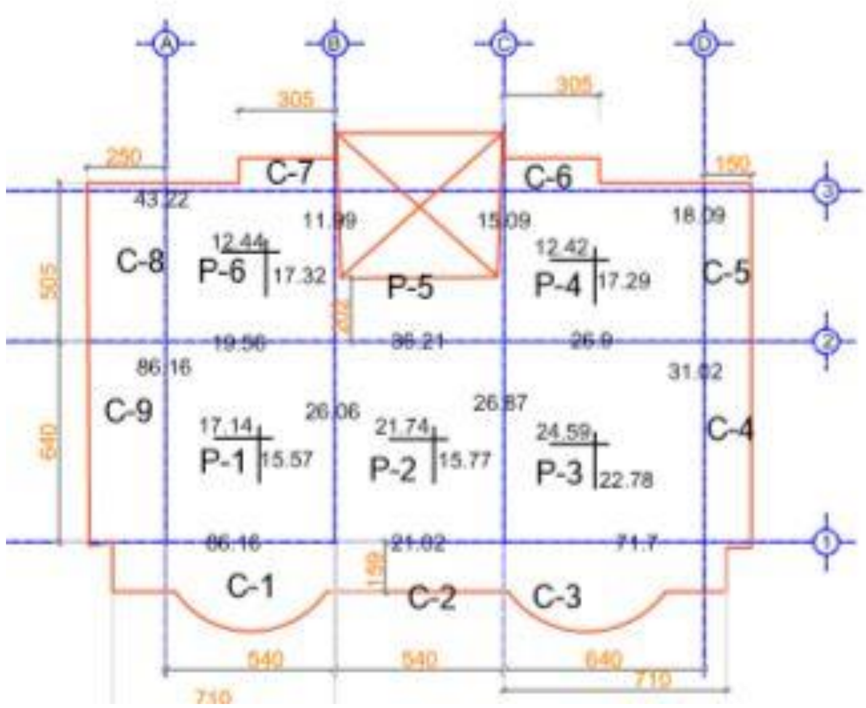


Figure A-9.2 2nd floor moment adjustment

Table A-9.6 support moment adjustment on Third floor

PANEL	MTD 1	MTD 2	MAX
P1&P2	15.61		
P1&P6	12.61		
P1&C1			36.94
P1&C9			28.42
P2&P3	14.06		
P2&P5			18.41
P2&C2			36.8
P3&P4		15.18	
P3&C3			36.26
P3&C4			11.32
P4&P5			9.49
P4&C5			10.14
P5&P6			13.09
P6&C7			13.09
P6&C8			27.88

Table- A-9.7 span moment adjustment on Third floor

PANEL	Msx-	Msx+	Madjust	Msx
P1	15.92	12.13	15.61	12.44
P2	15.3	11.65	14.06	12.89
P3	12.82	13.24	15.18	10.88
P4	17.03	10.33	15.18	12.18
P6	13.09	10.53	12.61	11.01
PANEL	Msy-	Msy+	Madjust	Msy
P1	12.13	9.1	12.61	8.62
P2	11.65	8.74	11.65	8.74
P3	13.24	9.93	14.06	9.11
P4	9.49	7.82	9.49	7.82
P6	13.09	7.97	13.09	7.97

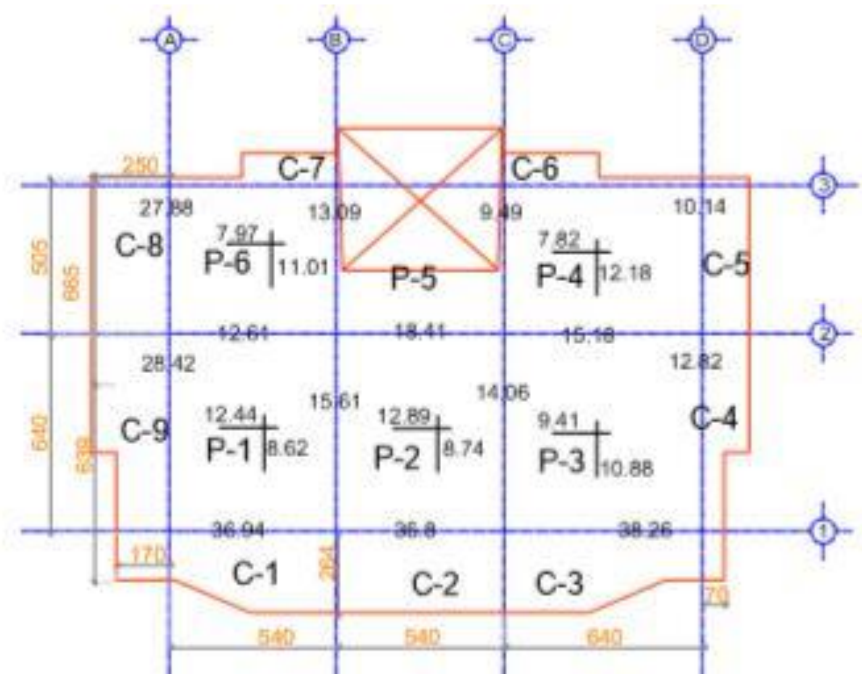


Figure A-9.3 3rd floor moment adjustment

Table- A-9.8 support moment adjustment on Forth floor

PANEL	MTD 1	MTD 2	MAX
P1&P2	23.12		
P1&P6	20.63		
P1&C1			56.35

P1&C7			23.37
P2&P3	25.31		
P2&P5			34.98
P2&C2			56.35
P3&P4	25.87		
P3&C3			56.35
P3&C4			25.6
P4&P5	15.02		
P4&C5			17.45
P6&P5	21.8		
P6&C6			21.8

Table-4 A-9.9 span moment adjustment on Third floor

PANEL	Msx-	Msx+	Madjust	Msx
P1	21.23	14.15	23.12	12.26
P2	25.01	19.05	25.31	18.75
P3	24.8	19.2	25.31	18.69
P4	26.94	16.34	25.87	17.41
P6	21.8	17.53	20.63	18.70
PANEL	Msy-	Msy+	Madjust	Msy
P1	19.46	14.15	20.63	12.98
P2	19.05	14.29	19.05	14.29
P3	25.6	19.2	25.87	18.93
P4	15.02	12.37	15.02	12.37
P6	21.8	13.27	21.8	13.27

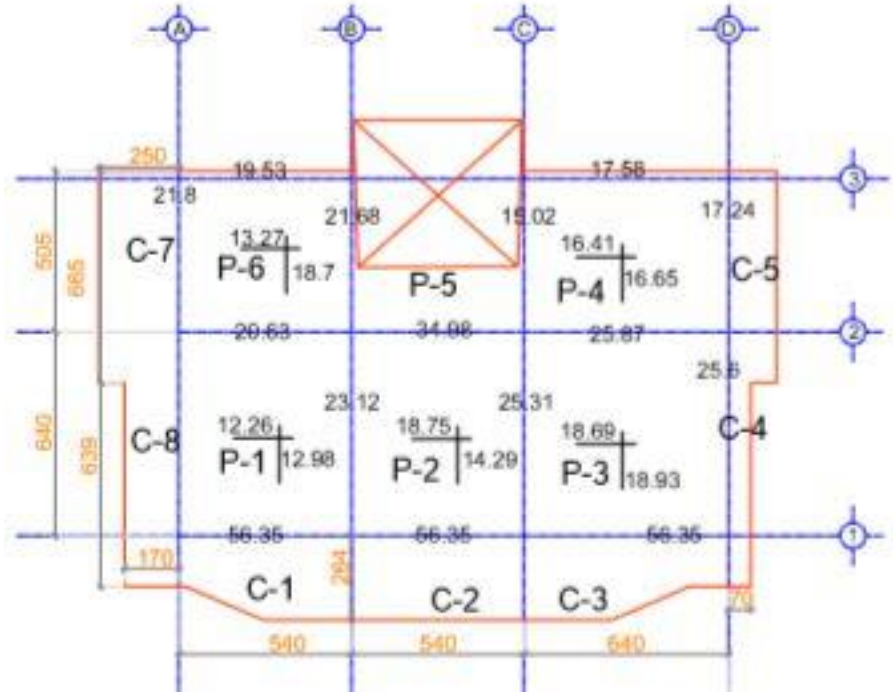


Figure A-9.4 4th floor moment adjustment

A-10 Reinforcement bar for slabs

Table A-10.1 reinforcement for First floor slab

Panel		M(KNm)	Rebar
P-1	Mxs-	23.69	Φ 10 cc 210mm
	Mxs+	16.48	Φ 10 cc 310mm
	Mys-	18.02	Φ 10 cc 270mm
	Mys+	12.36	Φ 10 cc 400mm
P-2	Mxs-	25.18	Φ 10 cc 200mm
	Mxs+	19.18	Φ 10 cc 260mm
	Mys-	19.18	Φ 10 cc 250mm
	Mys+	14.39	Φ 10 cc 340mm
P-3	Mxs-	22.42	Φ 10 cc 220mm
	Mxs+	23.15	Φ 10 cc 280mm
	Mys-	17.36	Φ 10 cc 280mm
	Mys+	17.36	Φ 10 cc 280mm
P-4	Mxs-	33.99	Φ 10 cc 150mm

	Mxs+	20.62	Φ 10 cc 240mm
	Mys-	25.63	Φ 10 cc 190mm
	Mys+	15.6	Φ 10 cc 310mm
P-5	Mxs	37.62	Φ 10 cc 130mm
P-6	Mxs-	20.72	Φ 10 cc 240mm
	Mxs+	16.66	Φ 10 cc 300mm
	Mys-	15.31	Φ 10 cc 320mm
	Mys+	12.61	Φ 10 cc 390mm
C-1	Mxs	58.81	Φ 10 cc 80mm
C-2	Mxs	84.2	Φ 10 cc 50mm
C-3	Mxs	59.44	Φ 10 cc 80mm
C-4	Mxs	62.19	Φ 10 cc 80mm
C-5	Mxs	55.19	Φ 10 cc 90mm
C-6	Mxs	55.19	Φ 10 cc 90mm
C-7	Mxs	62.19	Φ 10 cc 80mm

Table A-10.2 reinforcement for second floor slab

Panel		M(KNm)	Rebar
P-1	Mxs-	24.52	Φ 10 cc 190mm
	Mxs+	18.68	Φ 10 cc 250mm
	Mys-	18.68	Φ 10 cc 240mm
	Mys+	14.01	Φ 10 cc 320mm
P-2	Mxs-	27.59	Φ 10 cc 170mm
	Mxs+	21.02	Φ 10 cc 220mm
	Mys-	21.02	Φ 10 cc 210mm
	Mys+	15.77	Φ 10 cc 280mm
P-3	Mxs-	25.32	Φ 10 cc 180mm
	Mxs+	26.14	Φ 10 cc 180mm
	Mys-	19.6	Φ 10 cc 230mm
	Mys+	19.6	Φ 10 cc 230mm
P-4	Mxs-	27.07	Φ 10 cc 170mm
	Mxs+	16.42	Φ 10 cc 280mm
	Mys-	20.41	Φ 10 cc 220mm
	Mys+	12.42	Φ 10 cc 360mm
P-5	Mxs	36.21	Φ 10 cc 120mm
P-6	Mxs-	20.44	Φ 10 cc 230mm
	Mxs+	16.44	Φ 10 cc 280mm
	Mys-	15.1	Φ 10 cc 300mm
	Mys+	12.44	Φ 10 cc 360mm
C-1	Mxs	71.7	Φ 10 cc 60mm
C-2	Mxs	21	Φ 10 cc 220mm
C-3	Mxs	71.7	Φ 10 cc 60mm

C-4	Mxs	31.02	Φ 10 cc 150mm
C-5	Mxs	18.09	Φ 10 cc 260mm
C-6	Mxs	5.36	Φ 10 cc 780mm
C-7	Mxs	5.36	Φ 10 cc 780mm
C-8	Mxs	43.22	Φ 10 cc 100mm
C-9	Mxs	86.16	Φ 10 cc 50mm

Table A-10.3 reinforcement for Third floor slab

Panel		M(KNm)	Rebar
P-1	Mxs-	15.92	Φ 10 cc 280mm
	Mxs+	12.13	Φ 10 cc 370mm
	Mys-	12.13	Φ 10 cc 350mm
	Mys+	9.1	Φ 10 cc 470mm
P-2	Mxs-	15.3	Φ 10 cc 290mm
	Mxs+	11.65	Φ 10 cc 390mm
	Mys-	11.65	Φ 10 cc 370mm
	Mys+	8.74	Φ 10 cc 400mm
P-3	Mxs-	12.82	Φ 10 cc 350mm
	Mxs+	13.24	Φ 10 cc 340mm
	Mys-	13.24	Φ 10 cc 320mm
	Mys+	9.93	Φ 10 cc 430mm
P-4	Mxs-	17.03	Φ 10 cc 260mm
	Mxs+	10.33	Φ 10 cc 440mm
	Mys-	9.49	Φ 10 cc 450mm
	Mys+	7.82	Φ 10 cc 550mm
P-5	Mxs	18.41	Φ 10 cc 240mm
P-6	Mxs-	13.09	Φ 10 cc 340mm
	Mxs+	10.53	Φ 10 cc 430mm
	Mys-	13.09	Φ 10 cc 330mm
	Mys+	7.97	Φ 10 cc 540mm
C-1	Mxs	36.94	Φ 10 cc 120mm
C-2	Mxs	36.8	Φ 10 cc 120mm
C-3	Mxs	38.26	Φ 10 cc 110mm
C-4	Mxs	11.32	Φ 10 cc 400mm
C-5	Mxs	10.14	Φ 10 cc 450mm
C-6	Mxs	2.72	Φ 10 cc 750mm
C-7	Mxs	2.72	Φ 10 cc 750mm
C-8	Mxs	27.88	Φ 10 cc 160mm
C-9	Mxs	28.42	Φ 10 cc 150mm

Table A-10.4 reinforcement for Forth floor slab

Panel		M(KNm)	Rebar
P-1	Mxs-	21.23	Φ 10 cc 210mm
	Mxs+	14.15	Φ 10 cc 320mm
	Mys-	19.46	Φ 10 cc 220mm
	Mys+	14.15	Φ 10 cc 300mm
P-2	Mxs-	25.01	Φ 10 cc 180mm
	Mxs+	19.05	Φ 10 cc 230mm
	Mys-	19.05	Φ 10 cc 220mm
	Mys+	14.29	Φ 10 cc 300mm
P-3	Mxs-	24.8	Φ 10 cc 180mm
	Mxs+	19.2	Φ 10 cc 230mm
	Mys-	25.6	Φ 10 cc 160mm
	Mys+	19.2	Φ 10 cc 220mm
P-4	Mxs-	26.94	Φ 10 cc 160mm
	Mxs+	16.34	Φ 10 cc 280mm
	Mys-	15.02	Φ 10 cc 290mm
	Mys+	12.37	Φ 10 cc 350mm
P-5	Mxs	34.98	Φ 10 cc 120mm
P-6	Mxs-	21.8	Φ 10 cc 210mm
	Mxs+	17.53	Φ 10 cc 260mm
	Mys-	21.8	Φ 10 cc 200mm
	Mys+	13.27	Φ 10 cc 330mm
C-1	Mxs	56.35	Φ 10 cc 70mm
C-2	Mxs	56.35	Φ 10 cc 70mm
C-3	Mxs	56.35	Φ 10 cc 70mm
C-4	Mxs	15.66	Φ 10 cc 290mm
C-5	Mxs	17.45	Φ 10 cc 2600mm
C-6	Mxs	41.44	Φ 10 cc 100mm
C-7	Mxs	50.53	Φ 10 cc 80mm

A-11 Load transfer to supporting beam

Table A-11.1 Load transfer from First floor slab to beam

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
P1	Interior	5.4	7.1	1.31	17.66	0.47	0.36	44.82	34.33
P2	Interior	5.4	6.4	1.19	20.56	0.39	0.33	43.30	36.64
P3	Interior	6.4	6.4	1.00	17.66	0.33	0.33	37.30	37.30
P4	type 3	5.05	6.4	1.27	21.85	0.46	0.36	50.76	39.72
P6	type 3	5.05	5.4	1.07	17.66	0.39	0.36	34.78	32.11

PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	Wl(KN)
P5	interior	2.08	5.4	2.60	17.39	36.17
C 1	cantilever	2.5	7.9	3.16	18.82	47.05

C2	cantilever	3	5.4	1.80	18.71	56.13
C3	cantilever	2.5	8.9	3.56	19.02	47.55
C4	cantilever	2.5	6.4	2.56	19.9	49.75
C5	cantilever	2.5	5.05	2.02	17.66	44.15
C6	cantilever	2.5	5.05	2.02	17.66	44.15
C7	cantilever	2.5	6.4	2.56	19.9	49.75

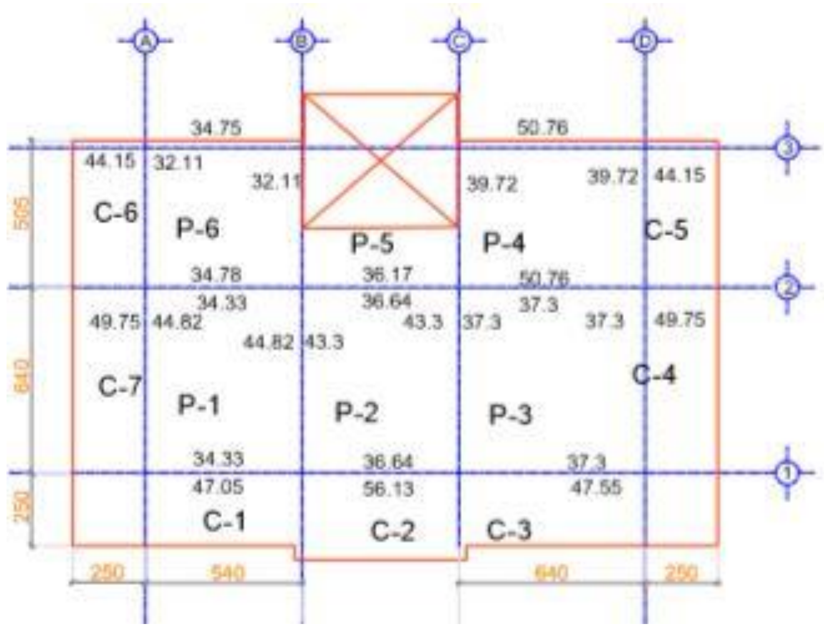


Figure A-11.1 load transfer from 1st floor

Table A-11.2 Load transfer from Second floor slab to beam

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
P1	Interior	5.4	6.4	1.19	20.02	0.39	0.33	42.16	35.68
P2	Interior	5.4	6.4	1.19	22.53	0.39	0.33	47.45	40.15
P3	Interior	6.4	6.4	1.00	19.94	0.33	0.33	42.11	42.11
P4	type 3	5.05	6.4	1.27	17.4	0.46	0.36	40.42	31.63
P6	type 3	5.05	5.4	1.07	17.42	0.39	0.36	34.31	31.67

PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	Wl(KN)
P5	Interior	2.08	5.4	2.60	16.74	34.82
C 1	cantilever	2.7	7.1	2.63	19.67	53.11
C2	cantilever	1.59	5.4	3.40	16.61	26.41
C3	cantilever	2.7	7.1	2.63	19.67	53.11
C4	cantilever	1.5	6.4	4.27	27.57	41.36
C5	cantilever	1.5	5.05	3.37	16.08	24.12

C6	cantilever	0.8	3.05	3.81	16.74	13.39
C7	cantilever	0.8	3.05	3.81	16.74	13.39
C8	cantilever	2.5	5.05	2.02	13.83	34.58
C9	cantilever	2.5	6.4	2.56	27.57	68.93

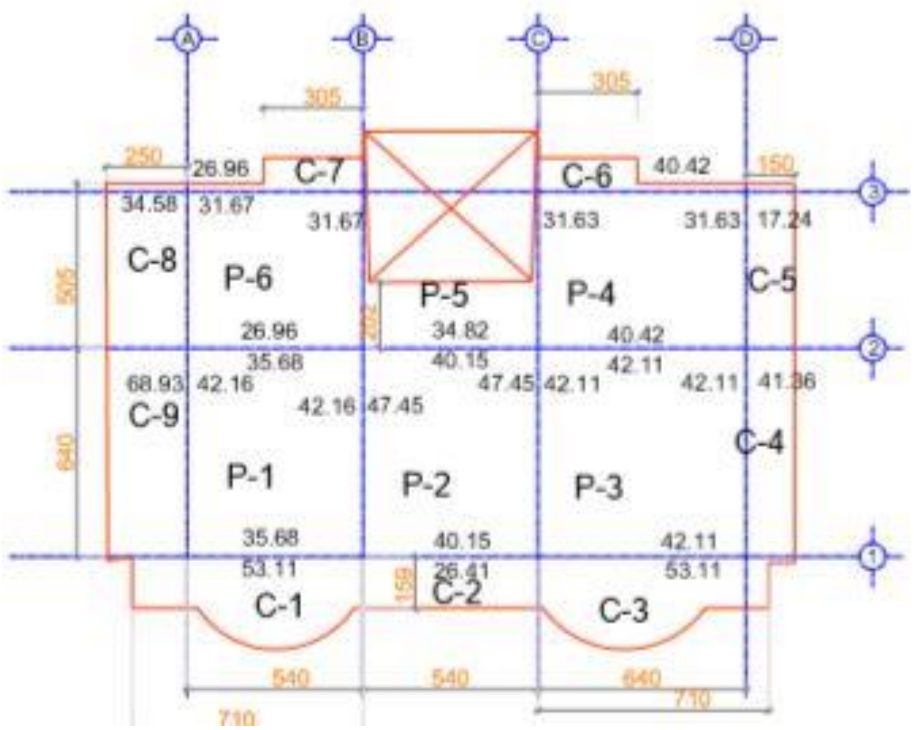


Figure A-11.2 load transfer from 2nd floor

Table A-11.3 Load transfer from Third floor slab to beam

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
P1	Interior	5.4	6.4	1.19	20.55	0.39	0.33	43.28	36.62
P2	Interior	5.4	6.4	1.19	22.11	0.39	0.33	46.56	39.40
P3	Interior	6.4	6.4	1.00	18.89	0.33	0.33	39.90	39.90
P4	type 3	5.05	6.4	1.27	17.78	0.46	0.36	41.30	32.32
P6	type 3	5.05	5.4	1.07	18.07	0.39	0.36	35.59	32.85

PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	Wl(KN)
P5	cantilever	2.08	5.4	2.60	16.74	34.82
C 1	cantilever	2.64	7.1	2.69	19.56	51.64
C2	cantilever	2.64	5.4	2.05	19.51	51.51
C3	cantilever	2.64	7.1	2.69	20.07	52.98
C4	cantilever	1.5	6.4	4.27	18.83	28.25

C5	cantilever	1.5	5.05	3.37	15.16	22.74
C6	cantilever	0.8	3.05	3.81	16.74	13.39
C7	cantilever	0.8	3.05	3.81	16.74	13.39
C8	cantilever	2.5	5.05	2.02	15.04	37.60
C9	cantilever	2.1	6.4	3.05	22.65	47.57

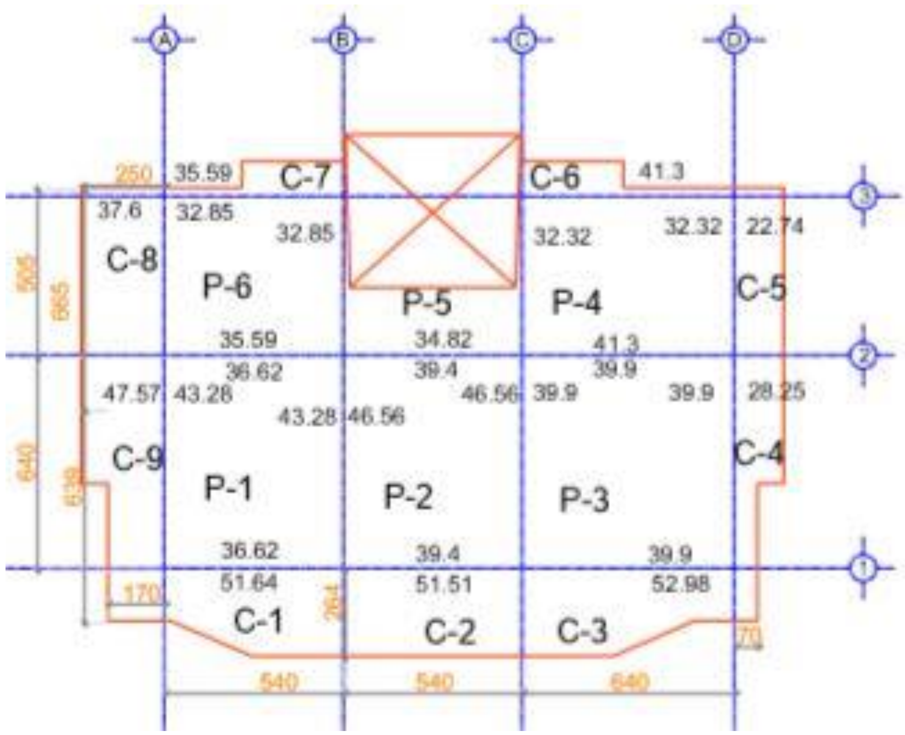


Figure A-11.3 load transfer from 3rd floor

Table A-11.4 Load transfer from Forth floor slab to beam

PANEL	TYPE	Lx	Ly	Ly/Lx	n(pd)	βvx	βvy	Vsx	Vsy
P1	Interior	5.4	6.4	1.19	20.22	0.39	0.33	42.58	36.03
P2	Interior	5.4	6.4	1.19	20.42	0.39	0.33	43.00	36.39
P3	Interior	6.4	6.4	1.00	19.53	0.33	0.33	41.25	41.25
P4	type 3	5.05	6.4	1.27	17.32	0.46	0.36	40.23	31.49
P6	type 4	5.05	5.4	1.07	18.58	0.36	0.33	33.78	30.96

PANEL	TYPE	Lx	Ly	Ly/Lx	n (Pd)	Wl
P5	cantilever	2.08	5.4	2.60	16.17	34.98
C 1	cantilever	2.64	5.4	2.05	16.17	56.35
C2	cantilever	2.64	5.4	2.05	16.17	56.35
C3	cantilever	2.64	6.4	2.42	16.17	56.35

C4	cantilever	1.5	6.4	4.27	13.92	15.66
C5	cantilever	1.5	5.05	3.37	15.51	17.45
C6	cantilever	2.5	5.05	2.02	13.26	41.44
C7	cantilever	2.5	6.4	2.56	16.17	50.53

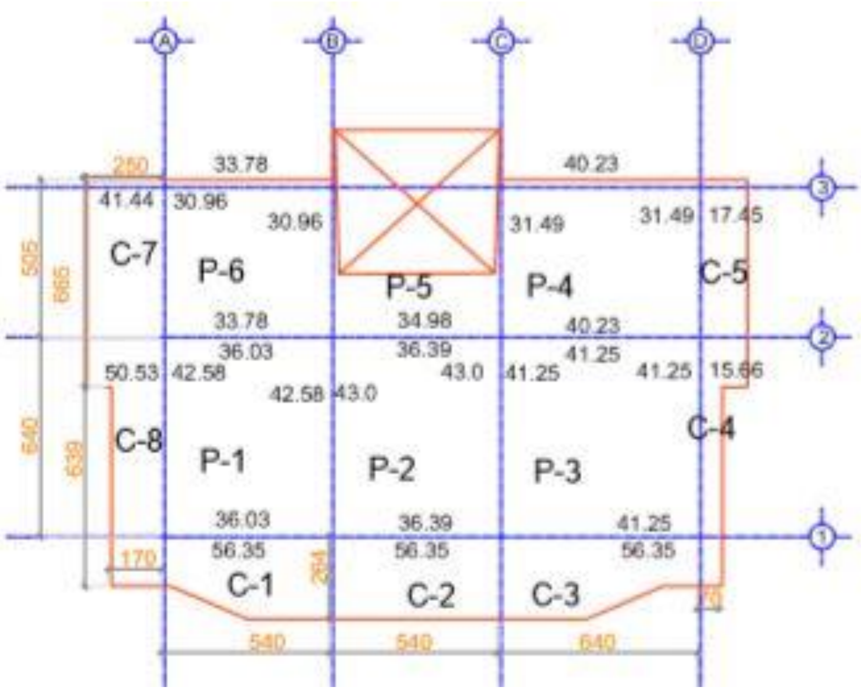


Figure A-11.3 load transfer from 4th floor

Appendix B- Beam

B-1 reinforcement for beam

Beam design on axis 2 for all story

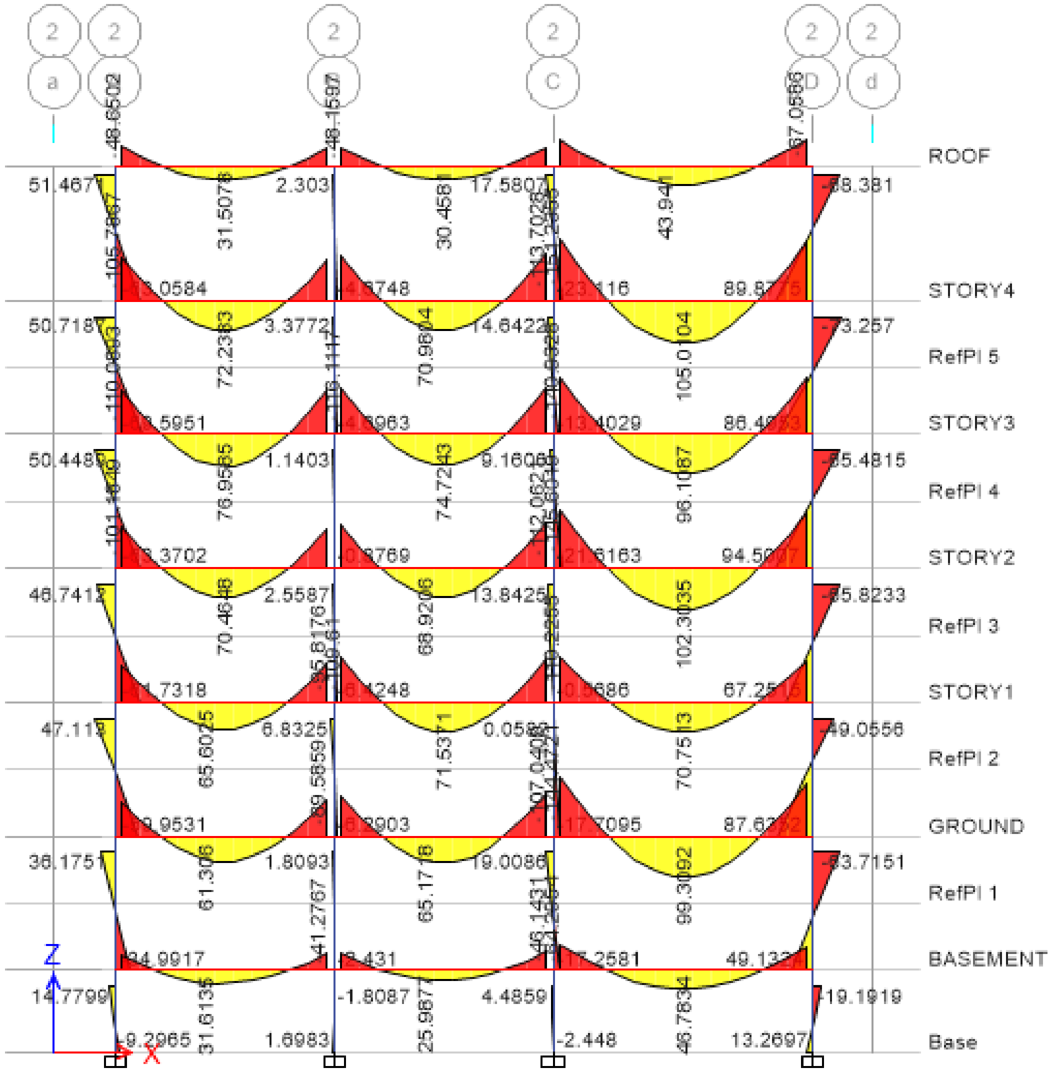


Figure-B-1.1 bending moment diagram of beam in axis 2

Table B-1.1 reinforcement of beam on axis 2

Axis 2	TOP TILE BEAM(ROOF)												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark

Support	A	48.65	300	375	0.058	354.8339	Single	146.25	4500	394.176	394.1764	1.255339	2Φ 20
Span	A-B	31.51	300	375	0.037	362.2045	Single	146.25	4500	250.108	250.108	0.796522	2Φ 20
Support	B	48.16	300	375	0.057	355.0491	Single	146.25	4500	389.97	389.9698	1.241942	2Φ 20
Span	B-C	30.458	300	375	0.036	362.6468	Single	146.25	4500	241.463	241.463	0.76899	2Φ 20
Support	C	113.703	300	375	0.135	323.2661	Single	146.25	4500	1011.22	1011.218	3.220439	4Φ 20
Span	C-D	43.941	300	375	0.052	356.8909	Single	146.25	4500	353.971	353.9708	1.127296	2Φ 20
Support	D	67.0566	300	375	0.079	346.5388	Single	146.25	4500	556.317	556.3174	1.771711	2Φ 20

Axis 2	SRORY 4												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	105.7867	300	375	0.125	327.4885	Single	146.25	4500	928.684	928.6839	2.957592	3Φ 20
Span	A-B	72.23	300	375	0.086	344.1283	Single	146.25	4500	603.435	603.4346	1.921766	2Φ 20
Support	B	87.48	300	375	0.104	336.7963	Single	146.25	4500	746.749	746.7485	2.37818	3Φ 20
Span	B-C	70.98	300	375	0.084	344.7141	Single	146.25	4500	591.984	591.9839	1.885299	2Φ 20
Support	C	151.23	300	375	0.179	301.1347	double	146.25	4500	1443.81	1443.81	4.598122	5Φ 20
Span	C-D	105.1	300	375	0.125	327.8488	Single	146.25	4500	921.642	921.6416	2.935164	3Φ 20
Support	D	89.49	300	375	0.106	335.8029	Single	146.25	4500	766.166	766.1662	2.44002	3Φ 20

Axis 2	SRORY 3												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	110.08	300	375	0.130	325.2146	Single	146.25	4500	973.131	973.131	3.099143	4Φ 20
Span	A-B	76.95	300	375	0.091	341.8962	Single	146.25	4500	647.064	647.0641	2.060714	3Φ 20
Support	B	116.11	300	375	0.138	321.956	Single	146.25	4500	1036.83	1036.827	3.301995	4Φ 20
Span	B-C	74.72	300	375	0.089	342.9548	Single	146.25	4500	626.373	626.3729	1.994818	2Φ 20
Support	C	140.03	300	375	0.166	308.1655	Single	146.25	4500	1306.38	1306.381	4.16045	5Φ 20
Span	C-D	96.1087	300	375	0.114	332.4837	Single	146.25	4500	831.047	831.0466	2.646645	3Φ 20
Support	D	86.49	300	375	0.103	337.2832	Single	146.25	4500	737.232	737.2319	2.347872	3Φ 20

Axis 2	SRORY 2												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark

Support	A	101.15	300	375	0.120	329.9035	Single	146.25	4500	881.479	881.4788	2.807257	3Φ 20
Span	A-B	70.46	300	375	0.084	344.9572	Single	146.25	4500	587.233	587.233	1.870169	2Φ 20
Support	B	112.06	300	375	0.133	324.1532	Single	146.25	4500	993.878	993.8785	3.165218	4Φ 20
Span	B-C	68.92	300	375	0.082	345.6748	Single	146.25	4500	573.206	573.2057	1.825496	2Φ 20
Support	C	145.6	300	375	0.173	304.7217	double	146.25	4500	1373.7	1373.697	4.374832	5Φ 20
Span	C-D	102.3	300	375	0.121	329.3084	Single	146.25	4500	893.112	893.1117	2.844305	3Φ 20
Support	D	94.5	300	375	0.112	333.2974	Single	146.25	4500	815.141	815.1413	2.595991	3Φ 20

Axis 2	SRORY 1												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	A	95.81	300	375	0.114	332.6351	Single	146.25	4500	828.087	828.0866	2.637218	3Φ 20
Span	A-B	65.6	300	375	0.078	347.2109	Single	146.25	4500	543.18	543.1795	1.729871	2Φ 20
Support	B	109.61	300	375	0.130	325.4654	Single	146.25	4500	968.23	968.2296	3.083534	4Φ 20
Span	B-C	71.53	300	375	0.085	344.4566	Single	146.25	4500	597.017	597.0169	1.901328	2Φ 20
Support	C	110.23	300	375	0.131	325.1345	Single	146.25	4500	974.697	974.6972	3.104131	4Φ 20
Span	C-D	70.75	300	375	0.084	344.8217	Single	146.25	4500	589.882	589.8816	1.878604	2Φ 20
Support	D	67.25	300	375	0.080	346.4493	Single	146.25	4500	558.066	558.0659	1.77728	2Φ 20

Axis 2	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	89.59	300	375	0.106	335.7533	Single	146.25	4500	767.136	767.1356	2.443107	3Φ 20
Span	A-B	61.3	300	375	0.073	349.1788	Single	146.25	4500	504.714	504.7142	1.60737	2Φ 20
Support	B	107.04	300	375	0.127	326.8286	Single	146.25	4500	941.584	941.584	2.998675	3Φ 20
Span	B-C	65.17	300	375	0.077	347.4088	Single	146.25	4500	539.312	539.3117	1.717553	2Φ 20
Support	C	144.47	300	375	0.171	305.4285	Double	146.25	4500	1359.88	1359.882	4.330834	5Φ 20
Span	C-D	99.3	300	375	0.118	330.8558	Single	146.25	4500	862.866	862.8663	2.747982	3Φ 20
Support	D	87.63	300	375	0.104	336.7224	Single	146.25	4500	748.193		2.382781	3Φ 20

Axis 2	BASEMENT												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	41.28	300	375	0.049	358.0422	Single	146.25	4500	331.466	331.4656	1.055623	2Φ 20
Span	A-B	31.61	300	375	0.037	362.1624	Single	146.25	4500	250.931	250.9309	0.799143	2Φ 20
Support	B	46.14	300	375	0.055	355.9334	Single	146.25	4500	372.685	372.6849	1.186894	2Φ 20
Span	B-C	25.98	300	375	0.031	364.517	Single	146.25	4500	204.906	204.9058	0.652566	2Φ 20
Support	C	64.28	300	375	0.076	347.8176	Single	146.25	4500	531.321	531.3214	1.692106	2Φ 20
Span	C-D	46.7834	300	375	0.055	355.6523	Single	146.25	4500	378.181	378.1805	1.204397	2Φ 20
Support	D	49.132	300	375	0.058	354.622	Single	146.25	4500	398.32	398.3197	1.268534	2Φ 20

Beam design on axis 3 for all story

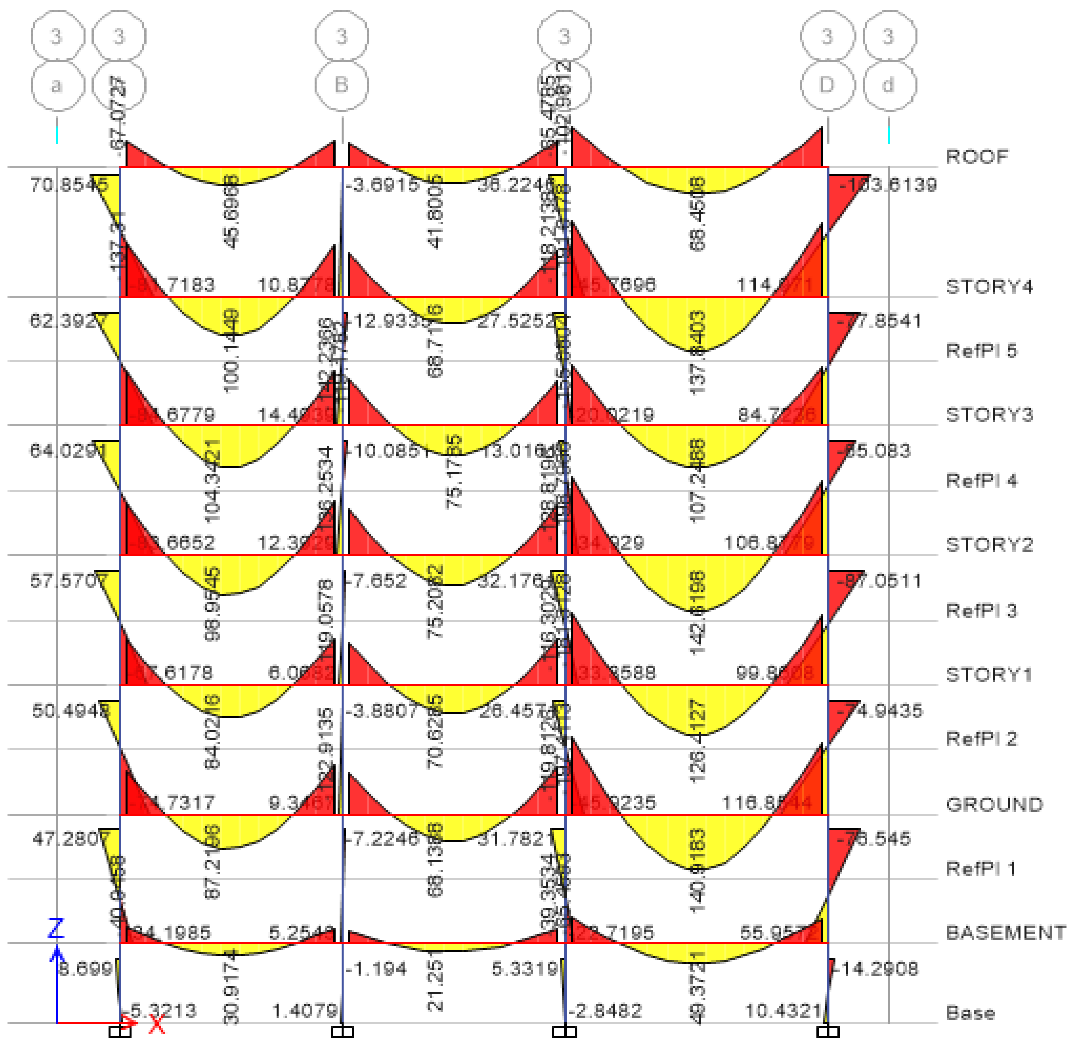


Figure- B-1.2 bending moment diagram of beam in axis 3

Table B-1.2 reinforcement of beam on axis 3

Axis 3	ROOF												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	67.07	300	375	0.079	346.5326	Single	146.25	4500	556.438	556.4385	1.772097	2Φ 20
Span	A-B	45.69	300	375	0.054	356.1298	Single	146.25	4500	368.847	368.8466	1.174671	2Φ 20
Support	B	65.47	300	375	0.078	347.2708	Single	146.25	4500	542.01	542.0097	1.726146	2Φ 20
Span	B-C	41.8	300	375	0.050	357.8178	Single	146.25	4500	335.851	335.8515	1.069591	2Φ 20
Support	C	102.98	300	375	0.122	328.9553	Single	146.25	4500	900.013	900.0133	2.866284	3Φ 20
Span	C-D	68.45	300	375	0.081	345.8932	Single	146.25	4500	568.937	568.9373	1.811902	2Φ 20
Support	D	103.6	300	375	0.123	328.6326	Single	146.25	4500	906.321	906.321	2.886373	3Φ 20

Axis 3	SRORY 4												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	137.31	300	375	0.163	309.812	Single	146.25	4500	1274.2	1274.198	4.057955	5Φ 20
Span	A-B	100.145	300	375	0.119	330.4216	Single	146.25	4500	871.352	871.3523	2.775007	3Φ 20
Support	B	118.21	300	375	0.140	320.8024	Single	146.25	4500	1059.37	1059.375	3.373805	4Φ 20
Span	B-C	68.71	300	375	0.081	345.7724	Single	146.25	4500	571.298	571.2978	1.81942	2Φ 20
Support	C	191.07	300	375	0.226	271.4824	double	146.25	4500	2023.41	2023.41	6.44398	7Φ 20
Span	C-D	137.84	300	375	0.163	309.493	Single	146.25	4500	1280.43	1280.435	4.077818	5Φ 20
Support	D	114.651	300	375	0.136	322.7516	Single	146.25	4500	1021.27	1021.274	3.252466	4Φ 20

Axis 3	SRORY 3												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	142.24	300	375	0.169	306.811	double	146.25	4500	1332.86	1332.858	4.24477	5Φ 20
Span	A-B	104.34	300	375	0.124	328.2465	Single	146.25	4500	913.868	913.8685	2.910409	3Φ 20
Support	B	110.17	300	375	0.131	325.1666	Single	146.25	4500	974.071	974.0706	3.102136	4Φ 20
Span	B-C	75.18	300	375	0.089	342.737	Single	146.25	4500	630.63	630.6295	2.008374	2Φ 20
Support	C	156.06	300	375	0.185	297.9646	double	146.25	4500	1505.77	1505.774	4.79546	5Φ 20
Span	C-D	107.25	300	375	0.127	326.7177	Single	146.25	4500	943.751	943.7514	3.005578	4Φ 20
Support	D	84.72	300	375	0.100	338.1498	Single	146.25	4500	720.294	720.2941	2.29393	3Φ 20

Axis 3	SRORY 2												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	136.25	300	375	0.161	310.4477	Single	146.25	4500	1261.77	1261.772	4.018383	5Φ 20

Span	A-B	98.95	300	375	0.117	331.0352	Single	146.25	4500	859.359	859.3589	2.736812	3Φ 20
Support	B	128.82	300	375	0.153	314.8145	Single	146.25	4500	1176.42	1176.418	3.746553	4Φ 20
Span	B-C	75.18	300	375	0.089	342.737	Single	146.25	4500	630.63	630.6295	2.008374	3Φ 20
Support	C	198.7	300	375	0.235	264.5116	double	146.25	4500	2159.66	2159.663	6.877909	7Φ 20
Span	C-D	142.62	300	375	0.169	306.5766	double	146.25	4500	1337.44	1337.44	4.259365	5Φ 20
Support	D	106.87	300	375	0.127	326.9183	Single	146.25	4500	939.831	939.8306	2.993091	3Φ 20

Axis 3	SRORY 1												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	119.06	300	375	0.141	320.3326	Single	146.25	4500	1068.56	1068.557	3.403047	4Φ 20
Span	A-B	84.02	300	375	0.100	338.4911	Single	146.25	4500	713.622	713.6223	2.272682	3Φ 20
Support	B	116.3	300	375	0.138	321.852	Single	146.25	4500	1038.86	1038.859	3.308467	4Φ 20
Span	B-C	70.62	300	375	0.084	344.8824	Single	146.25	4500	588.694	588.694	1.874822	2Φ 20
Support	C	181.3	300	375	0.215	279.6414	double	146.25	4500	1863.93	1863.929	5.936078	6Φ 20
Span	C-D	126.41	300	375	0.150	316.1991	Single	146.25	4500	1149.35	1149.354	3.660363	4Φ 20
Support	D	99.86	300	375	0.118	330.5682	Single	146.25	4500	868.487	868.4873	2.765883	3Φ 20

Axis 3	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm2)	As,max (mm2)	As,cal (mm2)	As,prov (mm2)	No.of bar	Remark
Support	A	122.91	300	375	0.146	318.1837	Single	146.25	4500	1110.56	1110.56	3.536817	4Φ 20
Span	A-B	87.22	300	375	0.103	336.9244	Single	146.25	4500	744.246	744.2462	2.370211	3Φ 20
Support	B	119.81	300	375	0.142	319.9168	Single	146.25	4500	1076.69	1076.686	3.428936	4Φ 20
Span	B-C	68.14	300	375	0.081	346.037	Single	146.25	4500	566.125	566.1252	1.802947	2Φ 20
Support	C	197.41	300	375	0.234	265.7338	double	146.25	4500	2135.77	2135.774	6.801828	7Φ 20
Span	C-D	140.91	300	375	0.167	307.628	Single	146.25	4500	1316.89	1316.888	4.193911	5Φ 20
Support	D	116.85	300	375	0.138	321.5506	Single	146.25	4500	1044.75	1044.75	3.327229	4Φ 20

Axis 3	BASEMENT												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	A	40.05	300	375	0.047	358.5718	Single	146.25	4500	321.114	321.1141	1.022656	2Φ 20
Span	A-B	30.92	300	375	0.037	362.4527	Single	146.25	4500	245.257	245.2569	0.781073	2Φ 20
Support	B	39.36	300	375	0.047	358.8682	Single	146.25	4500	315.321	315.3212	1.004208	2Φ 20
Span	B-C	21.25	300	375	0.025	366.4713	Single	146.25	4500	166.706	166.7063	0.530912	2Φ 20
Support	C	65.45	300	375	0.078	347.28	Single	146.25	4500	541.83	541.8298	1.725572	2Φ 20
Span	C-D	49.37	300	375	0.059	354.5172	Single	146.25	4500	400.367	400.3675	1.275056	2Φ 20
Support	D	55.95	300	375	0.066	351.5943	Single	146.25	4500	457.5	457.5001	1.457007	2Φ 20

Beam design on axis B for all story

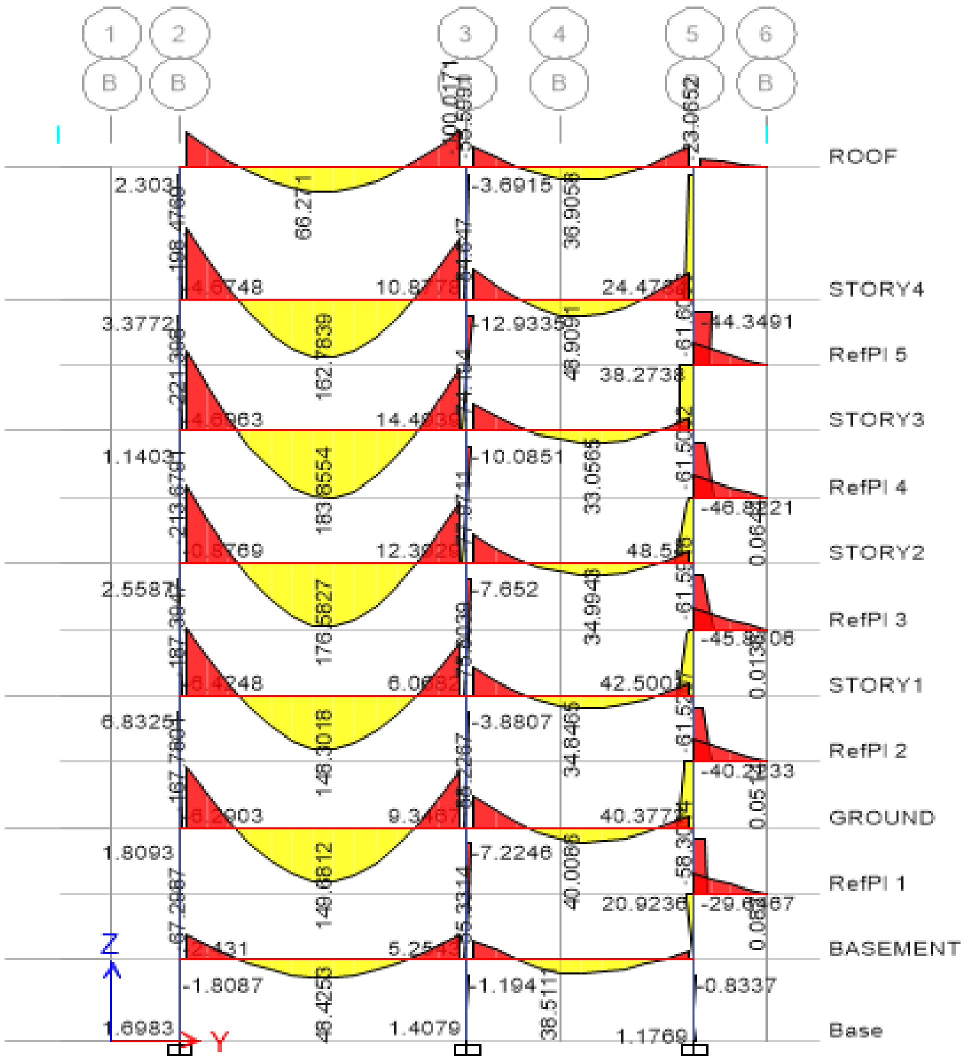


Figure B-1.3 moment force diagram of beam in axis B

Table B-1.4 reinforcement of beam on axis B

Axis B	ROOF												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	100.01	300	375	0.119	330.4911	Single	146.25	4500	869.995	869.995	2.771	3Φ 20
Span	2--3	66.271	300	375	0.079	346.9017	Single	146.25	4500	549.225	549.225	1.749	2Φ 20
Support	3	55.59	300	375	0.066	351.7556	Single	146.25	4500	454.348	454.348	1.447	2Φ 20
Span	3--5	36.905	300	375	0.044	359.9185	Single	146.25	4500	294.791	294.791	0.939	2Φ 20
Support	5	23.065	300	375	0.027	365.7239	Single	146.25	4500	181.315	181.315	0.577	2Φ 20

Axis B	SRORY 4												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	198.47	300	375	0.235	264.731	double	146.25	4500	2155.38	2155.376	6.864256	7Φ 20
Span	2--3	162.784	300	375	0.193	293.3935	double	146.25	4500	1595.12	1595.123	5.080009	6Φ 20
Support	3	84.647	300	375	0.100	338.1854	Single	146.25	4500	719.598	719.5976	2.291712	3Φ 20
Span	3--5	43.909	300	375	0.052	356.9047	Single	146.25	4500	353.699	353.6993	1.126431	2Φ 20
Support	5	24.47	300	375	0.029	365.1432	Single	146.25	4500	192.665	192.6654	0.613584	2Φ 20

Axis B	SRORY 3												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	221.308	300	375	0.262	238.5444	double	146.25	4500	2667.23	2667.232	8.49437	9Φ 20
Span	2--3	183.855	300	375	0.218	277.5791	double	146.25	4500	1904.24	1904.24	6.064458	7Φ 20
Support	3	74.104	300	375	0.088	343.2459	Single	146.25	4500	620.682	620.6821	1.976695	2Φ 20
Span	3--5	33.056	300	375	0.039	361.5525	Single	146.25	4500	262.852	262.8524	0.83711	2Φ 20
Support	5	38.27	300	375	0.045	359.3353	Single	146.25	4500	306.19	306.1904	0.975129	2Φ 20

Axis B	SRORY 2												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	213.679	300	375	0.253	248.5543	Double	146.25	4500	2471.57	2471.573	7.871252	8Φ 20
Span	2--3	176.583	300	375	0.209	283.3323	Double	146.25	4500	1791.78	1791.784	5.70632	6Φ 20
Support	3	77.67	300	375	0.092	341.5529	Single	146.25	4500	653.775	653.775	2.082086	3Φ 20
Span	3--5	34.99	300	375	0.041	360.7334	Single	146.25	4500	278.863	278.8628	0.888098	2Φ 20
Support	5	48.54	300	375	0.058	354.8823	Single	146.25	4500	393.232	393.2316	1.25233	2Φ 20

Axis B	SRORY 1												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	187.3	300	375	0.222	274.7212	Double	146.25	4500	1960.1	1960.101	6.242361	7Φ 20
Span	2--3	148.3	300	375	0.176	303.0153	Double	146.25	4500	1407.05	1407.05	4.481051	5Φ 20
Support	3	75.6	300	375	0.090	342.5379	Single	146.25	4500	634.521	634.5212	2.020768	3Φ 20
Span	3--5	34.48	300	375	0.041	360.9498	Single	146.25	4500	274.633	274.6335	0.874629	2Φ 20
Support	5	42.5	300	375	0.050	357.5153	Single	146.25	4500	341.765	341.7647	1.088423	2Φ 20

Axis B	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark

Support	2	167.78	300	375	0.199	289.8651	Double	146.25	4500	1664.09	1664.092	5.299655	6Φ 20
Span	2--3	149.68	300	375	0.177	302.1334	Double	146.25	4500	1424.29	1424.289	4.535951	5Φ 20
Support	3	68.22	300	375	0.081	345.9999	Single	146.25	4500	566.851	566.8507	1.805257	2Φ 20
Span	3--5	40.01	300	375	0.047	358.589	Single	146.25	4500	320.778	320.778	1.021586	2Φ 20
Support	5	40.38	300	375	0.048	358.4299	Single	146.25	4500	323.888	323.8882	1.031491	2Φ 20

Axis B	BASEMENT												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	67.297	300	375	0.080	346.4276	Single	146.25	4500	558.491	558.491	1.778634	2Φ 20
Span	2--3	48.425	300	375	0.057	354.9328	Single	146.25	4500	392.244	392.2441	1.249185	2Φ 20
Support	3	55.33	300	375	0.066	351.8719	Single	146.25	4500	452.073	452.0735	1.439724	2Φ 20
Span	3--5	38.511	300	375	0.046	359.2321	Single	146.25	4500	308.207	308.2071	0.981551	2Φ 20
Support	5	20.92	300	375	0.025	366.6069	Single	146.25	4500	164.057	164.0567	0.522474	2Φ 20

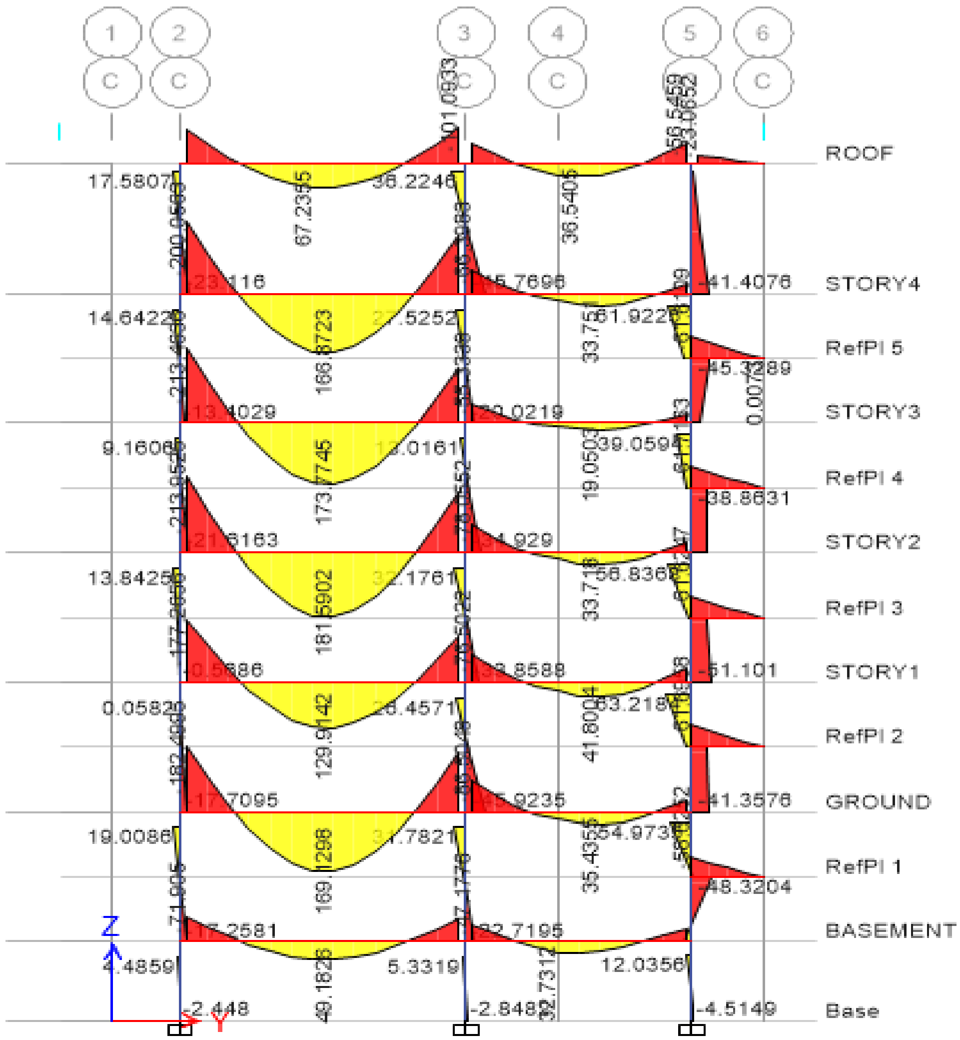


Figure B-1.4 moment force diagram of beam in axis C

Table B-1.5 reinforcement of beam on axis C

Axis C	ROOF											
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	101.09	300	375	0.120	329.9345	Single	146.25	4500	880.873	880.8732	2.805329	3Φ 20
Span	2--3	67.23	300	375	0.080	346.4586	Single	146.25	4500	557.885	557.8851	1.776704	2Φ 20
Support	3	58.54	300	375	0.069	350.4294	Single	146.25	4500	480.27	480.2696	1.529521	2Φ 20
Span	3--5	36.54	300	375	0.043	360.0741	Single	146.25	4500	291.749	291.7492	0.929137	2Φ 20

Support	5	23.07	300	375	0.027	365.7219	Single	146.25	4500	181.355	181.355	0.577564	2Φ 20

Axis C	STORY 4												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal (mm²)	As,prov (mm²)	No.of bar	Remark
Support	2	200.95	300	375	0.238	262.3322	double	146.25	4500	2202.26	2202.264	7.01358	8Φ 20
Span	2--3	166.87	300	375	0.198	290.5168	double	146.25	4500	1651.35	1651.353	5.259087	6Φ 20
Support	3	66.39	300	375	0.079	346.8468	Single	146.25	4500	550.298	550.2981	1.752542	2Φ 20
Span	3--5	33.75	300	375	0.040	361.259	Single	146.25	4500	268.589	268.5889	0.855379	2Φ 20
Support	5	41.4	300	375	0.049	357.9905	Single	146.25	4500	332.477	332.4772	1.058845	2Φ 20

Axis C	STORY 3												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal (mm²)	As,prov (mm²)	No.of bar	Remark
Support	2	213.45	300	375	0.253	248.8295	double	146.25	4500	2466.19	2466.194	7.85412	8Φ 20
Span	2--3	173.77	300	375	0.206	285.4672	double	146.25	4500	1750.05	1750.054	5.573421	6Φ 20
Support	3	55.39	300	375	0.066	351.8451	Single	146.25	4500	452.598	452.5982	1.441396	2Φ 20
Span	3--5	19.05	300	375	0.023	367.373	Single	146.25	4500	149.08	149.0804	0.474778	2Φ 20
Support	5	39.05	300	375	0.046	359.0012	Single	146.25	4500	312.722	312.7218	0.995929	2Φ 20

Axis C	STORY 2												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal (mm²)	As,prov (mm²)	No.of bar	Remark
Support	2	177.26	300	375	0.210	282.8114	double	146.25	4500	1801.97	1801.967	5.738748	6Φ 20
Span	2--3	181.59	300	375	0.215	279.4097	double	146.25	4500	1868.46	1868.459	5.950505	6Φ 20
Support	3	79.86	300	375	0.095	340.5038	Single	146.25	4500	674.28	674.28	2.147388	3Φ 20
Span	3--5	41.802	300	375	0.050	357.817	Single	146.25	4500	335.868	335.8684	1.069644	2Φ 20
Support	5	63.22	300	375	0.075	348.3031	Single	146.25	4500	521.831	521.8313	1.661883	2Φ 20

Axis C	STORY 1												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm²)	As,max (mm²)	As,cal (mm²)	As,prov (mm²)	No.of bar	Remark
Support	2	182.49	300	375	0.216	278.6867	double	146.25	4500	1882.59	1882.59	5.99551	6Φ 20

Span	2--3	129.91	300	375	0.154	314.1833	Single	146.25	4500	1188.76	1188.755	3.785845	4Φ 20
Support	3	76.5	300	375	0.091	342.1104	Single	146.25	4500	642.877	642.8774	2.04738	3Φ 20
Span	3--5	41.8	300	375	0.050	357.8178	Single	146.25	4500	335.851	335.8515	1.069591	2Φ 20
Support	5	54.97	300	375	0.065	352.0329	Single	146.25	4500	448.927	448.9267	1.429703	2Φ 20

Axis C	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	182.46	300	375	0.216	278.7109	double	146.25	4500	1882.12	1882.117	5.994004	6Φ 20
Span	2--3	169.12	300	375	0.200	288.8978	double	146.25	4500	1683	1682.998	5.359867	6Φ 20
Support	3	85.6	300	375	0.101	337.7196	Single	146.25	4500	728.703	728.7029	2.32071	3Φ 20
Span	3--5	35.43	300	375	0.042	360.5465	Single	146.25	4500	282.516	282.5159	0.899732	2Φ 20
Support	5	54.97	300	375	0.065	352.0329	Single	146.25	4500	448.927	448.9267	1.429703	2Φ 20

Axis C	GROUND												
Type	Loc.	Moment (KNm)	b	d	K	Z	Beam Type	As,min (mm^2)	As,max (mm^2)	As,cal (mm^2)	As,prov (mm^2)	No.of bar	Remark
Support	2	71.905	300	375	0.085	344.2808	Single	146.25	4500	600.453	600.4533	1.912272	2Φ 20
Span	2--3	49.1825	300	375	0.058	354.5997	Single	146.25	4500	398.754	398.7541	1.269917	2Φ 20
Support	3	47.17	300	375	0.056	355.4831	Single	146.25	4500	381.487	381.4871	1.214927	2Φ 20
Span	3--5	32.73	300	375	0.039	361.6902	Single	146.25	4500	260.161	260.161	0.828538	2Φ 20
Support	5	12.03	300	375	0.014	370.2207	Single	146.25	4500	93.4196	146.25	0.465764	2Φ 20

Shear on axis B

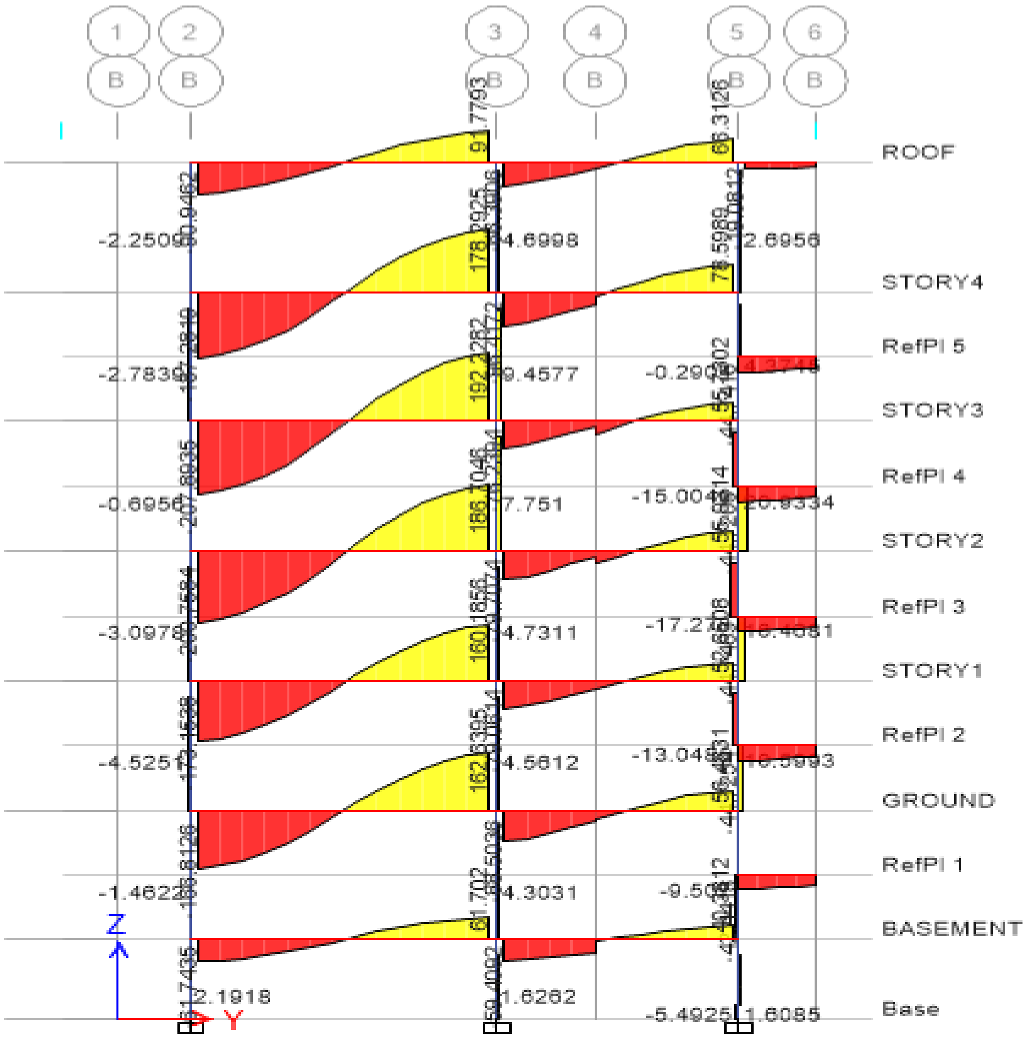


Figure-B-1.5 shear force diagram of beam in axis B

Table B-1.6 shear reinforcement of a beam on axis B

			ROOF							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	60.94	54.14	281.25	225	258.40	188.16	225	Φ 8 cc 210mm	
	Near 3	91.77	81.53	281.25	225	171.59	188.16	171.5912	Φ 8 cc 150mm	
3--5	Near 3	86.39	74.17	281.25	225	188.62	188.16	188.6184	Φ 8 cc 180mm	
	Near 5	66.31	56.93	281.25	225	245.74	188.16	225	Φ 8 cc 210mm	

			STORY 4							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	134.28	119.3	281.25	225	117.27	188.16	117.266	Φ 8 cc 110mm	
	Near 3	178.29	158.4	281.25	225	88.32	188.16	88.31964	Φ 8 cc 80mm	
3--5	Near 3	86.25	74.06	281.25	225	188.90	188.16	188.8986	Φ 8 cc 180mm	
	Near 5	78.59	67.47	281.25	225	207.35	188.16	207.3489	Φ 8 cc 210mm	

			STORY 3							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	207.89	184.697	281.25	225	75.74	188.16	75.74476	Φ 8 cc 70mm	
	Near 3	192.22	170.775	281.25	225	81.92	188.16	81.91966	Φ 8 cc 80mm	
3--5	Near 3	76.25	65.47	281.25	225	213.68	188.16	213.6831	Φ 8 cc 210mm	
	Near 5	55.7	47.825	281.25	225	292.52	188.16	225	Φ 8 cc 210mm	

			STORY 2							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	200.76	178.363	281.25	225	78.43	188.16	78.4346	Φ 8 cc 70mm	
	Near 3	166.7	148.103	281.25	225	94.46	188.16	94.46014	Φ 8 cc 80mm	
3--5	Near 3	79.25	65.47	281.25	225	213.68	188.16	213.6831	Φ 8 cc 210mm	
	Near 5	55.7	47.825	281.25	225	292.52	188.16	225	Φ 8 cc 210mm	

			STORY 1							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	173.15	153.83	281.25	225	90.94	188.16	90.94345	Φ 8 cc 80mm	
	Near 3	160.1856	142.315	281.25	225	98.30	188.16	98.30187	Φ 8 cc 80mm	
3--5	Near 3	79.045	67.865	281.25	225	206.14	188.16	206.1421	Φ 8 cc 210mm	
	Near 5	52.6	45.163	281.25	225	309.76	188.16	225	Φ 8 cc 210mm	

			GROUND							
Span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	166.81	148.2	281.25	225	94.40	188.16	94.39832	Φ 8 cc 80mm	
	Near 3	162.63	144.487	281.25	225	96.82	188.16	96.82415	Φ 8 cc 80mm	

3--5	Near 3	88.5	75.987	281.25	225	184.11	188.16	184.1082	Φ 8 cc 180mm
	Near 5	56.2	48.254	281.25	225	289.92	188.16	225	Φ 8 cc 180mm

			BESEMENT							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	174.35	154.899	281.25	225	90.32	188.16	90.31582	Φ 8 cc 80mm	
	Near 3	61.702	54.8184	281.25	225	255.20	188.16	225	Φ 8 cc 210mm	
3--5	Near 3	59.4	51	281.25	225	274.31	188.16	225	Φ 8 cc 210mm	
	Near 5	41.25	35.42	281.25	225	394.97	188.16	225	Φ 8 cc 210mm	

Shear on axis D

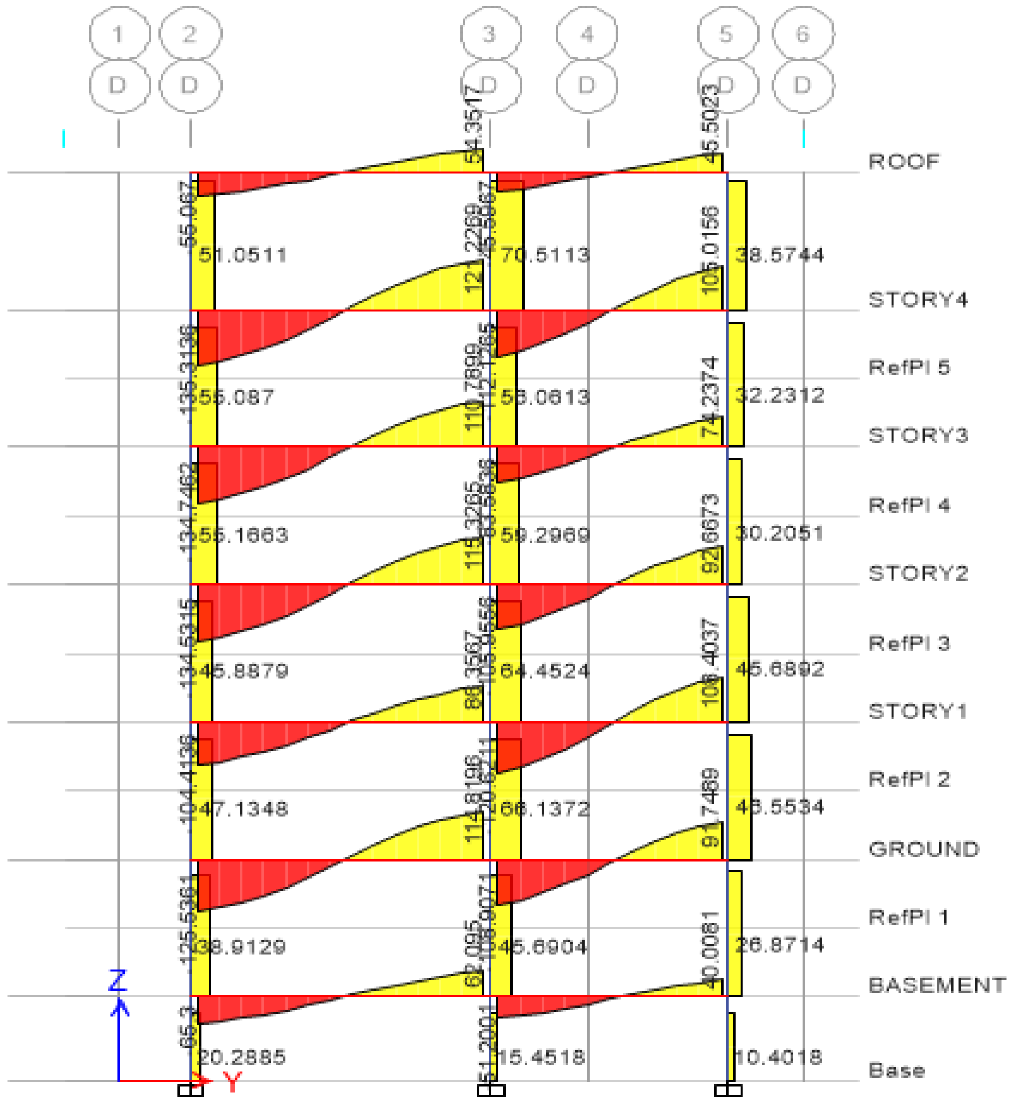


Figure-B-1.6 shear force diagram of beam in axis D

Table B-1.7 shear reinforcement of a beam on axis D

shear reinforcement of beam on axis 1				AXIS D						
			ROOF							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	55.067	74.158	281.25	225	188.65	188.16	188.649	Φ 8 cc 180mm	
	Near 3	54.35	54.62	281.25	225	256.13	188.16	225	Φ 8 cc 210mm	
3--5	Near 3	45.597	39.15	281.25	225	357.34	188.16	225	Φ 8 cc 210mm	
	Near 5	45.5	39.07	281.25	225	358.07	188.16	225	Φ 8 cc 210mm	

shear reinforcement of beam on axis 1				AXIS D						
		STORY 4								
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	135.32	120.223	281.25	225	116.37	188.16	116.3657	Φ 8 cc 110mm	
	Near 3	121.23	107.705	281.25	225	129.89	188.16	129.8903	Φ 8 cc 120mm	
3--5	Near 3	45.59	39.1442	281.25	225	357.39	188.16	225	Φ 8 cc 210mm	
	Near 5	45.502	39.0686	281.25	225	358.08	188.16	225	Φ 8 cc 210mm	

shear reinforcement of beam on axis 1				AXIS D						
		STORY 3								
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	134.31	119.823	281.25	225	116.75	188.16	116.7541	Φ 8 cc 110mm	
	Near 3	110.78	105.663	281.25	225	132.40	188.16	132.4005	Φ 8 cc 120mm	
3--5	Near 3	63.56	50.25	281.25	225	278.40	188.16	225	Φ 8 cc 210mm	
	Near 5	74.24	65.63	281.25	225	213.16	188.16	213.1621	Φ 8 cc 210mm	

shear reinforcement of beam on axis 1				AXIS D						
		STORY 2								
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	134.31	119.823	281.25	225	116.75	188.16	116.7541	Φ 8 cc 110mm	
	Near 3	115.32	106.32	281.25	225	131.58	188.16	131.5823	Φ 8 cc 120mm	
3--5	Near 3	105.95	90.9701	281.25	225	153.78	188.16	153.7849	Φ 8 cc 150mm	
	Near 5	92.67	79.57	281.25	225	175.82	188.16	175.8179	Φ 8 cc 170mm	

shear reinforcement of beam on axis 1				AXIS D						
		STORY 1								
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	104.414	92.77	281.25	225	150.80	188.16	150.8012	Φ 8 cc 150mm	
	Near 3	86.3567	76.7225	281.25	225	182.34	188.16	182.3433	Φ 8 cc 180mm	
3--5	Near 3	120.62	103.57	281.25	225	135.08	188.16	135.0761	Φ 8 cc 130mm	
	Near 5	108.404	93.08	281.25	225	150.30	188.16	150.299	Φ 8 cc 150mm	

shear reinforcement of beam on axis 1				AXIS D						
			GROUND							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	125.536	111.53	281.25	225	125.44	188.16	125.4356	Φ 8 cc 120mm	
	Near 3	114.82	102.01	281.25	225	137.14	188.16	137.1418	Φ 8 cc 130mm	
3--5	Near 3	108.9	93.503	281.25	225	149.62	188.16	149.6191	Φ 8 cc 140mm	
	Near 5	91.75	78.78	281.25	225	177.58	188.16	177.581	Φ 8 cc 170mm	

shear reinforcement of beam on axis 1				AXIS D						
			BASEMENT							
span	Loc	Vmax	Ved	Sl,max	Sb,max	Scalc	Smin	Sprov	Remark	
2--3	Near 2	65.3	58.015	281.25	225	241.14	188.16	225	Φ 8 cc 210mm	
	Near 3	62.095	55.167	281.25	225	253.59	188.16	225	Φ 8 cc 210mm	
3--5	Near 3	51.2	43.961	281.25	225	318.23	188.16	225	Φ 8 cc 210mm	
	Near 5	40.01	34.35	281.25	225	407.27	188.16	225	Φ 8 cc 210mm	

Appendix C- Column

C-1 Reinforcement Column

Table C-1.1 of reinforcement for column

Column	floor	Nsd (KN)	Msdx (KNm)	Msdy (KNm)	μsdx	μsdy	Vsd (KN)	ω	As (mm2)	Asmin (mm2)	Asmax (mm2)	no of bar	Provide
C1	Fourth	130.24	55.02	55.65	0.154	0.156	0.11	0.15	1207.7	37447	4200	3.85	4ϕ20
	Third	419.5	77.98	77.99	0.218	0.219	0.35	0.2	1610.3	210.7	4200	5.13	4ϕ20
	Second	732.75	68.03	78.03	0.191	0.219	0.62	0.35	2818.1	294	4200	8.97	9ϕ20
	First	1022.6	102.18	82.18	0.286	0.23	0.86	0.4	3220.6	367.8	4200	10.3	11ϕ20
	Ground	1279.3	85.54	85.54	0.24	0.24	1.08	0.45	3623.2	474.8	4200	11.5	12ϕ20
	Basement	1651.4	47.81	47.81	0.134	0.134	1.39	0.45	3623.2	474.8	4200	11.5	12ϕ20
C2	Fourth	217.8	55.02	55.65	0.154	0.156	0.18	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	677.8	48.26	58.26	0.135	0.163	0.57	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	1171.7	74.57	24.57	0.209	0.069	0.98	0.3	2415.5	336.9	4200	7.69	8ϕ20
	First	1684.8	38.83	39.12	0.109	0.11	1.42	0.3	2415.5	484.4	4200	7.69	8ϕ20
	Ground	2070.9	48.25	48.25	0.135	0.135	1.74	0.4	3220.6	595.4	4200	10.3	11ϕ20
	Basement	2657.1	54.94	55.54	0.154	0.156	2.23	0.45	3623.2	764	4200	11.5	12ϕ20
C3	Fourth	226.8	27.65	27.66	0.077	0.078	0.19	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	712.2	28.84	28.84	0.081	0.081	0.6	0.2	1610.3	344.4	4200	5.13	6ϕ20
	Second	1197.7	45.55	45.56	0.128	0.128	1.01	0.3	2415.5	484.2	4200	7.69	8ϕ20
	First	1684.2	47.52	47.52	0.133	0.133	1.42	0.35	2818.1	601.5	4200	8.97	9ϕ20
	Ground	2092.2	59.55	59.55	0.167	0.167	1.76	0.4	3220.6	788.9	4200	10.3	11ϕ20
	Basement	2743.8	59.38	59.38	0.166	0.166	2.31	0.4	3220.6	788.9	4200	10.3	11ϕ20
C4	Fourth	131.3	92.5	92.5	0.259	0.259	0.11	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	434.4	95.18	95.19	0.267	0.267	0.37	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	724.9	109	109	0.305	0.305	0.61	0.35	2818.1	293.3	4200	8.97	9ϕ20
	First	1020.1	129.99	129.99	0.364	0.364	0.86	0.4	3220.6	359.1	4200	10.3	11ϕ20
	Ground	1249	112.6	112.61	0.315	0.316	1.05	0.4	3220.6	483.9	4200	10.3	11ϕ20
	Basement	1683	52.86	52.86	0.148	0.148	1.41	0.45	3623.2	483.9	4200	11.5	12ϕ20
C5	Fourth	209.2	118.18	119.09	0.331	0.334	0.18	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	669.9	98.12	98.12	0.275	0.275	0.56	0.2	1610.3	303.8	4200	5.13	6ϕ20
	Second	1056.6	127.93	127.94	0.358	0.358	0.89	0.25	2012.9	434	4200	6.41	7ϕ20
	First	1509.6	129.99	129.99	0.364	0.364	1.27	0.3	2415.5	555.6	4200	7.69	8ϕ20
	Ground	1932.4	155.5	115.5	0.436	0.324	1.62	0.35	2818.1	737.7	4200	8.97	9ϕ20
	Basement	2565.7	65.61	127.85	0.184	0.358	2.16	0.45	3623.2	737.7	4200	11.5	12ϕ20
C6	Fourth	343.4	52.63	43.09	0.147	0.121	0.29	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	906.78	45.66	45.66	0.128	0.128	0.76	0.2	1610.3	260.7	4200	5.13	6ϕ20
	Second	1427.4	63.48	50.15	0.178	0.141	1.2	0.25	2012.9	410.4	4200	6.41	7ϕ20
	First	2026.8	74.34	80.34	0.208	0.225	1.7	0.3	2415.5	582.7	4200	7.69	8ϕ20
	Ground	2560.3	97.13	97.11	0.272	0.272	2.15	0.4	3220.6	736.1	4200	10.3	11ϕ20

	Basement	3343.4	72.2	72.2	0.202	0.202	2.81	0.45	3623.2	961.3	4200	11.5	12ϕ20
C7	Fourth	323.3	17.34	17.27	0.049	0.048	0.27	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	891.7	32.32	32.32	0.091	0.091	0.75	0.2	1610.3	256.4	4200	5.13	6ϕ20
	Second	1471.3	41.82	41.83	0.117	0.117	1.24	0.25	2012.9	423	4200	6.41	7ϕ20
	First	2041.5	48.48	48.48	0.136	0.136	1.72	0.25	2012.9	587	4200	6.41	7ϕ20
	Ground	2555.3	60.42	60.41	0.169	0.169	2.15	0.3	2415.5	734.7	4200	7.69	8ϕ20
	Basement	3292.3	67.26	73.07	0.188	0.205	2.77	0.3	2415.5	946.6	4200	7.69	8ϕ20
C8	Fourth	206	95.83	85.83	0.269	0.24	0.17	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	604	96.75	96.75	0.271	0.271	0.51	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	1035.6	104.37	109.37	0.292	0.306	0.87	0.3	2415.5	297.8	4200	7.69	8ϕ20
	First	1443.1	86.43	86.47	0.242	0.242	1.21	0.35	2818.1	414.9	4200	8.97	9ϕ20
	Ground	1857	111.89	111.49	0.314	0.312	1.56	0.35	2818.1	533.9	4200	8.97	8ϕ20
	Basement	2421.3	57.13	95.71	0.16	0.268	2.04	0.45	3623.2	696.2	4200	11.5	12ϕ20
C9	Fourth	106.8	47.81	47.81	0.134	0.134	0.09	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	299.5	50.71	50.71	0.142	0.142	0.25	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	496.68	54.94	54.94	0.154	0.154	0.42	0.3	2415.5	210	4200	7.69	8ϕ20
	First	687.87	50.62	50.61	0.142	0.142	0.58	0.3	2415.5	210	4200	7.69	8ϕ20
	Ground	874.04	47.53	58.95	0.133	0.165	0.73	0.4	3220.6	251.3	4200	10.3	11ϕ20
	Basement	1131.3	29.33	50.95	0.082	0.143	0.95	0.45	3623.2	325.3	4200	11.5	12ϕ20
C10	Fourth	203.4	28.54	28.54	0.08	0.08	0.17	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	538.8	55.13	55.13	0.154	0.154	0.45	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	850.3	65.51	65.51	0.184	0.184	0.71	0.25	2012.9	244.5	4200	6.41	7ϕ20
	First	1149.2	68.85	30.63	0.193	0.086	0.97	0.25	2012.9	330.4	4200	6.41	7ϕ20
	Ground	1426.6	68.9	68.9	0.193	0.193	1.2	0.35	2818.1	410.2	4200	8.97	9ϕ20
	Basement	1800	37.7	37.17	0.106	0.104	1.51	0.4	3220.6	517.5	4200	10.3	11ϕ20
C11	Fourth	210.7	45.61	45.62	0.128	0.128	0.18	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	545.2	72.82	72.8	0.204	0.204	0.46	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	816.9	55.4	55.4	0.155	0.155	0.69	0.25	2012.9	234.9	4200	6.41	7ϕ20
	First	1148.2	79.79	89.76	0.224	0.252	0.97	0.25	2012.9	330.1	4200	6.41	7ϕ20
	Ground	1526	93.73	93.74	0.263	0.263	1.28	0.3	2415.5	438.7	4200	7.69	8ϕ20
	Basement	2007.5	52.18	52.15	0.146	0.146	1.69	0.35	2818.1	577.2	4200	8.97	9ϕ20
C12	Fourth	106	69.26	69.27	0.194	0.194	0.09	0.15	1207.7	210	4200	3.85	4ϕ20
	Third	326.78	57.92	57.92	0.162	0.162	0.27	0.2	1610.3	210	4200	5.13	6ϕ20
	Second	483.24	65.76	65.76	0.184	0.184	0.41	0.3	2415.5	210	4200	7.69	8ϕ20
	First	692.07	90.94	90.94	0.255	0.255	0.58	0.3	2415.5	210	4200	7.69	8ϕ20
	Ground	938.46	94.34	64.81	0.264	0.182	0.79	0.4	3220.6	269.8	4200	10.3	11ϕ20
	Basement	1231.8	33.74	70.68	0.095	0.198	1.04	0.45	3623.2	354.2	4200	11.5	12ϕ20

C-2 detailing of column bar and footing pad

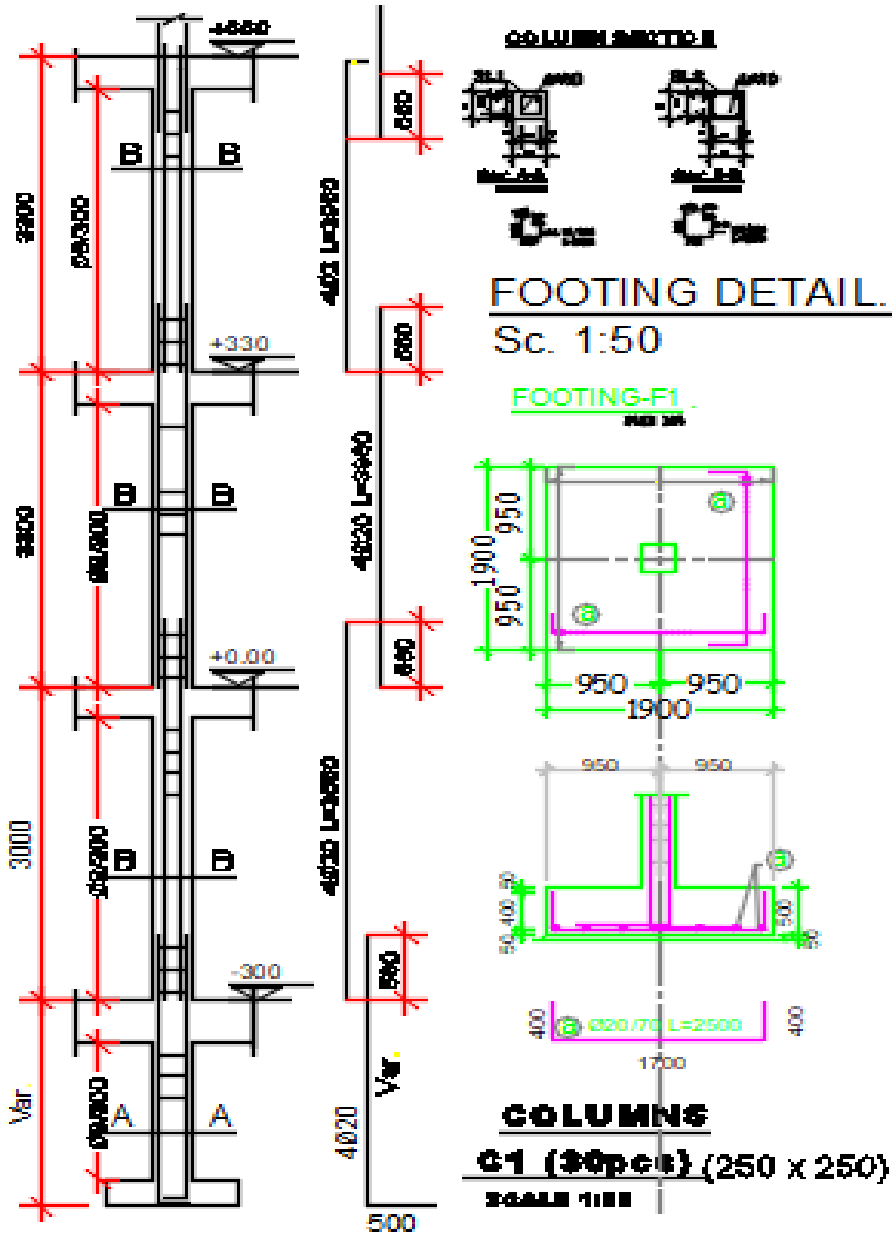


Figure C-2.1 of detailing of column and footing pad

Appendix D- Lateral load

D-1 Center mass of the floor

Table D-1.1 center of mass for ground floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.4	0.4	3	12	0	0	0	0
Axis-A3	0.4	0.4	3	12	0	6.4	76.8	0
Axis-A5	0.4	0.4	3	12	0	11.45	137.4	0
Axis-B2	0.4	0.4	3	12	5.4	0	0	64.8
Axis-B3	0.4	0.4	3	12	5.4	6.4	76.8	64.8
Axis-B5	0.4	0.4	3	12	5.4	11.45	137.4	64.8
Axis-C2	0.4	0.4	3	12	10.8	0	0	129.6
Axis-C3	0.4	0.4	3	12	10.8	6.4	76.8	129.6
Axis-C5	0.4	0.4	3	12	10.8	11.45	137.4	129.6
Axis-D2	0.4	0.4	3	12	17.2	0	0	206.4
Axis-D3	0.4	0.4	3	12	17.2	6.4	76.8	206.4
Axis-D5	0.4	0.4	3	12	17.2	111.45	1337.4	206.4
Total weight of Footing column				144			2056.8	1202.4
					Xm	8.35		
					Ym	14.28		

Table D-1.2 center of mass for first floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.3	0.35	3	7.875	0	0	0	0
Axis-A3	0.3	0.35	3	7.875	0	6.4	50.4	0
Axis-A5	0.3	0.35	3	7.875	0	11.45	90.16875	0
Axis-B2	0.3	0.35	3	7.875	5.4	0	0	42.525
Axis-B3	0.3	0.35	3	7.875	5.4	6.4	50.4	42.525
Axis-B5	0.3	0.35	3	7.875	5.4	11.45	90.16875	42.525
Axis-C2	0.3	0.35	3	7.875	10.8	0	0	85.05
Axis-C3	0.3	0.35	3	7.875	10.8	6.4	50.4	85.05

Axis-C5	0.3	0.35	3	7.875	10.8	11.45	90.16875	85.05
Axis-D2	0.3	0.35	3	7.875	17.2	0	0	135.45
Axis-D3	0.3	0.35	3	7.875	17.2	6.4	50.4	135.45
Axis-D5	0.3	0.35	3	7.875	17.2	111.45	877.6688	135.45
Total weight of Footing column				94.5			1349.775	789.075
					Xm	8.35		
					Ym	14.28		

Table D-1.3 center of mass for second floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.3	0.35	3	7.875	0	0	0	0
Axis-A3	0.3	0.35	3	7.875	0	6.4	50.4	0
Axis-A5	0.3	0.35	3	7.875	0	11.45	90.16875	0
Axis-B2	0.3	0.35	3	7.875	5.4	0	0	42.525
Axis-B3	0.3	0.35	3	7.875	5.4	6.4	50.4	42.525
Axis-B5	0.3	0.35	3	7.875	5.4	11.45	90.16875	42.525
Axis-C2	0.3	0.35	3	7.875	10.8	0	0	85.05
Axis-C3	0.3	0.35	3	7.875	10.8	6.4	50.4	85.05
Axis-C5	0.3	0.35	3	7.875	10.8	11.45	90.16875	85.05
Axis-D2	0.3	0.35	3	7.875	17.2	0	0	135.45
Axis-D3	0.3	0.35	3	7.875	17.2	6.4	50.4	135.45
Axis-D5	0.3	0.35	3	7.875	17.2	111.45	877.6688	135.45
Total weight of Footing column				94.5			1349.775	789.075
					Xm	8.35		
					Ym	14.28		

Table D-1.4 center of mass for third floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.3	0.35	3	7.875	0	0	0	0
Axis-A3	0.3	0.35	3	7.875	0	6.4	50.4	0
Axis-A5	0.3	0.35	3	7.875	0	11.45	90.16875	0
Axis-B2	0.3	0.35	3	7.875	5.4	0	0	42.525
Axis-B3	0.3	0.35	3	7.875	5.4	6.4	50.4	42.525
Axis-B5	0.3	0.35	3	7.875	5.4	11.45	90.16875	42.525
Axis-C2	0.3	0.35	3	7.875	10.8	0	0	85.05
Axis-C3	0.3	0.35	3	7.875	10.8	6.4	50.4	85.05
Axis-C5	0.3	0.35	3	7.875	10.8	11.45	90.16875	85.05
Axis-D2	0.3	0.35	3	7.875	17.2	0	0	135.45

Axis-D3	0.3	0.35	3	7.875	17.2	6.4	50.4	135.45
Axis-D5	0.3	0.35	3	7.875	17.2	111.45	877.6688	135.45
Total weight of Footing column				94.5			1349.775	789.075
					Xm	8.35		
					Ym	14.28		

Table D-1.5 center of mass for foundation floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.3	0.35	3	7.875	0	0	0	0
Axis-A3	0.3	0.35	3	7.875	0	6.4	50.4	0
Axis-A5	0.3	0.35	3	7.875	0	11.45	90.16875	0
Axis-B2	0.3	0.35	3	7.875	5.4	0	0	42.525
Axis-B3	0.3	0.35	3	7.875	5.4	6.4	50.4	42.525
Axis-B5	0.3	0.35	3	7.875	5.4	11.45	90.16875	42.525
Axis-C2	0.3	0.35	3	7.875	10.8	0	0	85.05
Axis-C3	0.3	0.35	3	7.875	10.8	6.4	50.4	85.05
Axis-C5	0.3	0.35	3	7.875	10.8	11.45	90.16875	85.05
Axis-D2	0.3	0.35	3	7.875	17.2	0	0	135.45
Axis-D3	0.3	0.35	3	7.875	17.2	6.4	50.4	135.45
Axis-D5	0.3	0.35	3	7.875	17.2	111.45	877.6688	135.45
Total weight of Footing column				94.5			1349.775	789.075
					Xm	8.35		
					Ym	14.28		

Table D-1.6 center of mass for basement floor

					Moment Arm		Moment	
Column on	Width(m)	Depth(m)	Height(m)	Weight(KN)	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis-A2	0.5	0.5	3	18.75	0	0	0	0
Axis-A3	0.5	0.5	3	18.75	0	6.4	120	0
Axis-A5	0.5	0.5	3	18.75	0	11.45	214.6875	0
Axis-B2	0.5	0.5	3	18.75	5.4	0	0	101.25
Axis-B3	0.5	0.5	3	18.75	5.4	6.4	120	101.25
Axis-B5	0.5	0.5	3	18.75	5.4	11.45	214.6875	101.25
Axis-C2	0.5	0.5	3	18.75	10.8	0	0	202.5
Axis-C3	0.5	0.5	3	18.75	10.8	6.4	120	202.5
Axis-C5	0.5	0.5	3	18.75	10.8	11.45	214.6875	202.5
Axis-D2	0.5	0.5	3	18.75	17.2	0	0	322.5
Axis-D3	0.5	0.5	3	18.75	17.2	6.4	120	322.5
Axis-D5	0.5	0.5	3	18.75	17.2	111.45	2089.688	322.5
Total weight of Footing column				225			3213.75	1878.75

					Xm	8.35		
					Ym	14.28		

D-2 Center of mass for beam

Table D-2.1 center of mass calculation for Ground floor

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.25	0.3	11.45	21.46875	0	11.45	245.8172	0
Axis B	2--5	0.25	0.3	11.45	21.46875	5.4	11.45	245.8172	115.9313
Axis C	2--5	0.25	0.3	11.45	21.46875	10.8	11.45	245.8172	231.8625
Axis D	2--5	0.25	0.3	11.45	21.46875	17.2	11.45	245.8172	369.2625
Axis 2	A-D	0.25	0.3	17.2	32.25	17.2	0	0	554.7
Axis 3	A-D	0.25	0.3	17.2	32.25	17.2	6.4	206.4	554.7
Axis 5	A-D	0.25	0.3	17.2	32.25	17.2	11.45	369.2625	554.7
					182.625			1558.931	2381.156
					Xm	13.0385			
					ym	8.536242			

Table D-2.2 center of mass calculation for 1st floor

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.25	0.3	11.45	21.46875	0	11.45	245.8172	0
Axis B	2--5	0.25	0.3	11.45	21.46875	5.4	11.45	245.8172	115.9313
Axis C	2--5	0.25	0.3	11.45	21.46875	10.8	11.45	245.8172	231.8625
Axis D	2--5	0.25	0.3	11.45	21.46875	17.2	11.45	245.8172	369.2625
Axis 2	A-D	0.25	0.3	17.2	32.25	17.2	0	0	554.7
Axis 3	A-D	0.25	0.3	17.2	32.25	17.2	6.4	206.4	554.7
Axis 5	A-D	0.25	0.3	17.2	32.25	17.2	11.45	369.2625	554.7
					182.625			1558.931	2381.156
					Xm	13.0385			
					ym	8.536242			

Table D-2.3 center of mass calculation for 2nd floor

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.25	0.3	11.45	21.46875	0	11.45	245.8172	0
Axis B	2--5	0.25	0.3	11.45	21.46875	5.4	11.45	245.8172	115.9313

Axis C	2--5	0.25	0.3	11.45	21.46875	10.8	11.45	245.8172	231.8625
Axis D	2--5	0.25	0.3	11.45	21.46875	17.2	11.45	245.8172	369.2625
Axis 2	A-D	0.25	0.3	17.2	32.25	17.2	0	0	554.7
Axis 3	A-D	0.25	0.3	17.2	32.25	17.2	6.4	206.4	554.7
Axis 5	A-D	0.25	0.3	17.2	32.25	17.2	11.45	369.2625	554.7
					182.625			1558.931	2381.156
					Xm	13.0385			
					ym	8.536242			

Table D-2.4 center of mass calculation for 3rd floor

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.25	0.3	11.45	21.46875	0	11.45	245.8172	0
Axis B	2--5	0.25	0.3	11.45	21.46875	5.4	11.45	245.8172	115.9313
Axis C	2--5	0.25	0.3	11.45	21.46875	10.8	11.45	245.8172	231.8625
Axis D	2--5	0.25	0.3	11.45	21.46875	17.2	11.45	245.8172	369.2625
Axis 2	A-D	0.25	0.3	17.2	32.25	17.2	0	0	554.7
Axis 3	A-D	0.25	0.3	17.2	32.25	17.2	6.4	206.4	554.7
Axis 5	A-D	0.25	0.3	17.2	32.25	17.2	11.45	369.2625	554.7
					182.625			1558.931	2381.156
					Xm	13.0385			
					ym	8.536242			

Table D-2.5 center of mass calculation for Roof floor

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis A	2--5	0.2	0.2	11.45	11.45	0	11.45	131.1025	0
Axis B	2--5	0.2	0.2	11.45	11.45	5.4	11.45	131.1025	61.83
Axis C	2--5	0.2	0.2	11.45	11.45	10.8	11.45	131.1025	123.66
Axis D	2--5	0.2	0.2	11.45	11.45	17.2	11.45	131.1025	196.94
Axis 2	A-D	0.2	0.2	17.2	17.2	17.2	0	0	295.84
Axis 3	A-D	0.2	0.2	17.2	17.2	17.2	6.4	110.08	295.84
Axis 5	A-D	0.2	0.2	17.2	17.2	17.2	11.45	196.94	295.84
					97.4			831.43	1269.95
					Xm	13.0385			
					ym	8.536242			

Table D-2.6 center of mass calculation for water tank

						moment arm		moment	
Beam on	B/n axis	width(m)	depth(m)	length(m)	weight(m)	x(m)	y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	2--3	0.2	0.2	5.05	5.05	5.4	11.45	57.8225	27.27
Axis c	2--3	0.2	0.2	5.05	5.05	10.8	11.45	57.8225	54.54
Axis 3	B-C	0.2	0.2	5.4	5.4	10.8	11.45	61.83	58.32
Axis 2	B-C	0.2	0.2	5.4	5.4	10.8	6.4	34.56	58.32
					20.9			212.035	198.45
					Xm	9.495215			
					Ym	10.14522			

D-3 Center of mass for slab

Table D-3.1 center of mass slab 1st floor

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	5.4	6.4	8.08	279.2448	5.4	6.4	1787.167	1507.922
P2	5.4	6.4	10.23	353.5488	10.8	6.4	2262.712	3818.327
P3	6.4	6.4	8.08	330.9568	17.2	6.4	2118.124	5692.457
P4	5.05	6.4	13.96	451.1872	17.2	11.45	5166.093	7760.42
P5	2.05	5.4	8.99	99.5193	10.8	8.45	840.9381	1074.808
P6	5.05	5.4	8.08	220.3416	5.4	11.45	2522.911	1189.845
C1	2.5	5.4	10.05	135.675	5.4	-2.5	-339.188	732.645
C2	2.5	5.4	9.97	134.595	10.8	-2.5	-336.488	1453.626
C3	2.5	8.9	10.2	226.95	19.7	-2.5	-567.375	4470.915
C4	2.5	6.4	10.85	173.6	19.7	6.4	1111.04	3419.92
C5	2.5	5.05	8.08	102.01	19.7	11.45	1168.015	2009.597
C6	2.5	5.4	8.08	109.08	10.4	14	1527.12	1134.432
C7	2.5	13.95	10.85	378.3938	-2.5	11.45	4332.608	-945.984
				2995.102			21593.68	33318.93
			Xm	11.12447				
			Ym	7.209663				

Table D-3.2 center of mass slab 2nd floor

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi

P1	5.4	6.4	13	449.28	5.4	6.4	2875.392	2426.112
P2	5.4	6.4	12.49	431.6544	10.8	6.4	2762.588	4661.868
P3	6.4	6.4	10.1	413.696	17.2	6.4	2647.654	7115.571
P4	5.05	6.4	10.95	353.904	17.2	11.45	4052.201	6087.149
P5	2.05	5.4	8.51	94.2057	10.8	8.45	796.0382	1017.422
P6	5.05	5.4	11.16	304.3332	5.4	11.45	3484.615	1643.399
C1	2.5	5.4	10.6	143.1	5.4	-2.5	-357.75	772.74
C2	2.5	5.4	10.56	142.56	10.8	-2.5	-356.4	1539.648
C3	2.5	8.9	10.98	244.305	19.7	-2.5	-610.763	4812.809
C4	2.5	6.4	10.06	160.96	19.7	6.4	1030.144	3170.912
C5	2.5	5.05	9.01	113.7513	19.7	11.45	1302.452	2240.9
C6	2.5	5.4	8.51	114.885	10.4	14	1608.39	1194.804
C7	2.5	13.95	8.51	296.7863	-2.5	11.45	3398.203	-741.966
				3263.421			22632.76	35941.37
			Xm	11.0134				
			Ym	6.935288				

Table D-3.3 center of mass slab 3rd floor

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	5.4	6.4	13	449.28	5.4	6.4	2875.392	2426.112
P2	5.4	6.4	12.49	431.6544	10.8	6.4	2762.588	4661.868
P3	6.4	6.4	10.1	413.696	17.2	6.4	2647.654	7115.571
P4	5.05	6.4	10.95	353.904	17.2	11.45	4052.201	6087.149
P5	2.05	5.4	8.51	94.2057	10.8	8.45	796.0382	1017.422
P6	5.05	5.4	11.16	304.3332	5.4	11.45	3484.615	1643.399
C1	2.5	5.4	10.6	143.1	5.4	-2.5	-357.75	772.74
C2	2.5	5.4	10.56	142.56	10.8	-2.5	-356.4	1539.648
C3	2.5	8.9	10.98	244.305	19.7	-2.5	-610.763	4812.809
C4	2.5	6.4	10.06	160.96	19.7	6.4	1030.144	3170.912
C5	2.5	5.05	9.01	113.7513	19.7	11.45	1302.452	2240.9
C6	2.5	5.4	8.51	114.885	10.4	14	1608.39	1194.804
C7	2.5	13.95	8.51	296.7863	-2.5	11.45	3398.203	-741.966
				3263.421			22632.76	35941.37
			Xm	11.0134				
			Ym	6.935288				

Table D-3.4 center of mass slab 4th floor

slab					moment arm		moment	

Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	5.4	6.4	11.09	383.2704	5.4	6.4	2452.931	2069.66
P2	5.4	6.4	11.24	388.4544	10.8	6.4	2486.108	4195.308
P3	6.4	6.4	10.58	433.3568	17.2	6.4	2773.484	7453.737
P4	5.05	6.4	10.61	342.9152	17.2	11.45	3926.379	5898.141
P5	2.05	5.4	8.09	89.5563	10.8	8.45	756.7507	967.208
P6	5.05	5.4	11.54	314.6958	5.4	11.45	3603.267	1699.357
C1	2.5	5.4	8.09	109.215	5.4	-2.5	-273.038	589.761
C2	2.5	5.4	8.09	109.215	10.8	-2.5	-273.038	1179.522
C3	2.5	8.9	8.09	180.0025	19.7	-2.5	-450.006	3546.049
C4	2.5	6.4	8.09	129.44	19.7	6.4	828.416	2549.968
C5	2.5	5.05	7.6	95.95	19.7	11.45	1098.628	1890.215
C6	2.5	5.4	7.6	102.6	10.4	14	1436.4	1067.04
C7	2.5	13.95	8.09	282.1388	-2.5	11.45	3230.489	-705.347
				2960.81			21596.77	32400.62
			Xm	10.94316				
			Ym	7.29421				

Table D-3.5 center of mass slab roof floor

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	6.4	7.1	10.2	463.488	5.4	6.4	2966.323	2502.835
P2	5.4	6.4	10.2	352.512	10.8	6.4	2256.077	3807.13
P3	6.4	6.4	10.2	417.792	17.2	6.4	2673.869	7186.022
P4	5.05	6.4	10.2	329.664	17.2	11.45	3774.653	5670.221
P5	5.05	5.4	10.2	278.154	10.8	8.45	2350.401	3004.063
P6	5.05	7.1	10.2	365.721	5.4	11.45	4187.505	1974.893
C1	2.64	5.4	10.2	145.4112	5.4	-2.5	-363.528	785.2205
C2	2.64	5.4	10.2	145.4112	10.8	-2.5	-363.528	1570.441
C3	2.64	6.4	10.2	172.3392	19.7	-2.5	-430.848	3395.082
C4	0.7	6.39	10.2	45.6246	19.7	6.4	291.9974	898.8046
C5	1.5	5.05	10.2	77.265	19.7	11.45	884.6843	1522.121
C6	1.6	5.4	10.2	88.128	10.4	14	1233.792	916.5312
C7	2.5	6.65	10.2	169.575	-2.5	11.45	1941.634	-423.938
				3051.085			21403.03	32809.43
			Xm	10.75336				
			Ym	7.014892				

Table D-3.6 Center of mass for water tank

slab					moment arm		moment	
Panel	Lx	Ly	DL	Weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
P1	5.05	5.4	9.64	262.8828	10.8	11.45	3010.008	2839.134
				Xm	10.8			
				Ym	11.45			

D-4 Center of mass for wall

Table D-4.1 center of mass for 1st floor wall

partition Wall						moment arm		moment	
wall on	b/n Axis	width	depth	length	weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	3&2	0.2	3	5.4	45.36	5.4	11.45	519.372	244.944
	2&1	0.2	3	1.6	13.44	5.4	6.4	86.016	72.576
Axis C	3&2	0.2	3	5.8	48.72	10.8	11.45	557.844	526.176
	1&2	0.2	3	1.6	13.44	10.8	6.4	86.016	145.152
Axis D	3&2	0.15	3	5.2	32.76	17.2	11.45	375.102	563.472
Axis A-B	3&2	0.15	3	2.8	17.64	2.5	11.45	201.978	44.1
Axis C-D	3&2	0.15	3	3.3	20.79	13.6	11.45	238.0455	282.744
Axis 2-3	C&D	0.2	3	5.02	42.168	13.8	8.6	362.6448	581.9184
	C&D	0.2	3	3	25.2	13.4	8.6	216.72	337.68
	C&D	0.15	3	2.56	16.128	12.6	8.4	135.4752	203.2128
	A&B	0.15	3	5.05	31.815	3.05	7.85	249.7478	97.03575
Axis B	Cantilever	0.2	3	2.5	21	5.4	14	294	113.4
	Cantilever	0.2	3	2.5	21	5.4	-2.5	-52.5	113.4
Axis C	Cantilever	0.2	3	2.5	21	10.8	14	294	226.8
Axis 2	Cantilever	0.2	3	2.5	21	-2.5	6.4	134.4	-52.5
	Cantilever	0.2	3	2.4	20.16	19.7	6.4	129.024	397.152
Axis 2-3	Cantilever	0.2	3	2.5	21	-2.5	11.45	240.45	-52.5
Axis 3	Cantilever	0.2	3	2.5	21	19.7	11.45	240.45	413.7
	Cantilever	0.2	3	2.4	20.16	-2.5	11.45	230.832	-50.4
Axis 3*	Cantilever	0.2	3	5.4	45.36	10.8	11.45	519.372	489.888
Axis A*	Cantilever	0.2	3	5.4	45.36	-2.5	11.45	519.372	-113.4
Axis D*	Cantilever	0.2	3	5.4	45.36	19.7	11.45	519.372	893.592
					609.861			6097.733	5478.143

Table D-4.2 Center of mass for 2nd floor wall

partition Wall						moment arm		moment	
wall on	b/n Axis	width	depth	length	weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	3&2	0.2	3	5.4	45.36	5.4	11.45	519.372	244.944

	2&1	0.2	3	1.9	15.96	5.4	6.4	102.144	86.184
Axis C	3&2	0.2	3	5.4	45.36	10.8	11.45	519.372	489.888
	1&2	0.2	3	1.7	14.28	10.8	6.4	91.392	154.224
Axis D	3&2	0.15	3	6	37.8	17.2	11.45	432.81	650.16
Axis A-B	3&2	0.15	3	2.5	15.75	2.5	11.45	180.3375	39.375
Axis C-D	3&2	0.15	3	3.9	24.57	13.6	11.45	281.3265	334.152
Axis 2-3	C&D	0.2	3	5.4	45.36	13.8	8.6	390.096	625.968
	C&D	0.2	3	3	25.2	13.4	8.6	216.72	337.68
	C&D	0.15	3	2.9	18.27	12.6	8.4	153.468	230.202
	A&B	0.15	3	3.8	23.94	3.05	7.85	187.929	73.017
Axis B	Cantilever	0.2	3	1.6	13.44	5.4	14	188.16	72.576
	Cantilever	0.2	3	1.6	13.44	5.4	-2.5	-33.6	72.576
Axis C	Cantilever	0.2	3	1.6	13.44	10.8	14	188.16	145.152
Axis 2	Cantilever	0.2	3	2.5	21	-2.5	6.4	134.4	-52.5
	Cantilever	0.2	3	2.4	20.16	19.7	6.4	129.024	397.152
Axis 2-3	Cantilever	0.2	3	2.5	21	-2.5	11.45	240.45	-52.5
Axis 3	Cantilever	0.2	3	2.5	21	19.7	11.45	240.45	413.7
	Cantilever	0.2	3	2.4	20.16	-2.5	11.45	230.832	-50.4
Axis A*	Cantilever	0.2	3	5.4	45.36	-2.5	11.45	519.372	-113.4
Axis D*	Cantilever	0.2	3	5.4	45.36	19.7	11.45	519.372	893.592
					546.21			5431.587	4991.742

Table D-4.3 center of mass for 3rd floor wall

partition Wall						moment arm		moment	
wall on	b/n Axis	width	depth	length	weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	3&2	0.2	3	5.4	45.36	5.4	11.45	519.372	244.944
	2&1	0.2	3	2	16.8	5.4	6.4	107.52	90.72
Axis C	3&2	0.2	3	5.4	45.36	10.8	11.45	519.372	489.888
	1&2	0.2	3	1.6	13.44	10.8	6.4	86.016	145.152
Axis D	3&2	0.15	3	2.5	15.75	17.2	11.45	180.3375	270.9
Axis A-B	3&2	0.15	3	2.1	13.23	2.5	11.45	151.4835	33.075
Axis C-D	3&2	0.15	3	3.6	22.68	13.6	11.45	259.686	308.448
Axis 2-3	C&D	0.2	3	5.02	42.168	13.8	8.6	362.6448	581.9184
	C&D	0.2	3	3.2	26.88	13.4	8.6	231.168	360.192
	C&D	0.15	3	2.4	15.12	12.6	8.4	127.008	190.512
	A&B	0.15	3	3.05	19.215	3.05	7.85	150.8378	58.60575
Axis B	Cantilever	0.2	3	2.5	21	5.4	14	294	113.4
	Cantilever	0.2	3	2.5	21	5.4	-2.5	-52.5	113.4
Axis C	Cantilever	0.2	3	2.5	21	10.8	14	294	226.8
Axis 2	Cantilever	0.2	3	2.5	21	-2.5	6.4	134.4	-52.5
	Cantilever	0.2	3	2.4	20.16	19.7	6.4	129.024	397.152

Axis 2-3	Cantilever	0.2	3	2.5	21	-2.5	11.45	240.45	-52.5
Axis 3	Cantilever	0.2	3	2.5	21	19.7	11.45	240.45	413.7
	Cantilever	0.2	3	2.4	20.16	-2.5	11.45	230.832	-50.4
Axis A*	Cantilever	0.2	3	5.4	45.36	-2.5	11.45	519.372	-113.4
Axis D*	Cantilever	0.2	3	5.4	45.36	19.7	11.45	519.372	893.592
					533.043			5244.846	4663.599

Table D-4.4 center of mass for 4th floor wall

partition Wall						moment arm		moment	
wall on	b/n Axis	width	depth	length	weight	X(m)	Y(m)	Mx=Wi*Yi	My=Wi*Xi
Axis B	3&2	0.2	3	5.4	45.36	5.4	11.45	519.372	244.944
	2&1	0.2	3	1.9	15.96	5.4	6.4	102.144	86.184
Axis C	3&2	0.2	3	5.4	45.36	10.8	11.45	519.372	489.888
	1&2	0.2	3	1.9	15.96	10.8	6.4	102.144	172.368
Axis D	3&2	0.15	3	3	18.9	17.2	11.45	216.405	325.08
Axis A-B	3&2	0.15	3	2.1	13.23	2.5	11.45	151.4835	33.075
Axis C-D	3&2	0.15	3	3.3	20.79	13.6	11.45	238.0455	282.744
Axis 2-3	C&D	0.2	3	5.02	42.168	13.8	8.6	362.6448	581.9184
	C&D	0.2	3	3	25.2	13.4	8.6	216.72	337.68
	C&D	0.15	3	2.26	14.238	12.6	8.4	119.5992	179.3988
	A&B	0.15	3	3.05	19.215	3.05	7.85	150.8378	58.60575
Axis B	Cantilever	0.2	3	1.6	13.44	5.4	14	188.16	72.576
	Cantilever	0.2	3	1.6	13.44	5.4	-2.5	-33.6	72.576
Axis C	Cantilever	0.2	3	1.6	13.44	10.8	14	188.16	145.152
Axis 2	Cantilever	0.2	3	2.5	21	-2.5	6.4	134.4	-52.5
	Cantilever	0.2	3	2.4	20.16	19.7	6.4	129.024	397.152
Axis 2-3	Cantilever	0.2	3	2.5	21	-2.5	11.45	240.45	-52.5
Axis 3	Cantilever	0.2	3	2.5	21	19.7	11.45	240.45	413.7
	Cantilever	0.2	3	2.4	20.16	-2.5	11.45	230.832	-50.4
Axis 1*	Cantilever	0.2	3	17.2	144.48	-2.5	11.45	1654.296	-361.2
Axis 3*	Cantilever	0.2	3	5.4	45.36	19.7	-2.5	-113.4	893.592
Axis A*	Cantilever	0.2	3	5.4	45.36	-2.5	11.45	519.372	-113.4
Axis D*	Cantilever	0.2	3	5.4	45.36	19.7	11.45	519.372	893.592
					700.581			6596.284	5050.226

D-5 Requirements from ES EN

Table 3.1: Ground types

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (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

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Table 3.4: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

Table 4.3 Importance classes for 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 EN 1990:2002, Annex B.