TRIBHUVAN UNIVERSITY
INSTITUTE OF ENGINEERING
PULCHOWK CAMPUS
DEPARTMENT OF CIVIL ENGINEERING

FINAL YEAR PROJECT REPORT on STUDY OF EARTHQUAKE RESISTANT ANALYSIS AND DESIGN OF MULTI-STOREY BUILDING

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Supervisor:

Asst. Prof. Sunita Ghimire

MAY 2023

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FINAL YEAR PROJECT REPORT on
STUDY OF
EARTHQUAKE RESISTANT ANALYSIS AND DESIGN
OF
MULTI-STOREY BUILDING

IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR DEGREE IN CIVIL ENGINEERING
(Course Code: CE755)

By:

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CERTIFICATE

This is to certify that this project work entitled **"STUDY OF EARTHQUAKE RESISTANT ANALYSIS AND DESIGN OF MULTISTORY BUILDING"** has been examined and declared successful for the fulfillment of academic requirement towards the completion of Bachelor Degree in Civil Engineering.

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We would like to express our gratitude to all those who have contributed towards the successful completion of this project report titled "Study of Earthquake Resistance Analysis and Design of a Multi-storey Building."

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We would also like to acknowledge the assistance provided by our colleagues and peers, who have been a constant source of support and encouragement throughout this project. Their insights and suggestions have been instrumental in shaping this report.

Finally, we would like to thank our families for their unwavering support and encouragement, which has been the driving force behind the successful completion of this project.

In conclusion, we extend our sincere gratitude to everyone who has played a role, directly or indirectly, in the completion of this project. We hope that this report will serve as a valuable resource for future research in this field.

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ABSTRACT

This project report presents a study on earthquake-resistant analysis and design of a Multi-storey building, located in a high-risk seismic zone in Nepal. The report covers a comprehensive analysis of the building's seismic behavior using two widely used analysis methods - Equivalent Static Method and Modal Response Spectrum Model.

The analysis includes a detailed seismic hazard assessment, dynamic analysis, and design of the building's structural components, such as columns, beams, and slabs. The design of the building is based on the NBC codes, including NBC-205:2017 and NBC-206:2017, and IS codes, including IS 1893:2016, IS 13920:2016, and IS 456:2000.

The report includes a detailed study of earthquake-resistant design techniques, such as ductility, energy dissipation, and their effectiveness in reducing the seismic vulnerability of the building. The study concludes that the use of ductile detailing and energy dissipation systems in the building's design improves its seismic performance and reduces the risk of collapse during an earthquake.

The project aims to provide a comprehensive understanding of earthquake-resistant design principles and their application in the design of Multi-storey buildings. The report can serve as a valuable resource for civil engineering students and professionals, providing them with practical knowledge of earthquake-resistant design principles.

The findings of this study can contribute to improving the seismic performance of buildings in high-risk seismic zones, reducing the risk of loss of life and property during an earthquake.

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NOTATIONS

f_{ck}	Compressive Strength of Concrete
f_y	Yield Strength of Steel
A_{st}	Area of Steel in Tension
M	Moment
V	Shear Force
ϕ	Diameter of Reinforcement Bars
M_{11}	Moment along Local 1 Axis of Element
M_{22}	Moment along Local 2 Axis of Element
S_{11}	Axial Stress along Local 1 Axis of Element
R	Response Reduction Factor
Ω	Over strength Factor
P	Axial Force
ρ	Percentage of reinforcement
V_b	Base Shear
h_w	Height of Wall
l_w	Length of Wall
τ_c	Shear Strength of Concrete
τ_{max}	Maximum allowable shear stress in concrete
J	Polar Moment of Inertia
F_x	Force in Horizontal X direction
F_y	Force in Horizontal Y direction
F_z	Force in Vertical Direction
e_x, e_y	Eccentricity in X and Y direction
M_u	Moment Resistance Capacity
A_g	Gross Area

ABBREVIATIONS

SYMBOL	Meaning
α_x, α_y	BM coefficients for Rectangular Slab Panels
ϕ	Diameter of Bar, Angle of internal friction of soil
δ_m	Percentage reduction in moment
τ_c	Shear Stress in Concrete
$\tau_{c,max}$	Max. shear stress in concrete with shear reinforcement
τ_{bd}	Design Bond Stress
σ_{ac}	Permissible Stress in Axial Compression (Steel)
σ_{cbc}	Permissible Bending Compressive Strength of Concrete
σ_{sc}, σ_{st}	Permissible Stress in Steel in Compression and Tension respectively
γ_m	Partial Safety Factor for Material
γ_f	Partial Safety Factor for Load
γ	Unit Weight of Material
A_B	Area of Each Bar
A_G	Gross Area of Concrete
A_H	Horizontal Seismic Coefficient
A_{SC}	Area of Steel in Compression
A_{ST}	Area of Steel in Tension
A_{SV}	Area of Stirrups
B	Width or shorter dimension in plan
B_F	Effective width of flange
D	Effective Depth
D′	Effective Cover
D	Overall Depth
D_F	Thickness of Flange
E_X	Eccentricity along x-direction
E_Y	Eccentricity along y-direction
E_C	Modulus of Elasticity of Concrete
E_S	Modulus of Elasticity of Steel

EL_X, EL_Y	Earthquake Load along X and Y direction respectively
F_{BR}	Bearing stress in concrete
F_{CK}	Characteristics Strength of Concrete
F_Y	Characteristic Strength of Steel
H	Height of building
H	Height of underground water tank
I	Importance Factor (For Base Shear Calculation)
I_{XX}, I_{YY}	Moment of Inertia (along x and y direction)
J	Neutral Axis Depth Factor
K	Coefficient of Constant or factor
k_1, k_2, k_3	Coefficient for wind pressure
K_A, K_P	Active and Passive Earth Pressure
L	Length of Member
L_{EF}	Effective Length of member
L_D	Development Length
M	Modular Ratio
M OR BM	Bending Moment
N_U OR P_U	Ultimate Axial Load on a compression member
P_C	Percentage of Compression Reinforcement
P_T	Percentage of Tension Reinforcement
P_Z	Wind Pressure
Q, Q_U	Permissible and Ultimate bearing capacity of soil
Q_I	Design Lateral Force in i^{th} Level
SR, R_{MIN}	Slenderness Ratio, (minimum) for structural steel section
R	Response Reduction Factor
S_a/g	Average Response Acceleration Coefficient
S_V	Spacing of Each Bar
T_I	Torsional Moment due to Lateral Force in i-direction
T_A	Fundamental Natural Period of Vibrations
V_B	Basic wind speed
V_Z	Design wind speed
V_B	Design Seismic Base Shear
V	Shear Force

W_I	Seismic Weight of i^{th} Floor
W_L	Wind Load
X_U	Actual Depth of Neutral Axis
X_{UL}	Ultimate Depth of Neutral Axis
Z	Seismic Zone Factor
CM	Center of Mass
CR	Center of Rigidity
D.L	Dead Load
HSDB	High Strength Deformed Bars
IS	Indian Standard
L.L	Live Load
RCC	Reinforced Cement Concrete
SPT, N	Standard Penetration Test
M25	Grade of Concrete
Fe500	Grade of Steel

1. INTRODUCTION

1.1 Background

The need for earthquake-resistant buildings in Nepal is of critical importance due to the country's high vulnerability to earthquakes. Nepal is located at the junction of the Indian and Eurasian tectonic plates and has a long history of devastating earthquakes, making it one of the most seismically active countries in the world. The country has experienced several earthquakes of magnitude 6.0 or greater in the past century, and the risk of large earthquakes in Nepal continues to increase due to the active tectonic plate boundary that runs through the region.

The most recent major earthquake that struck Nepal occurred in April 2015 and had a magnitude of 7.8 on the Richter scale. The earthquake was the largest to hit the country in over 80 years and resulted in widespread damage to buildings and infrastructure. The quake caused 9,000 casualties, injured 22,000 people, and left hundreds of thousands of people displaced. The financial losses due to the earthquake were estimated to be around $7 billion, which is equivalent to almost one-third of Nepal's gross domestic product (GDP).

The lack of earthquake-resistant buildings in Nepal was a significant factor in the extent of the damage caused by the 2015 earthquake. Many of the buildings in the country are poorly constructed and do not meet the necessary seismic standards, making them highly vulnerable to collapse during earthquakes. In the aftermath of the 2015 earthquake, it was estimated that over 70% of the buildings in the affected areas were either damaged or destroyed, and a large portion of these buildings were unreinforced masonry structures that were not designed to withstand seismic forces.

The need for earthquake-resistant buildings in Nepal is urgent and has been recognized by the government and international organizations. There is a growing emphasis on incorporating seismic considerations into the design and construction of new buildings, as well as retrofitting existing buildings to make them more earthquake-resistant. This is being achieved through the implementation of building codes, regulations, and standards that set minimum requirements for seismic resistance. The Nepal government has revised its building code to include provisions for seismic resistance, and has launched a nationwide campaign to educate builders and the public on the importance of earthquake-resistant construction.

In addition to implementing building codes and regulations, there is a growing awareness of the importance of seismic resistance in Nepal. Many non-governmental organizations and international aid organizations are working to promote earthquake-resistant construction and provide training and resources to local builders and communities. This includes providing technical assistance and funding for the retrofit of existing buildings, as well as providing training and education on seismic design and construction practices.

In conclusion, the need for earthquake-resistant buildings in Nepal is essential in order to reduce the risk of loss of life and property during earthquakes. The implementation of building codes, regulations, and standards, along with increased awareness of the importance of seismic resistance, will play a crucial role in reducing the impact of earthquakes in the future. However, much work still needs to be done in order to promote earthquake-resistant construction and build a more resilient and sustainable built environment in Nepal. This will require a sustained commitment from the government, international organizations, and the private sector to invest in the capacity of local builders and communities, as well as in the development of seismic-resistant technologies and practices.

1.2 Title and Theme of the Project Work

The proposed project is "**Earthquake Resistant Analysis and Design of Multi-Storey Building**". Our project lies in Nepal and it lies in seismic zone 'V' according to its seismic severity. Earthquake load dominates wind load and governs the lateral design loading. The report strictly follows Indian Standards with limit state design philosophy in general. The estimation of live and dead load could be predicted with reasonable accuracy but the loads due to earthquake can't be accurately predicted. So, statistical and probabilistic approaches are resorted to, considering one of factor economy. The seismic coefficient design method as stipulated in IS 1893:2016 is applied to analyze the building for earthquake. The 3- dimensional moment resistance frame is considered as the main structural system of the building.

This group under the project has undertaken the structural analysis and design of multi-storey framed building. The main aim of the project working under this title is to acquire the knowledge and skill to emphasis on practical application besides the utilization of analytical methods and design approaches, exposure and application of various available codes of practices.

1.3 Objective

The specific objectives of the project work are:

1. To analyze and design a multi-storey earthquake-resistant building.
2. To study architectural drawings and fix the structural system of the building to carry live load, dead load, and lateral loads.
3. To calculate loads, including lateral loads, and design structural elements based on the identified loads and load cases.
4. To determine the fundamental time period of the building using ETABS.
5. To calculate shear force and bending moment to determine the size of building components.
6. To review analysis output and design different components, such as beams, columns, and slabs, using the limit state method and following different applicable codes.
7. To design the staircase, shear wall, and foundation with appropriate loading following applicable codes.
8. To prepare final detailing of individual members and drawings applicable in the field.

1.4 Scope

The project work aims to provide a comprehensive analysis and design of a multi-storey earthquake-resistant building. The scope of the project includes a study of architectural drawings and fixing the structural system of the building to carry live load, dead load, and lateral loads. The project also involves the calculation of loads, including lateral loads, and preliminary design of structural elements, as well as the identification of loads and load cases.

The project requires the familiarity with different software for the structural analysis of the building, specifically ETABS for different cases of loads. Determination of the fundamental time period by ETABS is also necessary. Calculation of shear force and bending moment is crucial in determining the size of the building components.

Furthermore, the project requires a review of analysis output for the design of different components, such as beam, column, and slab, by limit state method while following different applicable codes. The project also includes the design of staircase, shear wall, and foundation with appropriate loading following applicable codes, as well as final detailing of individual members and preparation of drawing to be applicable in the field.

The field scope of the project involves the construction of Multi-storey buildings, which is crucial in accommodating people who come to the city in search of a better life, particularly in a developing nation such as Nepal. The project also takes into account the challenges in construction due to the lack of availability of materials, proper transportation, and other factors.

Considering that Nepal is an earthquake-prone area, the project emphasizes the importance of seismic design of buildings to counteract the loss of lives and property.

1.5 Literature Review

All engineering designs are based on past knowledge and experience. Either we perform a conservative design or introduce entirely new concepts, it becomes necessary to support or justify our actions with reference to pre-existing knowledge. This project, having a conventional design approach, is invariably based on certain established practices. The literatures relevant to this project are discussed below.

1. Nepal National Building Code (NBC 000-1994)

This code aims to standardize building construction practices in a way that is practicable in the Nepalese context. But its development is relatively recent and it still lacks many documents (codes) required to support it. To compensate for this unavailability, the code frequently refers to Indian Standard codes. This code has classified the sophistication in design and construction into the following four types:

- International state-of-art
- Professionally engineered structures
- Buildings of restricted size designed to simple rules-of-thumb
- Remote rural buildings where control is impractical

This project belongs to type-III according to this classification. Furthermore, this code allows the use of international codes provided their use also meets the NBC requirements. Hence, using IS codes for this project is justified.

2. Indian Standard (IS) codes

The following IS codes shall be referred to in particular:

i. IS 456: 2000 Plain and Reinforced Concrete – Code of Practice.

This code forms the main basis for the design of RCC structures in this region. It includes the design of structural elements such as beams, columns, slabs, staircase and footings. It also mentions the design criteria for limit states of flexure, shear, compression and torsion and for limit states of serviceability (i.e., deflection and cracking).

ii. IS 875: 1987 (Reaffirmed 1997) Code of Practice for Design Loads (Other than Earthquake) for Building and Structures

Part 1: Dead Loads – Unit Weights of Building Materials and Stored Materials

Part 2: Imposed Loads

Part 5: Special Loads and Combinations

The codes provide information for estimation of design loads on the structures. The part-1 provides the unit weights of several construction materials as well as the materials that are commonly stored in building. The part-2 presents data for live loads that may be assumed for various building types; it does not deal with the loads occurring in the construction phase and those caused due to special vibrations. Part-5 deals with load effects due to temperature, earth pressure, hydrostatic pressures etc. It also prescribes various load combinations. Parts 3 and 4 deal with wind and snow loads and are not relevant to the present design.

iii. IS 1893(Part 1): 2016 Criteria for Earthquake Resistant Design of Structures (Part 1: General Provisions and Buildings)

This code deals with the assessment of seismic loads on various structures and design of earthquake resistant design of buildings. It deals with the mechanics of seismic engineering insofar as it is concerned with the methods of determining seismic loads and the effects various irregularities in a building can have upon its seismic response.

iv. IS 13920: 2016 (Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces)

This standard provides the requirement for designing and detailing of members of reinforced concrete (RC) structures designed to resist lateral effects of earthquake shaking, so as to give them adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse. These standards address lateral load resisting structural systems of RC structures composed of,

a. RC moment resisting frames,
b. RC moment resisting frames with unreinforced masonry infill walls,
c. RC moment resisting frames with RC structural walls, and
d. RC structural walls.

3. Bureau of Indian Standards Special Publications (SP)

SP 16: Design Aids for Reinforced Concrete to IS 456-1978

This handbook explains the use of formulae mentioned in IS 456 and provides several design charts (for rectangular cross-sections) which can greatly expedite the design process if done manually. This shall be particularly useful in the preliminary design.

4. Textbooks on RCC Design, Earthquake Engineering and other books

i. Reinforced Concrete Limit State Design (Jain A.K.)

This textbook has been written in the Indian context and they present the practice as laid down by IS 456, SP 16 and SP 24 in a pedagogical manner.

ii. Design of reinforced concrete structures (Subramanian, Narayanan)
iii. Earthquake-resistant design of structures (Duggal, S. K)

Structural dynamics forms the very basis of earthquake engineering. These books present the subject in detail, with adequate explanations and examples, which will be essential in understanding the seismic codes.

5. Old Reports on the same subject

The report prepared by different past year student group helps to find the procedure that is involved for the design and analysis of structural components.

6. Structure Design of R.C.C. Building Component

It helps to study the stepwise process for the analysis and the design of R.C.C. buildings.

2. METHODOLOGY

In Nepal, design of buildings is mainly based upon the guidelines provided by the Nepal National Building Codes: 000- 1994. NBC codes are designed referring to IS codes and are less detailed and extensive comparing to IS code. So, it permits use of IS code for design such that building fulfills requirement of NBC codes on doing so. In this project, we are going to use IS codes which follows limit state design method.

Limit state design has been originated from ultimate or plastic design. The object of design based on the limit state concept is to achieve an acceptable probability that a structure will not become unserviceable in its lifetime for the use of which it is intended, i.e., it will not reach the limit state. A structure with appropriate degrees of reliability should be able to withstand safely all the loads that are liable to act on it through-out its life and it should satisfy the serviceability requirements. The three different design formats used in the limit states are; Multiple Safety Factor Format, Load and Resistance Factor Design Format and the Partial Safety Factor Format. All the relevant limit states must be considered in design to ensure an adequate degree of safety and serviceability.

The analysis and design of the building will be carried out following these steps:

Firstly, the design and analysis require knowledge of RCC which is studied at 7th semester along with knowledge and understanding from all previous semester. The "Earthquake resistant design of structure" is studied on 8th semester which is needed for analysis of seismic performance of building. Secondly, before commencing the project, essential software like **ETABS, AutoCAD, SAFE**, etc. are learned. Thirdly, after choosing the building for analysis, the proposal is created and submitted to the supervisor. Architectural drawings of the selected building are obtained and studied in-depth. Fourthly, after the proposal is approved, the detailed design process begins with the following steps:

- Analyzing the provided architectural drawing, modifying it as per the Supervisor's guidance to make an economic seismic resistant design.

- Estimation and idealization of loads, such as dead load and live load, are done based on Indian Standard Code of Practice **IS:875-1987** for Design Loads for Buildings and Structures.

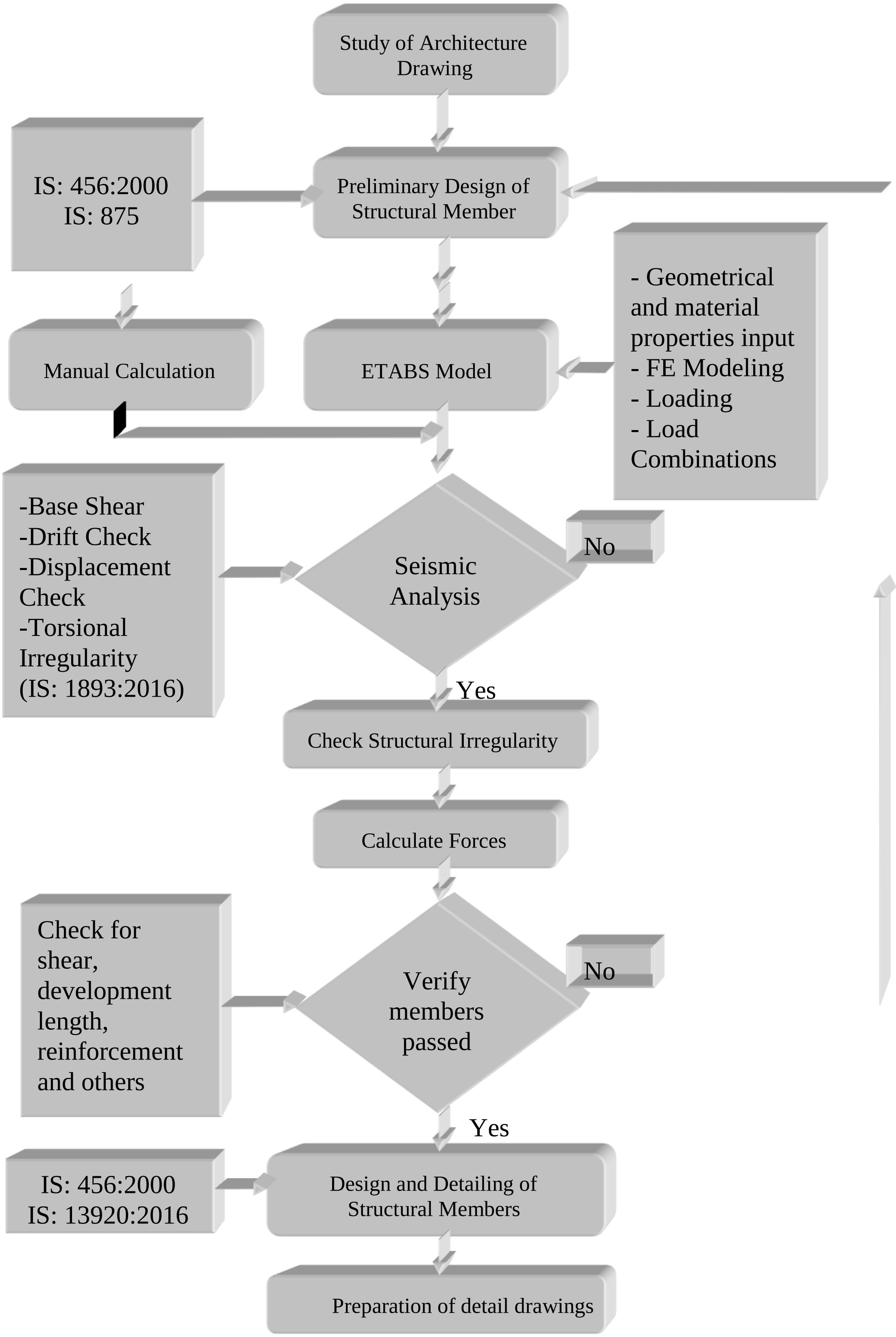
- Estimation of seismic load is based on **IS: 1893-2016** for Earthquake Resistant Design of Structures, Part 1 while IS:13920**-2016** is referred to for reinforcement detailing.

- Preliminary design is done to determine the approximate shape and size of structural members based on the deflection control criteria provided in codes. The ETABS software is used for modeling and analyzing the structure based on the Finite Element Method.

- All calculations for the design are based on **IS: 456-2000** and **IS: 13920-2016**, along with the textbooks [**Reinforced Concrete Limit State Design (Jain A.K., Design of reinforced concrete structures (Subramanian, Narayanan), Earthquake-resistant design of structures (Duggal, S. K)]** and design aids (**Indian Standard Special Publications SP 16, SP 22, SP 24 and SP 34)**

- The final outcome of the analysis and design process is the structural drawing, including detailed ductile detailing of the reinforcement bars.

- Required modifications are made to the provided architectural drawing, such as the size of structural members and partition walls, and all drawings are printed in an appropriate format for inclusion in the final report.


Study of Architecture Drawing
IS: 456:2000
IS: 875
Preliminary Design of Structural Member
Manual Calculation
ETABS Model
- Geometrical and material properties input
- FE Modeling
- Loading
- Load Combinations
-Base Shear
-Drift Check
-Displacement Check
-Torsional Irregularity
(IS: 1893:2016)
Seismic Analysis
No
Yes
Check Structural Irregularity
Calculate Forces
Check for shear, development length, reinforcement and others
Verify members passed
No
Yes
IS: 456:2000
IS: 13920:2016
Design and Detailing of Structural Members
Preparation of detail drawings

2.1 Planning Phase

Planning of building is grouping and arrangement of different component of a building so as to form a homogenous body which can meet all its function and purposes. Proper orientation, safety, healthy, beautiful and economic construction are the main target of building planning. It is done based on the following criteria:

2.1.1 **Functional Planning**

- Client requirement is the main governing factor for the allocation of space required which is based upon its purposes. Thus, demand, economic status and taste of owner features the plan of building.
- Building design should favor with the surrounding structures and weather.
- Building is designed remaining within the periphery of building codes, municipal bylaws and guidelines.

It is carried out in two steps in detail as below.

2.1.1.1 **Planning of Space and Facilities**

The layout of the building plan was prepared and finalized as per client requirements.

For vertical mobility, doglegged staircase is provided.

All other functional amenities are only used for load assessment and ignoring their aesthetic and functional planning which is beyond the scope of this project.

2.1.1.2 **Architectural planning of 3D framework of Building**

The building to be designed is a multi-storey RCC office building. For reinforced concrete frames, a grid layout of beams is made considering the above functional variables. In most of grid intersection points, columns are placed.

This framework for each floor is then utilized with positioning of masonry wall between the columns. Separation of individual commercial spaces is done with masonry wall.

A total of 36 numbers of columns are provided. The overall dimension of the building is 100’ 2.75”*100’2.75” without any provision of expansion joint, the justification for which is presented in detail in following subheadings.

Arrangement of beams is done along the grid interconnecting the columns at grid intersections. With this framework of beam and column having RCC slab in the floor and roof, architectural planning of the building is complete and 3D framework is thus complete.

2.1.1.3 Compliance to Municipal By-Laws

All the functional planning of building is done conforming to Municipal By-Laws of Kathmandu Metropolitan City for Urbanized and urbanizing localities. Specific points in the by-laws that need special focus of designer are:

- Type of Building
- Land Area Available
- Floor Area Ratio (FAR)
- Maximum Ground Coverage (GCR)
- Maximum height of the building, etc.

These variables are also dictated by specific location of site in different wards. Building height is a restricted by the position of widest road along the site and the light plane of 63.5^0 between the top of the building and the centerline of the road.

This completes the overall functional planning of the building with coverage of maximum number of variables in preliminary stage planning.

2.1.2 Structural Planning

The four main desirable attributes of an earthquake resistant building are:

a. Robust structural configuration
b. At least a minimum elastic lateral stiffness,
c. At least a minimum lateral strength,
d. Adequate ductility

Buildings with simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage than buildings with irregular configurations. All efforts shall be made to eliminate irregularities by modifying architectural planning and structural configurations.

The building system is functionally and legally planned appropriately following the codal provisions. Our focus in the current section is the structural orientation of the building in horizontal and vertical plane avoiding irregularities mentioned in IS 1893 (part 1):2016.

The following types of irregularities mentioned in Table 4 & 5 of IS 1893 (part 1):2016 should be avoided as far as practicable during functional planning.

2.1.2.1 Plan Irregularities

1. Torsion Irregularity

Usually, a well-proportioned building does not twist about its vertical axis, when

a. The stiffness distribution of the vertical elements resisting lateral loads is balances in plan according to the distribution of mass in plan (at each storey level); and
b. The floor slabs are stiff in their own plane.

A building is said to be torsionally irregular if:

Maximum horizontal displacement of any floor in the direction of lateral force at one end of floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction.

2. Re-entrant corners

A building is said to have reentrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15% of its overall plan dimension in that direction.

3.Diaphragm Discontinuity

A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area more than 50 percent of the full area of the floor slab.

4.Out of plane Offsets

A building is said to have out plane offset in vertical elements, when structural walls or frames are moved out of plane in any storey along the height of the building.

5.Non-parallel Lateral force Systems: A building is said to have non-parallel system when the vertically oriented structural systems resisting lateral forces are not oriented along the two principal orthogonal axes in plan.

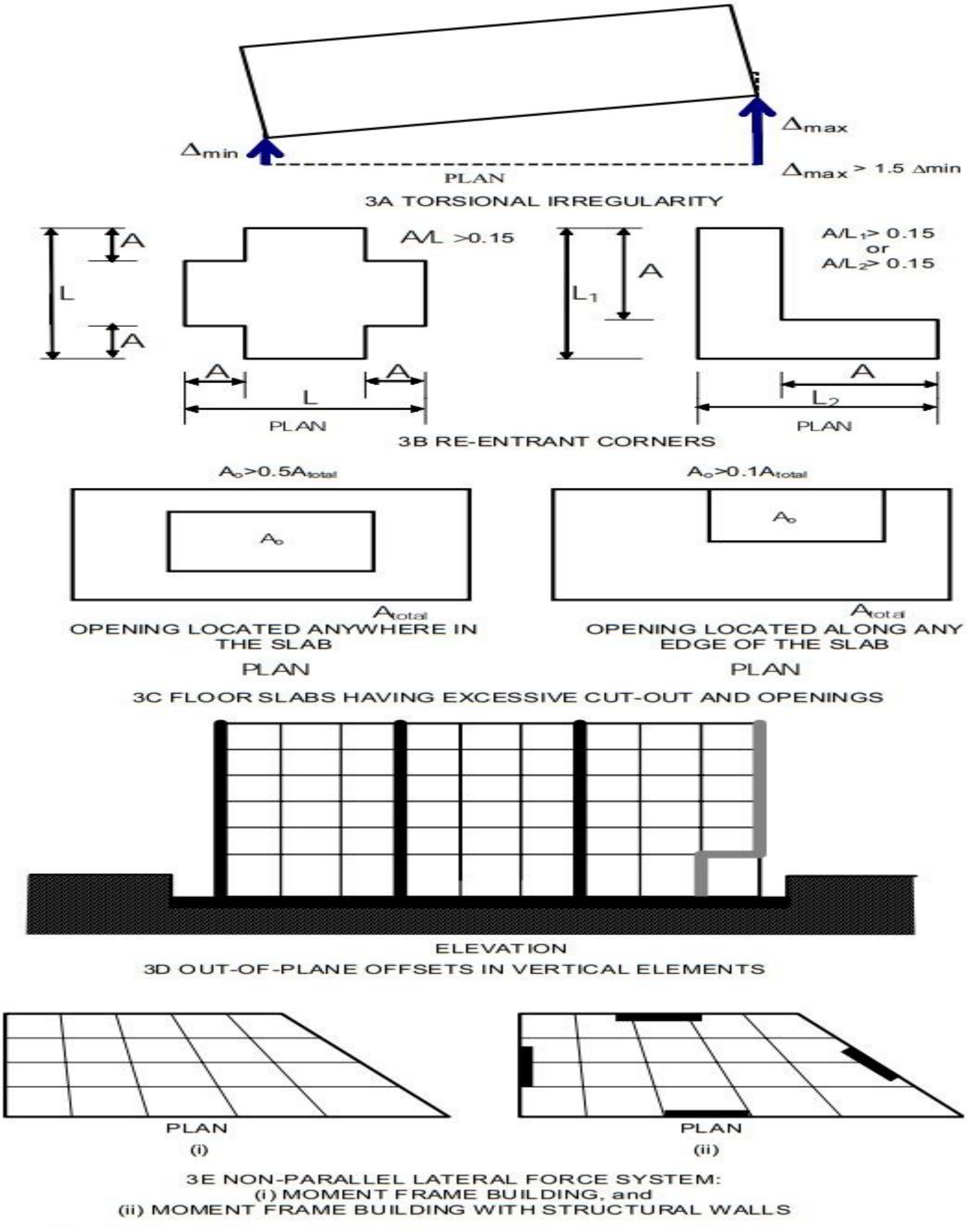


Figure 1: Definition of Irregular Buildings- Plan Irregularities

2.1.3 Vertical Irregularities

1. Stiffness Irregularity –Soft Storey

A building is said to have soft storey if;

a. Ratio of stiffness of lower storey to upper storey is less than 0.7.
b. Ratio of lower storey to average of upper three storey is less than 0.8.

2. Mass irregularity

Mass irregularity shall be considered to exist, when the seismic weight of any floor is more than 150 percent of that of the floors below.

3. Vertical Geometric Irregularity

Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 125 percent of the storey below.

4. In-plane discontinuity

In plane discontinuity in vertical elements which are resisting lateral force shall be considered to exist, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan of those elements.

5. Strength Irregularity- Weak Storey

A weak storey is a storey whose lateral strength is less than that of the storey above.

6. Irregular modes of oscillation in two principal plan directions

A building is said to have lateral storey irregularity in a principal plan direction, if

a. The first three modes contribute less than 65 percent mass participation factor in each principal plan direction.
b. The fundamental lateral natural periods of the building in two principal plan directions are closer to each other by 10 percent of the larger value.

The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended service life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability.

Structural planning of the building is done over the proposed architectural plan for providing and preserving the structural integrity of the entire building. This is dealt in detail for each structural element with necessary justification.

Finalized structural plan is then employed for load assessment and preliminary design of structural members for modeling in ETABS.

2.2 Load Assessment

Once the detailed architectural drawing of building is drawn, the buildings subjected to different load are found out and the calculations of load are done. The loads on building are categorized as below

a. Gravity load

This includes the self-weight of the building such as structural weight, floor finish, partition wall, other household appliances, etc. To assess these loads, the materials to be used are chosen and their weights are determined based on Indian standard code of practice for design loads (other than earthquake) for buildings and structures:

i. IS 875 (part I):1987 Dead Loads
ii. IS 875 (part II):1987 Imposed Loads

b. Lateral load:

Lateral load includes wind load and earthquake load. Wind load acts on roof truss while an earthquake act over the entire structure. Wind load calculation is based on IS 875 (part III):1987 and earthquake on IS 1893 (part I):2016.

The earthquake is an unpredictable natural phenomenon. Nobody knows the exact timing and magnitude of such loads. Seismic loads are to be determined essentially to produce an earthquake resistant design. Since the probable maximum earthquake occurrence is not frequent, designing building for such earthquake isn't practical as well as economically prudent. Instead, reliance is placed on kinetic dissipation in the structure through plastic deformation of elements and joints and the design forces are reduced accordingly. Thus, the philosophy of seismic design is to obtain a no-collapse structure rather than no-damage structure.

2.3 Estimation of Load

It is most important step in structural design. Proper recording of them required for confusion free analysis.

2.3.1 Dead loads:

i. Calculate the weight of those elements of building whose dimensions are fixed already from functional considerations and can be worked out carefully. These are generally non-structural elements and of parapets, rooftop, railings etc.

ii. From pre design, calculate weight of structural elements such as beam, column, slab etc.

iii. Put all loads systematically on sketches, say plan wise, showing their gravity lines with reference to column center-lines.

2.3.2 Live loads:

Select live load intensity occupancy-wise as applicable for slabs and beams from the code and write this on plan. The reduction of live load intensities for the number of storey in the columns and that for calculating earthquake loads may be considered in the calculations later.

2.3.3 Seismic or earthquake loads:

Earthquake or seismic load on a building depends upon its geographical location, lateral stiffness and mass, and is reversible.

IS: 1893- 2016 was followed for the calculation of the earthquake loads, which specifies two methods viz.

a) Equivalent Static method, and

b) Dynamic Analysis method

Dynamic analysis method can be performed in three ways:

1. Response Spectrum method
2. Modal time history method, and
3. Time history method

Equivalent Static Method:

The equivalent static method the response of the structure in the case of dynamic loading are replaced by the static load at various levels to approximately represent the magnitude and direction of the dynamic loading as in the case of earthquakes. The concentrated lateral force due to dynamic loading occur at floor or ceiling level where the concentration of mass is highest.

The equivalent static method or seismic coefficient method is generally applicable to general building up to 15 m in height in seismic zone II.

Equivalent static method may be used for analysis of regular structure with approximate natural period T_a less than 0.4s.

As per this method, first the design base shear VB is computed for the building as a whole. Then the VB is distributed at various floor levels at the corresponding center of mass after which the design seismic force is distributed to individual lateral load resisting elements through the structural analysis considering the floor diaphragm action.

The design base Shear VB along any principal direction of the building is determined by:

Base Shear (V_b) = A_h * W

Where,

A_h = Design horizontal acceleration spectrum

W = Seismic weight of building

$$Ah = \frac{\left(\frac{Z}{2}\right) x \left(\frac{Sa}{g}\right)}{\frac{R}{I}}$$

Where, Z= Zone factor, From Table 3 clause 6.4.2

I = Importance factor, Table 8 clause 7.2.3

R= Response reduction factor

S_a/g= Structural response factor

The fundamental time period of the vibration,

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$$

$$A_w = \sum_{i=1}^{N_w} [A_{wt}\{0.2 + (\frac{L_{wi}}{h})^2\}]$$

h = height of building as defined in CL 7.6.2(a), in m;

A_{wi} = effective cross-sectional area of wall in first storey of building, in m^2;

L_{wi}= length of structural wall i in first storey in the considered direction of lateral forces, in m;

d = base dimension of the building at the plinth level along the considered direction of earthquake shaking, in m; and

N_w = number of walls in the considered direction of earthquake shaking.

The design base shear V_B is distributed along the height of the building as:

$$Q_i = \left[\frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}\right] V_B$$

Where, Q_i = Design lateral force at floor i;

W_i = Seismic weight of the floor i;

h_i = height of the floor i measured from the base;

n = number of stories in the building, that is number of levels at which masses are located.

Response spectra

The representation of the maximum response of idealized single degree of freedom system having certain period of vibration and damping during given earthquake is referred to as response spectrum. The maximum response, that is, maximum absolute acceleration, maximum relative velocity or maximum relative displacement of single degree of freedom system is plotted against the damped natural period and for various damping values.

The seismic analysis can be performed using design spectrum given in below figure; which is based on strong motion records of eight earthquakes in India.

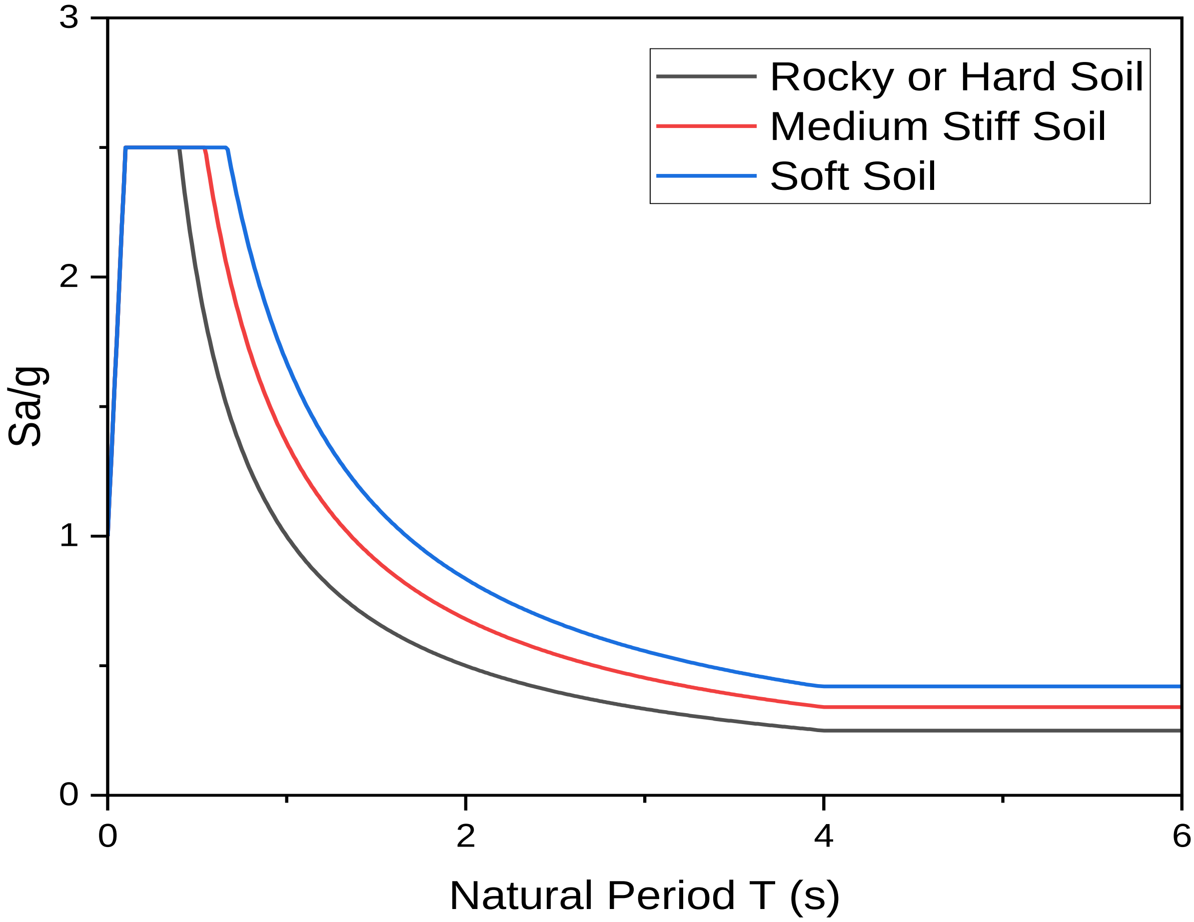


Figure 2:Response spectra for rock and soils for 5% damping

An elastic response spectrum has been proposed for Maximum Considered Earthquake (MCE), which is divided by factor 2 to get Design Basis Earthquake (DBE) and again by factor R to get inelastic response spectra. MCE is a very rare event which has a 2% probability of being exceeded in 50 years. DBE is that earthquake which has reasonably been expected to occur at least once during design life of structure and has a 10% probability of being exceeded in 50 years. The intention is to let the designer know about the whole scenario of elastic and inelastic response spectra and also to know the need of providing ductility in structure.

The seismic analysis can be performed using design spectrum. Response spectrum method is dynamic analysis used for the analysis of seismic loads for unsymmetrical buildings.

Base Shear (V_b) = A_h * W

Where,

A_h = Design horizontal acceleration spectrum

W = Seismic weight of building

$$Ah = \frac{\left(\frac{Z}{2}\right) x \left(\frac{Sa}{g}\right)}{\frac{R}{I}}$$

Where, Z= Zone factor, From Table 3 clause 6.4.2

I = Importance factor, Table 8 clause 7.2.3

R= Response reduction factor

S_a/g= Structural response factor

The fundamental time period of the vibration,

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$$

$$A_w = \sum_{i=1}^{N_w} [A_{wt}\{0.2 + (\frac{L_{wi}}{h})^2\}]$$

h = height of building as defined in CL 7.6.2(a), in m;

A_{wi} = effective cross-sectional area of wall in first storey of building, in m^2;

L_{wi}= length of structural wall i in first storey in the considered direction of lateral forces, in m;

d = base dimension of the building at the plinth level along the considered direction of earthquake shaking, in m; and

N_w = number of walls in the considered direction of earthquake shaking.

Dynamic analysis shall be performed in accordance to clause 7.7, IS: 1893-2016.

2.4 Load Combinations

Combinations of different loads are based on IS 875 (part V):1987 Load combinations.

1) 1.5 DL
2) 1.5(DL + LL)
3) 1.2 (DL + LL ± EQx)
4) 1.2 (DL + LL ± EQy)
5) 1.5 (DL ± EQx)
6) 1.5 (DL ± EQy)
7) 0.9 DL±1.5 EQx
8) 0.9 DL ± 1.5 EQy

For considering eccentricity in building, additional load combinations are considered in which **EQ** is replaced with **EQ±e.**

2.5 Preliminary Design

Before proceeding for load calculation, Preliminary size of slabs, beams and columns and the type of material used are decided. Preliminary Design of structural member is based on the IS Code provisions for slab, beam, column, wall, staircase and footing of serviceability criteria for deflection control and failure criteria in critical stresses arising in the sections at ultimate limit state i.e., Axial loads in the columns, Flexural loads in slab and beams, etc. Appropriate sizing is done with consideration to the fact that the preliminary design based on gravity loads is required to resist the lateral loads acting on the structure. The following remarks will be helpful in choosing the sections:

a. Too many variations in the sizes of beam and columns, width and depth are not desirable from both aesthetic and economical point of view. Minimum dimensions of 200mm for small spans and 230mm~ 300mm for large spans may be set for structural members.
b. Richer concrete mixes can be used in lower storey elements to avoid frequent change in sections. Some size variation can also be avoided by reducing column steel upwards in building.
c. Frequently column steel may be at odds with the longitudinal steel of beams crossing it from one or more directions. Also cover required differs. It may be useful to keep column wider than the beam and the number of bars be kept even in column and odd in beam or vice-versa so that bars pass uninterruptedly.

d. Narrow-deep beams may show shrinkage, temperature cracking in web and also lateral buckling if laterally unsupported. This should be considered in surface reinforcement detailing and ensuring lateral support on the compression face at less than 25*b, b being beam breadth, where the effective depth of beam exceeds 3 times of b.
e. At the preliminary design stage, calculation of reinforcement may be excessive, but it will be good to know the maximum steel required to check that it lies within a reasonable percentage of the concrete section and can be located in it without congestion.

Normally preliminary size will be decided considering following points:

- Slab: The thickness of the slab is decided on the basis of span/d ratio assuming appropriate modification factor.
- Beam: The depth is generally taken as 1/12-1/15 of the span. The width of beam is taken ½ to 2/3rd of the depth of the beam.
- Column: Size of column depends upon the moments from the both direction and the axial load. Preliminary Column size may be finalized by approximately calculation of axial load and moments.

The sections worked out as a result of preliminary design should be now recorded, discussed with the architect and finalized before undertaking the further final analysis which is more time consuming as well as more expensive.

The approximate dimensions of structural elements were determined in preliminary design so that they act as guidelines in analysis and aid to make final design safe and economical.

2.6 Idealization of structure

2.6.1 Idealization of support

It deals with the fixity of the structure at the foundation level. In more detail terms, this idealization is adopted to assess the stiffness of soil bearing strata supporting the foundation. Although the stiffness of soil is finite in reality and elastic foundation design principles address this property to some extent, our adoption of rigid foundation overlooks it. Elastic property of soil is addressed by parameters like Modulus of Elasticity, Modulus of Subgrade reaction, etc.

Idealization of support is done in the light of assessing the fixity of structure at the foundation level. Columns are assumed to be fully fixed at the raft surface with raft underneath supporting the load of the superstructure. Plinth beams are provided at a certain height from the existing ground surface as a means of tying the columns and also serve as Damp Proof Course (DPC).

2.6.2 Idealization of load

The load acting on the clear span of a beam should include floor or any types of loads acting over the beam on the tributary areas bounded by 45^0 lines from the corner of the panel i.e., Yield line theory is followed. Thus, triangular or trapezoidal types of load act on the beam.

2.6.3 Idealization of joint constraints

Joints are defined with constraints to serve as rigid floor diaphragm at individual floor level. Due to this definition of joints, the slabs subjected to lateral loads behave in in plane action of thin shell and hence act as rigid floor diaphragm.

2.6.4 Idealization of Structural system

Initially individual structural elements like beam, column, slab, staircase, footing, etc. are idealized. Once the individual members are idealized, the whole structural system is idealized to behave as theoretical approximation for first order linear analysis and corresponding design. Main beam elements though cast integrally with the slab are idealized to serve as rectangular beams. This is done considering the fact that during the reversal of load in seismic loading, concrete is subjected to tension on both the top and bottom faces. This tensile stress induced at the flange renders the concrete unfit for taking load.

However, the secondary beams are idealized to behave as hinged beams with partial fixity at the supports. The effect of earthquake load is not seen in secondary beam and only gravity load dictates design. For this reason, secondary beam is idealized as flanged beam.

Various general assumptions have been made in analysis and design of the structures, for consideration of simplicity and economy, viz.:

1. Tensile strength of concrete is ignored.
2. Shrinkage and temperature strength are negligible.
3. Adhesion between concrete and steel is adequate to develop full strength.
4. Seismic and wind load do not occur simultaneously.
5. Centerlines of beams, columns and shear walls are concurrent everywhere.

The building is idealized as unbraced space frame. This 3D space frame work is modeled in ETABS for analysis. Loads are modeled into the structure in several load cases and load combination.

2.6.5 Idealization of Slabs

Idealization of slab element is done in earthquake resistant design to perform as a rigid floor diaphragm. This idealization is done for the slab to behave as a thin shell element subjected to out-of-plane bending only under the action of gravity loads. Due to infinite in-plane stiffness of the shell element, lateral loads are not taken by the floor slab and hence resisted completely by the columns. Hence, such an idealized slab is then modeled in ETABS2018 program for analysis.

2.6.6 Idealization of Staircase

Open welled staircase used in the building is idealized to behave as simply supported slabs, supported on beams at the floor and landing levels. This idealization helps us analyze the staircase slab in strips subjected to distributed loading on the landing strip and going of the slab. Detailing rules are then followed to address the negative bending moment that are induced on the joint of going and top flight in the staircase, the rigorous analysis of which is beyond or scope. Staircase being an area element is also assumed not to be a part of the integral load bearing frame structure. The loads from staircase are transferred to the supports as vertical reactions and moments.

2.6.7 Idealization of Beam and Column

Beam column idealization is one of the most critical aspects of structural idealization to achieve the desired behavior of the overall integrated structure. Beams and columns are idealized to behave as linear elements in 3D. Beam column joints in the structural planning are assumed to behave as perfectly rigid joints. In reality, perfectly rigid joints do not exist. Effects of partial fixity can be addressed in modeling by rigorous analysis of sectional and material properties, which is beyond the limits of this project. Assumptions of rigid joints are also found to perform well in nature, seen from years of practice. Main beam and secondary beam joints are idealized as hinged joints owing to the detailing adopted in such joints. Hinge beam assumption can have two impacts on structural behavior of secondary beams. Firstly, lateral loads aren’t transferred to the secondary beams from main beams and hence they can be idealized as flanged sections.

Secondly, hinge connection at their extremities lets us address the partial fixity of the beams in taking moments due to gravity loads.

Another idealization is addressing the section of main beam as rectangular in shape despite being integrally connected with the slabs. The flange portions of these beams when subjected to reversal of loading during earthquakes become ineffective in taking the tension induced in them and hence, we ignore their contribution in design.

2.7 Modeling and Analysis of structure

2.7.1 Salient Features of ETABS

ETABS represents one of the most sophisticated and user-friendly release of SAP series of computer programs. Creation and modification of the model, execution of the analysis, and checking and optimization of the design are all done through this single interface. Graphical displays of the results, including real-time display of time-history displacements are easily produced.

The finite element library consists of different elements out of which the three-dimensional FRAME element was used in this analysis. The Frame element uses a general, three-dimensional, beam-column formulation which includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations.

Structures that can be modeled with this element include:

- Three-dimensional frames
- Three-dimensional trusses
- Planar frames
- Planar grillages
- Planar trusses

A Frame element is modeled as a straight line connecting two joints. Each element has its own local coordinate system for defining section properties and loads, and for interpreting output. Each Frame element may be loaded by self-weight, multiple concentrated loads, and multiple distributed loads. End offsets are available to account for the finite size of beam and column intersections. End releases are also available to model different fixity conditions at the ends of the element. Element internal forces are produced at the ends of each element and at a user specified number of equally-spaced output stations along the length of the element. Loading options allow for gravity, thermal and pre-stress conditions in addition to the usual nodal loading with specified forces and or displacements. Dynamic loading can be in the form of a base acceleration response spectrum, or varying loads and base accelerations.

2.7.2 Input and Output

The design of earthquake resistant structure should aim at providing appropriate dynamic and structural characteristics so that acceptable response levels result under the designed earthquake. The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads, deformations of normal construction, use; have adequate durability and adequate resistance to the effects of misuse and fire.

The building is modeled as a 3D bare frame. Results from analysis are used in design of beams and columns only (i.e., linear elements). ETABS doesn't design shell elements. Joints are defined with constraints to serve as rigid floor diaphragm and hence slabs are designed manually as effect of seismic load is not seen on slab. The linear elements are also designed primarily by hand calculation to familiarize with hand computation and exude confidence where we are unable to trust fully on design results of ETABS. This has been done as we are quite unfamiliar with fundamentals of FEM analysis techniques based on which the software package performs analysis and gives results.

As we are working with a computer-based system, the importance of data input is as important as the result of output derived from analysis. Hence with possibility of garbage-in-garbage-out, we need to check our input parameters in explicit detail.

Material properties are defined for elements in terms of their characteristic strength i.e., M25 for slabs, beams and columns. Also, section properties are defined as obtained from preliminary design. Loading values are input as obtained from IS 875. Loading combination based on IS 875 (part V):1987 and IS 1893 (part 1):2016 for ultimate limit state and IS 456:2000 for serviceability limit state is prepared. An envelope load case of all load combinations is prepared to provide us with the envelope of stresses for design.

The design moments, shear forces, axial forces and torsions are taken as computed by computer software program "ETABS" for the worst possible combinations and number of hand calculations are done as to verify the reliability of the design results suggested by the software.

2.7.3 Structural Analysis Procedure:

1. **Base Shear Calculation**

- First, all the loads acting on the floor are determined.
- The lumped seismic weights of the floor are determined with load reduction factors accompanied with codal provisions.

  Seismic Weight = Total dead load + (25% or 50%) of live

  Total dead load = Floor wt. + Beam wt. + Half column wt. (from lower and upper part of slab) +Half wall weight (from lower and upper part of slab)
- The total seismic weight multiplied by seismic base coefficient gives the seismic base shear of the structure.

  Base shear (V_b) = W_T *A_h from cl. 7.6.1. pg. 21

The shear is then distributed among the floor respective to their load. From cl. 7.6.3.

$$Q_i = \left[\frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}\right] V_B$$

Where, Q_i = Design lateral force at floor i;

W_i = Seismic weight of the floor i;

h_i = height of the floor i measured from the base;

- n = number of stories in the building, that is number of levels at which masses are located.
- The shear forces calculated manually and with ETABS is then compared which is processed further if the error lies within the percentage of 5.

2. **Check for Eccentricity (COM/COR)**

Eccentricity in Center of mass (COM) and center of rigidity (COR) causes torsion effect in the building. As Earthquake load acts through the COM, the force creates torsion force rotating the building about COR axis. Thus, eccentricity of the building must be brought to about 5% to reduce such effect and building vibrates in the direction of earthquake.

The COM and COR coordinates of the structure along both principal axes are derived with the EABS analysis. The eccentricity between them is then checked to 5%.

3. **Check for Torsional Irregularity**

A building is said to be torsionally irregular if:

Maximum horizontal displacement of any floor in the direction of lateral force at one end of floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction.

The maximum and minimum displacement in each floor is given from ETABS with the joint displacements data and the ratio is then checked.

In torsionally irregular buildings, when the ratio of maximum horizontal displacement at one end and the maximum horizontal displacement at the other end is

- In the range of 1.5-2.0 a) the building configuration should be revised to ensure that the natural period of the fundamental torsional mode of oscillation shall be smaller than those of the first two translational modes along each of the principal plan directions, and then

  b) Three-dimensional dynamic analysis method shall be adopted;
- More than 2.0, the building configuration shall be revised.

4. Check for Stiffness Irregularity

A soft storey is that storey whose lateral stiffness is less than that of the storey above.

A building is said to have soft storey if;

i. Ratio of stiffness of lower storey to upper storey is less than 0.7.
ii. Ratio of lower storey to average of upper three storeys is less than 0.8.

The stiffness calculated with ETABS is tabulated and then checked. A graph is plotted which must show that stiffness of bottom storey to be highest and decreasing gradually but not increasing to particular extent.

5. Check for Mass Irregularity

Mass irregularity shall be considered to exist, when the seismic weight of any floor is more than 150 percent of that of the floors below.

In buildings with mass irregularity and located in Seismic zones III, IV, and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per 7.7).

6. Drift Analysis

The relative inter-storey horizontal displacement is referred to as storey drift.

A limitation on storey drift is necessary to avoid discomfort to occupants of the building and to save non-structural elements from damage. A drift of 0.004 times or 0.4% the storey height in the elastic range is imposed by IS 1893:2016 clauses 7.11.1.1. Storey Drift Limitation.

The maximum displacement of each floor is derived from the ETABS and relative displacement (drift) is then worked out from the data which must be in accordance to the code. The displacement and drift off the building is plotted with Storey and represented.

7. Dynamic Analysis (Response Spectrum)

Response spectrum method is dynamic analysis used for the analysis of seismic loads for unsymmetrical buildings. Dynamic Analysis is performed to obtain forces and its distribution to different levels/ elements of building on following cases:

Table 1: Dynamic analysis cases

Regularity Cases	Height (m)	Zones
Regular Buildings	>40	IV, V
	>90	II, III
Irregular Buildings	>12	IV, V
	>40	II, III

Response spectrum method of analysis shall be performed using the design spectrum as defined in clause 6.4.2 or by a site-specific design spectrum mentioned in clause 6.4.7.

The design base shear V_B shall be compared with a base shear $\bar{V}_{\bar{B}}$ calculated using a fundamental time period Ta. Where $V_{\mathbf{B}}$ is less than $\bar{V}_{\bar{B}}$, all response quantities shall be multiplied by $\bar{V}_{\bar{B}}/V_B$.

- Undamped free vibration analysis of the entire building shall be performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system to obtain natural periods (T) and mode shapes (Φ) of those of its mode of vibration considered as per clause 7.7.5.2
- The number of modes to be used in the analysis should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent.
- If modes with natural frequency beyond 33Hz are to be considered, modal combination shall be carried out only for modes up to 33 Hz. The effect of higher modes shall be included by considering missing mass correction following well established procedures.

- The first three modes altogether must contribute at least 65% mass participation factor in each principal direction.
- Regular buildings shall be analyzed as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case the following shall hold in the computation of various quantities:
  - i. Modal Mass
  - ii. Mode participation factor
  - iii. Design Lateral force
  - iv. Storey shears forces in each mode
  - v. Lateral forces at each storey due to all modes considered

2.8 Design Philosophy

There are three philosophies for the design of reinforced concrete viz.

1. Working Stress Method
2. Ultimate Load Method
3. Limit State Method

2.8.1 Limit State Method of Design for Reinforced Concrete Structures

Design of Reinforced Concrete Members is done based on the limit state method of design following IS 456:2000 as the code of practice. The basic philosophy of design is that the structure is designed for strength at the ultimate limit state of collapse and for performance at limit state of serviceability. A check for these two limit states is done based on code of practice to achieve safe, economic and efficient design.

Limit state design has been originated from ultimate or plastic design. The object of design based on the limit state concept is to achieve an acceptable probability that a structure will not become unserviceable in its life time for the use for which it is intended, i.e. it will not reach a limit state. A structure with appropriate degrees of reliability should be able to withstand safely all the loads that are liable to act on it through-out its life and it should satisfy the serviceability requirements. All relevant limit states must be considered in design to ensure an adequate degree of safety and serviceability.

2.8.1.1 Limit state of collapse

This state corresponds to the maximum load carrying capacity. Violation of collapse limit state implies failure in sense that a clearly defined limit state of structural usefulness has been exceeded. However, it does not mean a complete collapse. This limit state may correspond to:

1. Flexure,
2. Compression,
3. Shear and
4. Torsion.

Assumptions for the limit state of collapse in flexure:

a. The plane section normal to the axis of member remains plane after bending.
b. The maximum strain in concrete at the outermost compression fiber is 0.0035.
c. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape. For design purpose, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor y_m =1.5 shall be applied.
d. The tensile strength of concrete is ignored.
e. The stresses in the reinforcement are derived from the representative stress-strain curve for the type of steel used. For design purpose the partial safety factor y_m =1.15 shall be applied.
f. The maximum strain in the tension reinforcement in the section at failure shall not be less than:

$$\frac{fy}{1.15 * E_s} + 0.002$$

Where, f_y = characteristics strength of steel

E_s = Modulus of elasticity of steel.

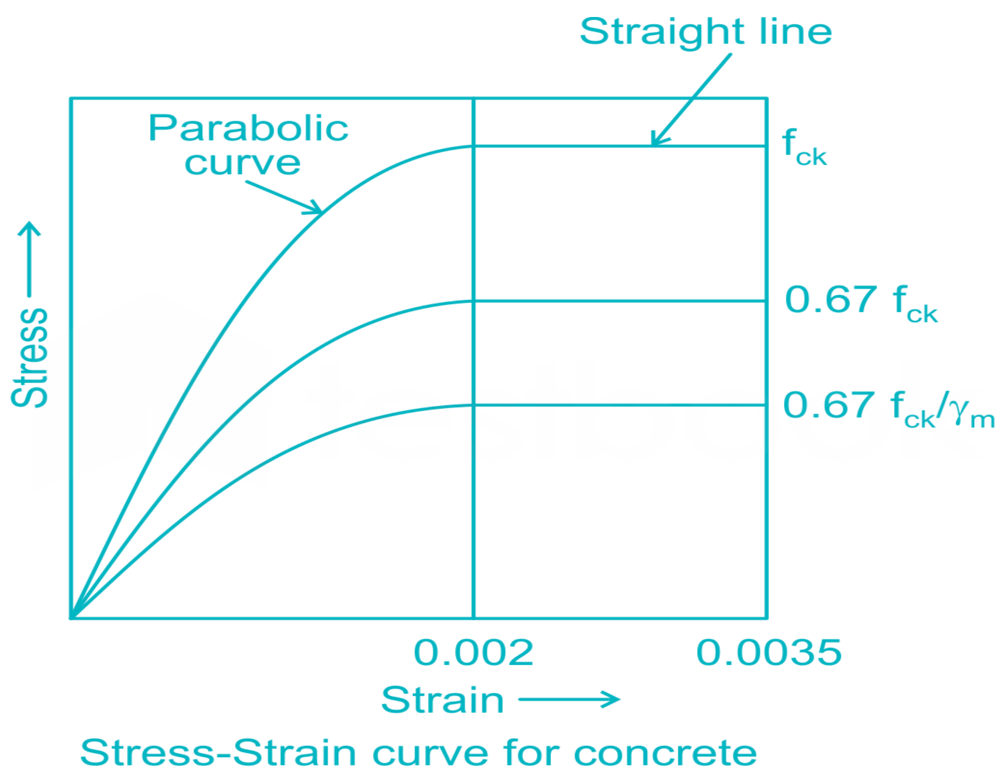


Figure 3: Stress-Strain curve for concrete

Assumptions for the limit state of collapse in compression:

In addition to the assumptions for limit state of collapse in flexure from a to f, the following shall be assumed:

a. The maximum compressive strain in concrete in axial compression is taken as 0.002.

b. The maximum compressive strain at the highly compressed extreme fiber in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at least compressed extreme fiber.

2.8.1.2 Limit state of serviceability

This state corresponds to development of the excessive deformation and is used for checking members in which magnitude of deformation may limit the use of the structure or its components. This state may correspond to:

a. Deflection
b. Cracking
c. Vibration.

a. Control of deflection:

The deflection of a structure or part there of shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. Two methods are given in code for checking the deflections. These are:

- Limiting the span/effective depth ratio given in clause 23.2, IS 456:2000 which should be used in all normal cases, and
- Calculation of deflection given in Appendix C of code to be followed in special cases.

b. Cracking is a very complex phenomenon. Design considerations for crack control would require the following.

- Expression for crack width and spacing, and (Annex F of IS 456:2000).
- Allowable crack widths under different service conditions with due considerations to corrosion and durability of concrete (Clause 35.3.2 of IS 456:2000).
- Unless the calculation of crack widths shows that a greater spacing is acceptable, for the flexural members in normal internal or external conditions of exposure, the maximum distance between bars in tension shall not exceed the value as given in Clause 26.3.3, IS 456:2000.
- Cracks due to bending in compression member subjected to design axial load >0.2fck*Ac, need not be checked. For flexural members (A member which is subjected to design load.

c. Control of Vibration:

A dynamic load is any load of which the magnitude, direction or position varies with the time and almost any RCC structural system may be subjected to one form or another loading during its life-time. Similarly, structural response i.e., resulting stresses or deflections is also time varying or dynamic and is expressed in terms of displacements.

The limit state concept of design of reinforced concrete structures takes into account the probabilistically and structural variation in the material properties, loads and safety factors.

2.9 Detailing Principle for Reinforced Concrete and Steel Structures

2.9.1 Ductile Detailing of Reinforced Concrete Structure

Ductile detailing of reinforced concrete structure is done based on IS 13920:2016 for the provision of compliance with earthquake resistant design philosophy. Special consideration is taken in detailing of linear frame elements (BEAMS & COLUMNS) to achieve ductility in the concrete to localize the formation of plastic hinge in beams and not columns to assure the capacity theory of STRONG COLUMN | WEAK BEAMS.

Detailing provisions of IS 13920:2016 and IS 456:2000 are used extensively for these members to comply with the relevant codes of practice.

2.9.2 Ordinary Detailing of Reinforced Concrete Structure

SP 34 detailing handbook for IS 456 is used extensively for reinforcement detailing of area elements (SLABS & STAIRCASE). Defining the slabs to function as rigid floor diaphragm limits the necessity of special reinforcement provision for slabs eliminating the possibility of out-of-plane bending. Hence same follows for staircase slabs and detailing is done with the help of SP34.

Detailing of Substructures (MAT FOUNDATION) is also done based on SP34 to comply with the design requirement of IS 456:2000.

Reinforcement Detail drawings for typical representative elements are shown in detail in chapter 7 on structural drawings.

Thus, the detailing rules from different handbooks are followed along with enlisted codes of practice and then rebar arrangement is finalized. In this way, detailing of reinforcement is achieved to required specifications by code.

2.9.3 Codal References

The project report has been prepared in complete conformity with various stipulations in Indian Standards, Code of Practice for Plain and Reinforced Concrete IS 456:2000, Design Aids for Reinforced Concrete to IS 456:2000(SP-16), Criteria Earthquake Resistant Design Structures IS 1893 (Part 1):2016, Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces- Code of Practice IS 13920:2016, Handbook on Concrete Reinforcement and Detailing SP-34.

Use of these codes have emphasized on providing sufficient safety, economy, strength and ductility besides satisfactory serviceability requirements of cracking and deflection in concrete structures. These codes are based on principles of Limit State of Design.

2.9.4 Design and Detailing of Structural Element

2.9.4.1 Slab

Slabs are plate elements forming floors and roofs of buildings and carrying distributed loads primarily by flexure. Inclined slabs may be used as ramps for Multi-storey car parks. A staircase can be considered to be an inclined slab. A slab may be supported by beams or walls and may be used as the flange or a T- or L-beam. Moreover, a slab may be simply supported, or cantilever over one or more supports and is classified according to the manner of support.

- One-way slabs spanning in one direction
- Two-way slabs spanning in both direction
- Circular slabs
- Flat slabs resting directly on column with no beams
- Grid floor and ribbed slabs

Slabs are designed using theories of bending and shear as for beams. Since percentage of steel is usually minimum in slab compared to other structural elements. Some points to consider in slab design are:

- Slab is analyzed and designed as having a unit width.
- Compression reinforcement is used only in exceptional cases.
- Shear stresses are very low and shear reinforcement is never provided. It is preferred to increase depth of slab to reduce shear stress.
- Temperature reinforcement is invariably provided at right angles to the main longitudinal reinforcement in slab.
- Slabs are much thinner than beam.

Design Procedure for Slab:

The design procedure for slab elements adopted is given below.

Slab panels are to be designed for the limit state of bending moment and deflection. The thickness of slab is governed by deflection, while the steel areas at mid span and support sections depend on the bending moments.

The slab is designed for 1m wide strips. The subsequent steps are followed to design the slab:

1. Clear size, L_x and L_y.

2. Effective depth is taken from preliminary design.
3. Calculation of effective span:
    a. $L_x + d$
    b. Centre-to-center distance between the supports

    (Less of the above two values are taken.)
4. Calculation of the load (Dead load and Live load)
5. If $L_y/L_x \leq 2$ Two-way slab is designed.
6. Calculation of bending moments:

    Positive and negative moments (α_x and α_y) are taken from Table 26, IS 456:2000, according to L_y/L_x ratio.

    Bending moment is calculated using following formula:

    $M_x = \alpha_x \omega L_x^2$

    $M_y = \alpha_y \omega L_{x2}$

    Where, M_x and M_y are the moments on the strips of unit width spanning l_x and l_y respectively.

    α_x and α_y are bending moment coefficients,

    l_x and l_y are the length of short and long span respectively.
7. Effective depth from moment criteria is calculated to check the required effective depth for moment criteria using following formula: $M_{max} = 0.133 f_{ck} b d^2$
8. Area of steel required for negative moment at continuous edge and positive moment at mid span.

    For short span, it is calculated using following formula,
9. Check for minimum steel from codes: $M_x = 0.87\ f_y A_{st}\ d\ (1- \frac{fy*Ast}{f_{ck}*bd})$

    For Fe 500, Minimum area of steel= 0.12% of bD
10. Maximum spacing:

    Spacing$\leq$ 3d

    $\leq$ 300mm
11. Minimum area of steel required is provided in edge strip.
12. Corner steels (torsion steel):

    Area of each layer of steel at A = 75% of area required for maximum mid span moment

    Area of each layer of steel at B = 0.5 x Area of steel at A

13. Shear is checked at the edge of short span.

$V_{max} = \frac{w*L_x}{2}$

$\tau_{max} = \tau v = \frac{V_{msx}}{bd}$

For p & M25 grade concrete, Ʈc is taken from Table 19, IS 456:2000 and k is taken Clause 40.2.1.1, IS 456:2000.

$\tau'_c = k\tau_c > \tau_v$ O.K.

14. Development Length is checked at both short and long edge.

$$L_d = \frac{\emptyset * \sigma}{4 * \tau_{bd}}$$

So, If $L_d \leq 1.3M_L/V + L_0$

15. Deflection is checked at mid span of short span according to Clause 23.2.1, IS456:2000.

$({}^{L}/d) \leq \alpha\beta\gamma\delta\lambda$

The detail design calculation of critical slab has been shown in Annex IV. Furthermore, While using design of two-way slabs with the help of coefficients, restrained slabs are considered to be divided in each direction into middle and edge strips. The moments apply only to the middle strips and no further redistribution is allowed for these moments. The edge strips have to be reinforced only with nominal minimum steel for crack control. In addition, corner steel to resist the torsion stresses produced in these slabs are provided at discontinuous edges.

2.9.4.2 Column

Columns are compression members whose effective length exceeds three times the least lateral dimension. They are subjected to large axial compression force. Any compression member of a structure, or column in particular, takes the load from flexure and slab members present above it and transfers the load to the foundation and consecutively to the earth below. Thus, it is very important to design columns of adequate size and with adequate reinforcement in order to safely transfer the incoming loads.

Types of columns:

- According to the slenderness ratio of column, it can be divided into short and long columns type. A column is considered as short when slenderness ratio of column in both directions are less than 12, while it is considered as long when its slenderness ratio in both directions is more than 12.

- According to the cross-sectional shape, the column can be circular, square, rectangular and trapezoidal.
- According to material used, column can be of concrete, RCC/PCC, steel, timber, etc.

Design of column is mainly governed by axial load, length & slenderness ratio, material grades, etc. Bending moment, axial compression and shear due to lateral earthquake loads are considered and either axially, uniaxially or biaxially loaded columns are designed particularly. Interaction diagram for columns is used to calculate load carrying capacity or moment carrying capacity of given section of column.

All compression members are to be designed for eccentricity of load in two principal directions, i.e $e = \frac{L}{500} + \frac{D}{30}$, subjected to a minimum of 2cm. After determining the eccentricity, the section is then designed for combined axial load and bending. However, as a simplification, when the value of minimum eccentricity calculated is less than or equal to 0.05D, the design of short axially loaded compression member is to be performed by design equation and if the minimum eccentricity is more than 0.05D, the design of column loaded with combined axial load and bending is to be performed. In our case, even if the minimum eccentricity is less than 0.05D, the columns are probably subjected to moments in both principal axes. Hence, it is required to design these columns as biaxially loaded columns.

Design procedure for column:

The design procedure for biaxially loaded column elements is given below:

1. After modeling and analyzing the structure in ETABS, the ETABS output of frame reinforcement is checked and a column having maximum reinforcement percentage is taken. From details, the axial load and bending moments for corresponding reinforcement area are chosen for corresponding load combination for design.
2. From preliminary design consideration, size of column (b x D) can be obtained, f_{ck} and f_y are obtained as well.
3. Calculation of effective length:
   - Unsupported length of column (L) = Floor-floor height – depth of beam
   - Effective length (L_e) = 0.65 L (IS456: 2000 table-28, consider the column is fixed on both sides)
4. Calculation of slenderness ratio (λ) = L_e / D, IS456: 2000, cl 25.1.2

- For $\lambda \leq 12$, short column
- For $\lambda > 12$, long column

5. Check for minimum eccentricity, IS456: 2000, cl 25.4 & cl. 39.3

•e_x = L/500 + D/30 should be >20mm

<ex,min =0.05D

•e_y= L/500 + B/30 should be >20mm }

<ey,min = 0.05b

- For $e_x < 0.05D$ and $e_y < 0.05b$, column is axially loaded.
- For $e_x > 0.05D$ or $e_y > 0.05b$, column is uni-axially loaded.
- For $e_x > 0.05D$ and $e_y > 0.05b$, column is bi-axially loaded.

6. Calculation of design moments:

- For $e_x \leq e_{x,min}$&$e_y \leq e_{y,min}$, moment due to eccentricity = 0
- For $e_x > e_{x,min}$&$e_y > e_{y,min}$, moment due to eccentricity = M_e = P_u*e
- If the columns are long, there occur additional moments M_{ax} and M_{ay} as well.
- Design moments = M_d = M_u (from ETABS) + M_e + M_a

7. Using interaction diagram SP16: 1980,

- Assume appropriate percentage of reinforcement for the column considered and assume d'=effective cover, such that d'/D and d'/b gives the values corresponding to particular interaction chart type.
- Calculate $\frac{P}{f_{ck}}$&$\frac{P_u}{f_{ck}bD}$, and assuming either reinforcement to be placed equally on two or four sides, corresponding interaction diagram is observed and $\frac{M_{ux,1}}{f_{ck}bD^2}$&$\frac{M_{uy,1}}{f_{ck}bD^2}$ 2 are obtained. By multiplying the factor with fckbD^2& fckDb^2, we determine the ultimate moments.

8. Check for equation on columns subjected to biaxial bending:

- Determine $P_{uz} = 0.446 f_{ck} A_{concrete} + 0.75 f_y A_{steel}$
- Determine P_u / P_{uz} and determine α^n from SP16: 1980, cl3.3
- Check for equation: $\left(\frac{M_{dx}}{M_{ux,1}}\right)^{an} + \left(\frac{M_{dy}}{M_{uy,1}}\right)^{an} \leq 1$

9. Calculation of reinforcement area:

The percentage of steel, for which above criteria satisfies, is taken under consideration. $A_{s,min}$ = 0.8% of A_g and $A_{s,max}$ = 6% of A_g is calculated and checked with considered p%. Required area of reinforcement is calculated.

10. Provide adequate number of bars (considering two or four side equally distributed) such that provided area of reinforcement is greater than required area of reinforcement. But provide the bars considering the economic aspect of construction.
11. For shear reinforcements on column, ultimate shear force due to loading is determined from ETABS.
12. IS13920: 2016 cl 7.5, determine plastic shear force on column due to adjacent beam moments using formula:

$$V_{px} = 1.4\left(\frac{M_{ux}^{s,L} + M_{ux}^{h,R}}{clear\ span}\right) or\ 1.4\left(\frac{M_{ux}^{h,L} + M_{ux}^{s,R}}{clear\ span}\right)$$

$$V_{py} = 1.4\left(\frac{M_{uy}^{s,L} + M_{uy}^{h,R}}{clear\ span}\right) or\ 1.4\left(\frac{M_{uy}^{h,L} + M_{uy}^{s,R}}{clear\ span}\right)$$

13. Determine design shear forces as the greater among V_u and V_p.
14. Determine design shear strength of concrete:
    - Calculate % of steel provided in longitudinal reinforcement
    - Using table-19, determine τ_c.
    - For compression members, determine $\tau_{cd} = k\ \delta\tau_c$
15. Determine shear strength due to design shear forces $\tau_v = \frac{V_u}{bD}$ and determine $\tau_{c,max}$ from IS456: 2000, table 20.
    - If $\tau_{cd} > \tau_v$, minimum shear reinforcement should be provided.
    - If $\tau_{cd} < \tau_v < \tau_{c,max}$, minimum shear reinforcement should be provided.
    - If $\tau_v > \tau_{c,max}$, redesign the section.
16. Determine $V_{us} = (\tau_v - \tau_{cd})\ bD$ for shear reinforcement required.
17. Adopt diameter and legs of stirrup bars and calculate spacing of ties. Determine whether extra tie is required or not as per IS456: 2000, cl. 26.5.3.2.b
18. Perform ductile detailing and design checks using IS13920: 2016:
    - Check minimum dimension of column
    - Check cross-sectional aspect ratio of column (should be greater than 0.45)
    - Rebar Splice Design
    - Check for minimum diameter of transverse reinforcement
    - Check for maximum spacing of links

- Design for spacing of links over confining zone, as per IS13920: 1993, cl. 8.1,

$$A_{sh} \geq \begin{cases} 0.18\ s_v\ h\frac{fck}{fy}\left(\frac{Ag}{Ak}-1\right) \\ 0.05\ s_v h\frac{fck}{fy} \end{cases}$$

Determine spacing of links and check for this spacing over confining zone.

The detail design calculation of critical column has been shown in Annex IV. The reinforcement area and detailing is then performed for each of the critical column of each blocks A, B and C.

2.9.4.3 Beam

Beam is a flexural member which distributes the vertical load to the column and resists the bending moment. 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. 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 beams 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.

The design of the beam deals with the determination of the beam section and the steel required. Here, the design of beam is governed by limit state of moment, shear and deflection. Shear stirrups are provided to take care of the excess shear, beyond the shear capacity of the concrete section of the beams.

We have referred IS 456:2000 for the design and checked the values with ductile design code IS 13920:2016.

Beam is designed as rectangular beam. Dimension of the beam was fixed from preliminary design.

Design procedure for beam:

1. Size of the beam from preliminary design.
2. Factored bending moment (M_u) and factored shear force (V_u) was taken from ETABS analysis.

3. Assuming diameter of reinforcement bars, with 25 mm clear cover, effective depth is calculated as d = D-(clear cover + stir up + longitudinal dia/2)
4. Determination of limiting bending moment is calculated using following formula:

$M_{u,lim}$=0.36* ($\frac{x}{d}$ul) * {1-0.42 ($\frac{x}{d}$ul)} *bd^2fck

If M_u>M_{lim}, Doubly reinforced section is designed. If
M_u<M_{lim}, Singly reinforced section is designed.

5. Area of tension steel required for M_{lim} is calculated as:

M_{lim} = 0.87*f_y* A_{st} d* $\{1 - \frac{A_{st}*F_y}{b*d*f_{ck}}\}$

6. Area of tension steel required for additional bending moment (M_u - M_{lim}) is calculated as

M_u= $(f_{sc} - f_{cc}) * A_{sc} * \{d - d'\}$

where, fsc for $\frac{d}{d'}$ is taken from Table F, SP-16.

f_{cc} = 0.446f_{ck}

7. Area of compression steel required for additional bending moment M_u - M_{lim} is calculated:

(f_{sc}- f_{cc})A_{sc} = 0.87$f_y$$A_{st2}$

A_{st}= A_{st1} + A_{st2}

8. Check for minimum area of tension steel from Clause 26.5.1.1, IS 456:2000.

$A_o = \frac{0.85*bd}{f_y}$

9. Check for maximum area of tension steel from Clause 26.5.1.1 (a), IS 456:2000 A_o= 0.04 bD
10. Check for shear

Permissible shear stress τ_c is taken from Table 19, IS 456:2000 for designed p% steel.

Nominal Shear stress (τv) =$\frac{V_u}{bd}$

$\tau_{c,max}$ is taken from Table 20, IS 456:2000 for designed grade of concrete, if

$\tau_c \leq \tau_v \leq \tau_{c,\ max}$

Design shear force, V_{us} - V_u – $\tau_c$bd

where, $\frac{A_{sv}}{sv} = \frac{V_{us}}{0.87*f_y*d}$

Area and spacing of the stirrups are taken considering, spacing x< 0.75d and <300mm.

11. Check for ductility

Percentage of minimum and maximum area of tension reinforcements according to IS 13920:1993:

11.1. The tensile steel ratio on any face at any section shall not be less than; Tensile steel ratio $\geq 0.24 \frac{\sqrt{f_{ck}}}{f_y}$bd

11.2. The maximum steel ratio on the face at any section shall not exceed

Maximum steel ratio ≤ 0.025

$\frac{A_{stmax}}{bD} \leq 0.025$

12. Ductility check for shear

12.1. The spacing of hoops over a length of 2d at either end of a beam shall not exceed $\frac{d}{4}$ and at $\frac{d}{2}$ for mid span.

12.2. 8 times the diameter of the smallest longitudinal bar. However, it need not be less than 100 mm.

2.9.4.4 Foundation

The foundation forms a very important part of the structure which transfers the load to the soil on which it rests. The ground surface in contact with the lower surface of the foundation is known to be the base of the foundation. The ground on which the foundation rests is called the subgrade soil. A foundation, thus, should be designed to safely transmit the load of the structure on to a sufficient area of the soil so that the stresses induced in the soil are within the safe limit of the bearing capacity of the soil, as if the soil is overstressed, it may lead to shear failure resulting in the sliding of the soil along a plane of rupture and thus result in the collapse of the structure.

Foundations can be broadly classified into shallow and deep foundations. If the depth of foundation is equal to or less than its width, the foundation is classified as shallow while if the depth of foundation is greater than its width, the foundation is classified as deep. Shallow foundation can be further classified as strip footing, isolated footing, combined footing, raft footing. Deep foundation can be classified as pile foundation and well foundation.

Mat foundation is used where the load to be supported is too great or the bearing capacity of the soil is too less. When the area required for individual footings become too great such that they overlap with each other or cover more than one-half of the area, raft foundation can provide a suitable solution. A raft foundation may consist of a slab of uniform thickness or slab stiffened by beams either above or below the strap or an inverted flat slab floor. The raft covers the entire building area and supports all the walls and columns. In our case, we plan to design a slab-only type raft foundation which is similar to inverted flat slab with soil pressure as the load on the plate.

Design procedure of mat foundation:

Design of mat foundation is performed by dividing a whole mat foundation into several strips of definite width and these strips are considered as beam having definite length in order to determine the maximum bending moment to which it is subjected, i.e.$\frac{\omega L^2}{10}$, where ω = soil pressure intensity per unit length of the beam. This bending moment is taken to determine the depth and area of steel reinforcement required per unit strip of foundation as slab. The detail design procedure of mat foundation can be listed as follows:

i. Using the soil parameters and Rankine's formula, determine the minimum depth of foundation below the ground level.
ii. The minimum area of foundation required to transfer the above load to the ground is calculated using the formula:

 $A = \frac{1.1*P}{q_u}$ where, P = total unfactored axial load on to the footing from columns, shear walls and stairs above.
iii. Layout the desired foundation and determine the geometric center of this mat foundation.
iv. Consider an origin and determine the x and y coordinates for each point of action of load. Multiplying this load with its x and y distance w.r.t. origin, calculate the moment due to this load along the corresponding axes.
v. Determine load centroid on this foundation, and calculate its eccentricity w.r.t. the geometric center of the foundation; i.e:

 $\bar{X} = \frac{\Sigma M_x}{\Sigma P_u}$, $\bar{Y} = \frac{\Sigma M_y}{\Sigma P_u}$, $e_x = X-\bar{X}$, $e_y = Y-\bar{Y}$

Calculate moment on foundation due to the total axial load over this eccentricity on both axes. Calculate moment of inertia along both axes as well.

vi. Determine stress due to loading on different point of action of loads and corners of the foundation area, which is to be checked in equivalent with soil bearing capacity of the soil support below, using formula:

$$\sigma = \frac{P_u}{A} \pm \frac{M_{ex}}{I_{xx}} y \pm \frac{M_{ey}}{I_{yy}} x$$

Where, x and y is the distance of point considered from the geometric center of the foundation. Divide the raft into several strips along both X-axis and Y-axis to design the foundation strips as equivalent beams. The respective maximum soil pressure along the strip is ωL^2 taken and bending moment per unit width is determined using the formula. .

vii. Calculation of depth using two major criteria:

- Taking the ultimate bending moment from all calculated BMs above, using IS456: 2000 cl. 38.1, we calculate depth using:

  $$M_{u,lim} = 0.36\frac{x_{u,lim}}{d}\left(1 - 0.42\,\frac{x_{u,lim}}{d}\right) bd^2 f_{ck}$$

  Where, in our case, for Fe500, $\frac{x_{u,lim}}{d}$= 0.46 & f_{ck} = 25N/mm d

- Considering the critical corner, edge and center column based on the largest axial load that it carries, depth is calculated by checking on 2-way shear, i.e.

  The design shear capacity $K_s\tau_c$ should be greater than the calculated shear stress (τ_v) at critical section at d/2 distance from the face of the column in order to allow the foundation two safely resist such two-way shear stress. i.e. $\tau_v \leq K_s\tau_c$ where,

  $K_s = (0.5 + \beta_c)$ but not greater than 1, β_c being the ratio of short side to long side of the column,

  $$\tau_c = 0.25\sqrt{f_{ck}}$$

&, $\tau_v = \frac{Ultimate\ axial\ load\ on\ colum}{Area\ under\ 2\ way\ shear}$

The maximum value of depth is taken considering either maximum moment criteria or two-way shear criteria that governs it.

ix. Calculation of reinforcement area per unit strip width:

Using the maximum effective depth required, the area of steel reinforcement along both X and Y axes can be determined using the maximum moments along X and Y axes obtained using the load stress as mentioned in step 7. The area is calculated using formula:

$$0.87*F_y* A_{st}\, d* \{1 - \frac{A_{st}*F_y}{b*d*f_{ck}}\}$$

x. Check for development length:

As in slab design, the development length for the anchorage of the reinforcement in the foundation is checked, i.e.$L_d \leq 1.3\frac{M_l}{V} + L_0$

xi. Design of load transfer from column to footing:

According to IS456: 2000, cl.34.4.1, where the permissible bearing stress on the concrete in the supporting or supported member would be exceeded, reinforcement shall be provided for developing the excess force, either by extending the longitudinal bars into the supporting member, or by dowels.

The bearing stress of column is determined and is compared with any excess stress to be transferred if required.

Transfer load from column to footing by extending the column reinforcement into the footing or using dowels up to length equal to L_d, such that extended longitudinal reinforcement or area of dowels is at least 0.5% of CSA of the supporting column.

xii. Calculation of chair dimensions required to support the top and bottom reinforcement during concreting.

2.9.4.5 Lift/Shear Wall

In structural engineering, a shear wall is a vertical element of a seismic force resisting system that is designed to resist in-plane lateral forces, typically wind and seismic loads. In many jurisdictions, the International Building Code and International Residential Code govern the design of shear walls.

A shear wall resists loads parallel to the plane of the wall. Collectors, also known as drag members, transfer the diaphragm shear to shear walls and other vertical elements of the seismic force resisting system. Shear walls are typically light-framed or braced wooden walls with shear panels, reinforced concrete walls, reinforced masonry walls, or steel plates.

Design procedure of lift/shear wall:

Following steps are followed to design a shear/lift wall:

1. Determine the lump mass and seismic weight of the shear wall.
2. Determine the base shear and moment at different point where mass is lumped. Base Shear (V_b) =Ah*W

$$\text{Storey Shear } (Q_i) = V_b W_i h_i^2 / \Sigma W_i h_i^2$$

$$\text{Moment} = \Sigma Q_i h_i$$

3. Determine the minimum and additional eccentricity by using following equation:

$$e_{min}(t) = 0.05t \text{ and } e_a = H^2 \text{ we } /2500t \text{ 4.}$$

Determine the ultimate load carrying capacity as:

$$P_{uw} = 0.3(t\text{-}1.2e\text{-}2e_a)f_{ck}$$

5. Determine the moment, shear and load applied on each strip of shear wall along X and Y-direction.
6. Calculate main vertical reinforcement referring to chart of SP-16.
7. Determine spacing of bars.
8. Calculate the area of horizontal reinforcement steel bars. A_h =0.2% of bH
9. Check for shear as per IS 456-2000 Cl.32.4.2.

2.9.4.6 Basement Wall

Basement wall is constructed to retain the earth and to prevent moisture from seeping into the building. Since the basement wall is supported by the mat foundation, the stability is ensured and the design of the basement wall is limited to the safe design of vertical stem.

Basement walls are exterior walls of underground structures (tunnels and other earth sheltered buildings), or retaining walls must resist lateral earth pressure as well as additional pressure due to other type of loading. Basement walls carry lateral earth pressure generally as vertical slabs supported by floor framing at the basement level and upper floor level. The axial forces in the floor structures are, in turn, either resisted by shear walls or balanced by the lateral earth pressure coming from the opposite side of the building.

Although basement walls act as vertical slabs supported by the horizontal floor framing, keep in mind that during the early construction stage when the upper floor has not yet been built the wall may have to be designed as a cantilever, however the basement wall is designed as propped cantilever in this project. This is based on the assumption that the backfilling is withheld or basement wall strutted until final construction of the wall.

Design Procedure of basement wall:

1. Determine the design constants, height of wall, unit weight of soil, angle of friction of soil, surcharge load, safe bearing capacity of soil.
2. Calculate slenderness ratio (l/d)
3. Lateral load due to soil pressure,

$$P_a = K_a \times \gamma \times h^2/25$$

4. Lateral Load due to surcharge load,

$$P_s = K_a \times W_s \times h$$

5. Calculate bending moment at base of wall due to these lateral loads.
6. Check for depth of wall required from moment consideration

$$M_u = 0.133 f_{ck}\, bd^2$$

7. Calculate requirement of main steel reinforcement

$$M = 0.87 f_y A_{st} \left(d - \frac{f_y A_{st}}{f_{ck} b}\right)$$, where A_{st} is steel required

8. Calculate the shear and determine the requirement of shear steel.
9. Check for deflection

$$\text{Allowable deflection} = \frac{l_{eff}}{250}$$

$$\text{Actual Deflection} = \frac{P_s l_{eff}^3}{8EI} + \frac{P_a l_{eff}^3}{30EI}$$

10. Calculate the horizontal reinforcement bar

Area of Hz. Reinforcement = 0.002Dh

As the temperature change occurs at front face of basement wall, 2/3 of horizontal reinforcement is provided at front face and 1/3 of horizontal reinforcement is provided in inner face.

11. Curtailment Reinforcement

No bars can be curtailed in less than L_d distance from the bottom of stem.

$$L_d = \frac{0.87 f_y * \Phi}{4\tau}$$

2.10 Drawings

As specified in the requirement of the project assignment, the report also includes the following drawings:

1. Architectural Plan of Typical floors, Elevation and Cross Section of the building.
2. Detailed Structural drawing of full-size beam, full size column, slab, staircase, mat foundation and shear walls. Longitudinal and Cross section drawings are made to represent specifically the proper detailing of rebar in individual elements, at beam column joints, at the end support of slabs, in staircase and in the foundation.

3. PRELIMINARY DESIGN OF STRUCTURAL MEMBERS

Before proceeding for the actual modeling of the building, it is necessary to fix approximate dimensions for the structural elements. This is done through preliminary design and acts as guidelines in analysis which are liable to be changed in future after response evaluation. Drawings received from architect were thoroughly studied and elements at maximum exploited location were chosen for preliminary design.

3.1 Preliminary Design of Slab

Preliminary design of RCC slab for the floor and roof of the proposed building is done in such a way that it complies with deflection control criteria of **IS 456:2000** and behavior of floor slab as a rigid diaphragm. Being equal spacing of columns in both the axes, planar dimensions of all slabs being equal, on ly a single panel is taken for preliminary design.

lx=6000mm

ly=6000mm

<u>Classification of slab</u>

$$\frac{ly}{lx} = \frac{6000}{6000} = 1 < 2$$

So, the slab behaves as two-way slab.

For slab, as per **IS 456:2000 CL 24.1**, the provision for beams app ly to slabs also.

Therefore, from **IS 456:2000 CL 23.2**

We have,

$$\frac{span(l)}{effective\ depth\ (d)} \leq \alpha\beta\gamma\delta\lambda$$

α = 26 (for continuous slab) (IS **456:2000 Cl 23.2.1**)

β = span factor = 1 (Since span is less than 10m) (**IS 456:2000 Cl 23.2.1 b**)

For γ (modification factor for tension reinforcement)

Assuming 0.2% reinforcement

$$fs = 0.58 * fy * \frac{Area\ of\ cross - section\ of\ steel\ required}{Area\ of\ cross - section\ of\ steel\ provided}$$

$$s = 0.58 * fy * \frac{1}{1} = 290N/mm^2 \text{(Assuming } f_y\text{-500 N/mm}^2\text{)}$$

From Figure 4 IS 456:2000, γ =1.40

δ = modification factor for compression reinforcement = 1 (Since no compression reinforcement is to be provided in slab)

λ = Reduction factor for ratios of span to effective depth for flanged section =1 (Since slab is rectangular in section)

Calculating effective depth,

$$\frac{span(l)}{effective\ depth\ (d)} \leq \alpha\beta\gamma\delta\lambda$$

$$\frac{6000}{d_{eff}} \leq 26 * 1 * 1.4 * 1 * 1$$

$$d_{eff} \geq 164.84\ mm$$

Since slab depth to be provided is greater than 150 mm, it will lead to higher seismic mass. Therefore, we have divided slab into both axes by providing secondary beams.

<u>Re-calculating required depth</u>

l_x=3000mm

l_y=3000mm

Classification of slab: l_y/ l_x=3600/3600=1<2

So, the slab behaves as two-way slab.

All factors calculated above are valid for this slab too.

Calculating effective depth,

$$\frac{span(l)}{effective\ depth\ (d)} \leq \alpha\beta\gamma\delta\lambda$$

$$\frac{3000}{d_{eff}} \leq 26 * 1 * 1.4 * 1 * 1$$

$$d_{eff} \geq 82.418\ mm$$

∴ D (Total depth of slab) = d_{eff} + clear cover + 0.5* diameter of bar

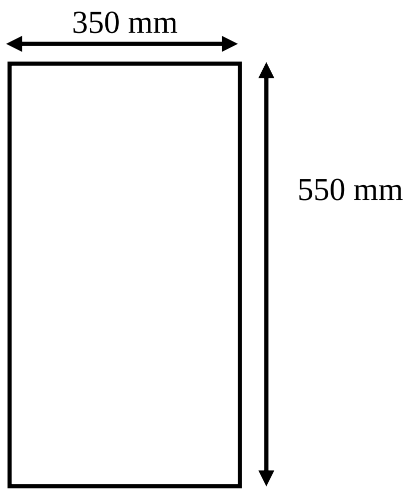
= 82.418+20+0.5*12

= 108.418 mm

Adopt D= 140mm for all slabs.

3.2 Preliminary Design of Beam

Preliminary design of RCC beam element was done similar to that of slab i.e., through deflection criteria.



Preliminary design of Main Beam

Depth

l=6000mm

From **IS 456:2000 CL 23.2**

We have,

$$\frac{span(l)}{effective\ depth\ (d)} \leq \alpha\beta\gamma\delta\lambda$$

Calculating effective depth,

$$\frac{span(l)}{effective\ depth\ (d)} \leq 15$$

$$\frac{6000}{d_{eff}} \leq 15$$

$$d_{eff} \geq 500\ mm$$

∴ D (Total depth of beam) = d_{eff} + clear cover + 0.5* diameter of bar

= 500+25+0.5*20

= 535 mm

Rounding off to nearest 50mm so as to ease in construction,

Adopt D = 550mm

Breadth

$$\frac{D}{b} = 1.5\ to\ 2$$

Taking $\frac{D}{b} = 1.75$

∴ b = 314.28 mm

Adopt B = 350mm

Preliminary design of Secondary Beam

l=3000 mm

From **IS 456:2000 CL 23.2**

We have,

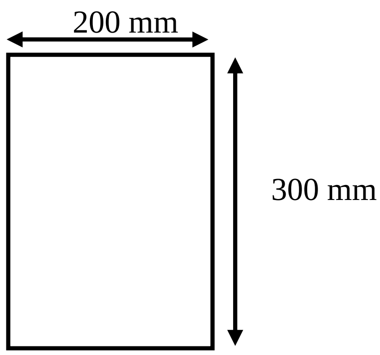
$$\frac{span(l)}{effective\ depth\ (d)} \leq \alpha\beta\gamma\delta\lambda$$

Calculating effective depth,

$$\frac{span(l)}{effective\ depth\ (d)} \leq 12$$

$$\frac{3000}{d_{eff}} \leq 12$$

$$d_{eff} \geq 250\ mm$$



∴ D (Total depth of beam) = d_{eff} + clear cover + 0.5* diameter of bar
= 250+25+0.5*20
= 285 mm

Adopt D = 300mm

Breadth

$$\frac{D}{b} = 1.5 - 2$$

Taking $\frac{D}{b} = 1.75$

∴ b = 171.43 mm, Adopt B = 200 mm

3.3 Preliminary Design of Column

Load Calculation:

Tabulation of unit weight used

Partition Wall Weight	**16.2324 KN/m**
Height(m)	3.35
Thickness(mm)	230
Area(m^2)	24.12
Finishing (12.5mm)	25
Weight (kN/m^2)	4.8455

Beam Weight per length	**4.125 KN/m**
Depth(mm)	550
Width(mm)	300
Unit weight (kN/m^3)	25

Secondary Beam Weight per length	**2.1875 KN/m**
Depth(mm)	350
Width(mm)	250
Unit weight (kN/m^3)	25

Slab Weight per area	**3.5 KN/m^2**
Depth (mm)	140
Unit weight (kN/m^3)	25

Individual calculations are shown in tabular form below.

Floor finish

Categories	**Thickness**	**Unit Weight**	**Area(m^2)**	**Load (kN)**
Screed	25 mm	$21kN/m^3$	35.561756	18.6699219

Finishing	15mm	0.12kN/m^3	35.561756	0.064011161
Celling	13mm	20.4kN/m^3	35.561756	9.430977691
			Total	28.16491075

Imposed Load (Typical)

At	Unit Weight	Area(m^2)	Load (kN)
General Rooms	2.5 kN/m^2	35.561756	88.90439

Imposed Load (Roof not accessible)

At	Unit Weight	Area(m^2)	Load (kN)
Roof	0.75 kN/m^2	35.561756	26.671317

Imposed Load (Roof)

At	Unit Weight	Area(m^2)	Load (kN)
Roof	0.4 kN/m^2	36	14.4

.

Dead Load (Typical Floors)

Categories	Length/Area	Unit weight	Load(kN)	Remarks
Beam	12m	4.8125 kN/m	57.75	
Secondary Beam	24m	1.5 kN/m	36	
Slab	35.561756m^2	3.75 kN/m^2	133.356585	
Infill Wall	12m	10.599kN/m	127.19322	30% opening
Partition Wall (brick)	11.398m	10.599kN/m	120.807402	
Partition Wall (UPVC)	3.056*3.465 m^2	1 kN/m^2	10.58904	
		Total	485.696247	

Dead Load of column

Assuming 600mm*600mm column initial ly,

Type	Height	Area(mm^2)	Unit Weight	Load (kN)
Normal	3.465m	360000	25kN/m^3	31.185

∴**Total Load at Bottom of Ground Floor Column**

Load due to	Multiplier	Unit value(KN)	Load(KN)
Floor Finish	6	28.16	168.96
Imposed Load (Typical)	5	88.90439	444.52195

Imposed Load (Roof)	1	14.4	14.4
Imposed Load (Roof not accessible)	1	26.67	26.67
Dead Load (Typical Floors)	6	485.696247	2914.177482
Column Self Wt.	6	31.185	187.11
		Total Load	3755.839432
		Factored Load	5633.759148

Floor Finish: Ground Floor

Categories	**Thickness**	**Unit Weight**	**Area(m^2)**	**Load (kN)**
Screed	25 mm	21kN/m^3	36	18.9
Finishing	15mm	0.12kN/m^3	36	0.0648
Celling	13mm	20.4kN/m^3	36	9.5472
			Total	28.512

Imposed Load (Typical): Ground floor

At	**Unit Weight**	**Area(m^2)**	**Load (kN)**
General Rooms	2.5 kN/m^2	36	90

Dead Load: Ground floor

Categories	**Length/Area**	**Unit weight**	**Load(kN)**	**Remarks**
Beam	12m	4.8125 kN/m	57.75	
Secondary Beam	24m	1.5 kN/m	36	
Slab	36m^2	3.75 kN/m^2	135	
		Total	228.75	

Floor Finish: Upper Basement

-Categories	**Thickness**	**Unit Weight**	**Area(m^2)**	**Load (kN)**
Screed	25 mm	21kN/m^3	27	14.175
Finishing	15mm	0.12kN/m^3	27	0.0486
Celling	13mm	20.4kN/m^3	27	7.1604
			Total	21.384

Imposed Load (Typical): Upper Basement

At	**Unit Weight**	**Area(m^2)**	**Load (kN)**
General Rooms	5 kN/m^2	27	67.5

Dead Load of Upper Basement

Categories	**Length/Area**	**Unit weight**	**Load(kN)**	**Remarks**

Beam	9m	4.8125 kN/m	43.3125	
Secondary Beam	18m	1.5 kN/m	27	
Slab	27m^2	3.75 kN/m^2	101.25	
		Total	171.5625	

Dead Load of column

Assuming 600mm*600mm column initially,

Type	**Height**	**Area(mm^2)**	**Unit Weight**	**Load (kN)**
Normal	3.465m	360000	25kN/m^3	31.185

∴**Total Load at Bottom of Lower Basement Column**

Load due to	**Multiplier**	**Unit value(KN)**	**Load(KN)**
Floor Finish Ground Floor	1	28.512	28.512
Imposed Load (Typical) Ground floor	1	90	90
Dead Load (of Ground Floor)	1	228.75	228.75
Floor Finish Upper Basement	1	21.384	21.384
Imposed Load (Typical) Upper Basement	1	67.5	67.5
Dead Load (of Upper Basement)	1	171.5625	171.5625
Column Self Wt.	2	31.185	62.37
		Total Load	670.0785
		Factored Load	1005.11775

Total Factored Load=**6638.876898 kN**

Calculation of dimensions required for above calculated load

From **IS 456:2000, CL 39.3**

Assuming 2% steel and for f_y =500N/mm2 and M25 grade of concrete

$P_u = 0.4 * f_{ck} * A_c + 0.67 * f_y * A_{sc}$

$P_u = 0.4 * f_{ck} * A_g * (1 - 0.02) + 0.67 * f_y * A_g * 0.02$

$A_g = \dfrac{Pu}{0.4 * 25 * (1 - 0.02) + 0.67 * 500 * 0.02} = 634086.628 mm^2$

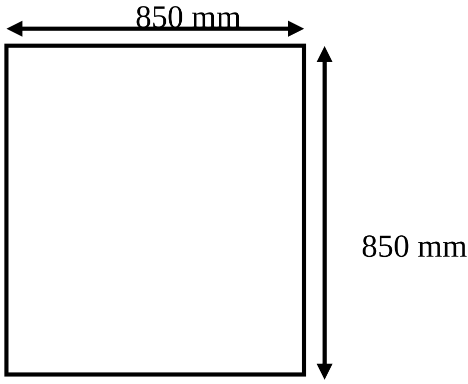
For square column,

$D^2 = 634086.628$

$\therefore D = 796.295 mm$

Adopt D = 850mm

$\therefore Size\ of\ columns = 850 * 850$



4 LOAD CALCULATION

- **Slab:**

UPPER BASEMENT:

Slab ID	Length (m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness (m)	Dead Load (kN)	Live Load (kN/m2)	Live Load (kN)	Total Load (kN)
BF26	24	24	576	25	0.14	2016	5	1440	3456
Deduction	4.4	2	-8.8	25	0.14	-30.8	5	-22	-52.8
Deduction	6	6	-36	25	0.14	-126	5	-90	-216
Deduction	1.2		-9.047	25	0.14	-31.6672	5	-22.61946711	-54.286
OTHERS	12	3	144	25	0.14	504	5	360	864
Total									3996.9132

GROUND FLOOR:

Slab ID	Length (m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness (m)	Dead Load (kN)	Live Load (kN/m2)	Live Load (kN)	Total Load (kN)
DE13	9	6	54	25	0.14	189	5	135	324
Deduction	4.4	2	-8.8	25	0.14	-30.8	5	-22	-52.8
BC23	6	6	36	25	0.14	126	3.484029	62.712	188.712
EF23	6	6	36	25	0.14	126	3.1039	55.870	181.8705
AB34	6	3	18	25	0.14	63	3.198055	28.782	91.7825
AB45	6	3	18	25	0.14	63	3	13.5	76.5
BC34	6	6	36	25	0.14	126	3.5313	63.564	189.5642
BC45	6	6	36	25	0.14	126	3.0345	54.621	180.621
CE35	12	12	144	25	0.14	504	4	288	792
EG35	12	9	108	25	0.14	378	4	216	594
BC56	6	6	36	25	0.14	126	40.837	735.06	861.066
CD57	9	6	54	25	0.14	189	4	108	297
DE57	9	6	54	25	0.14	189	3.825625	103.29	292.2918
EF56	6	6	36	25	0.14	126	4	72	198
Total									4214.6090

FIRST FLOOR:

Slab ID	Length (m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness (m)	Dead Load (kN)	Live Load (kN/m2	Live Load (kN)	Total Load (kN)
12DE&23DE									
General Store Stationary	7.14	3.64	25.98	25	0.14	90.963	5	64.974	155.9376
Room	3	2.215	6.645	25	0.14	23.257	2.5	4.1531	27.412
Lobby	4.14	2	8.28	25	0.14	28.98	4	16.56	45.54
23EF(Room)	5.965	6.075	36.23	25	0.14	126.830	2.5	22.6483	149.4791
34EF(Meeting Hall)	6	5.89	35.34	25	0.14	123.69	2.5	22.0875	145.7775
45EF&45FG									
General Room	4.357	6.115	26.64	25	0.14	93.2506	2.5	16.6519	109.90260
Toilet	1.5	2.175	3.262	25	0.14	11.4187	2	1.63125	13.05
PA Room	5.69	2.85	16.21	25	0.14	56.7577	2.5	10.1353	66.893062
Pantry	1.42	2.175	3.088	25	0.14	10.8097	2.5	1.93031	12.740062
34FG(Room)	6.275	3.075	19.29	25	0.14	67.5346	2.5	12.0597	79.594453
34AB(Room)	6.4	3.275	20.96	25	0.14	73.36	2.5	13.1	86.46
45AB(Room)	5.925	3.275	19.40	25	0.14	67.9153	2.5	12.1277	80.043046
23BC(Room)	5.1	6.44	32.84	25	0.14	114.954	2.5	20.5275	135.4815
	1.184	3.44	4.072	25	0.14	14.2553	2.5	2.5456	16.80096
34BC(Room)	6.275	3.17	19.89	25	0.14	69.6211	2.5	12.4323	82.053468
	2.69	2.83	7.612	25	0.14	26.6444	2.5	4.75793	31.4023
Store	4.33	2.775	12.01	25	0.14	42.0551	5	30.0393	72.0945
45BC(Room)	6.2	2.92	18.10	25	0.14	63.364	2.5	11.315	74.679
	1.94	3.03	5.87	25	0.14	20.5737	2.5	3.67387	24.247575
Store	4.025	3.03	12.19	25	0.14	42.6851	5	30.4893	73.1745

56BC(Toilet)	4.2	2.7	11.34	25	0.14	39.69	2	5.67	45.36
Toilet	4.565	2.115	9.654	25	0.14	33.7924	2	4.8274	38.6199
Disabled Toilet	2.5	1.85	4.625	25	0.14	16.1875	2	2.3125	18.5
Lobby	1.42	2.9	4.118	25	0.14	14.413	4	8.236	22.649
	6.1	1.9	11.59	25	0.14	40.565	4	23.18	63.745
56CD&67CD								0	
Deputy General Room	5.965	4.464	26.62	25	0.14	93.197	2.5	16.6423	109.83
PA	3	4.465	13.39	25	0.14	46.882	2.5	8.37187	55.2543
Pantry	2.066	1.42	2.933	25	0.14	10.268	2.5	1.833	12.101
Store	1.5	1.42	2.13	25	0.14	7.455	5	5.325	12.78
Toilet	2.29	1.42	3.25	25	0.14	11.381	2	1.6259	13.002
56DE&67DE									
Archive	4.425	1.64	7.257	25	0.14	25.399	5	18.1425	43.542
Financial Administration	3.075	5.965	18.34	25	0.14	64.198	2.5	11.4639	75.6622
Lobby	6.275	1.2	7.53	25	0.14	26.355	4	15.06	41.415
Room	1.8	1.8	3.24	25	0.14	11.34	2.5	2.025	13.365
56EF(Administration)	5.965	8.93	53.26	25	0.14	186.436	2.5	33.292	219.7282
35CE	4.675	4.375	20.45	25	0.14	71.585	2.5	12.783	84.36914
Centre lobby			92.21	25	0.14	322.735	4	184.42	507.155
23CD(BEFORE STAIR)	2.875	5.45	15.66	25	0.14	54.8406	4	31.3375	86.178125
SLAB ASIDE FIRE EXIT			7.423	25	0.14	25.9813	4	14.8465	40.827875
								TOTAL	**2986.8612**

SECOND FLOOR:

Slab Id	Length (m)	Breadth (m)	Area (m^2)	Thickness (m)	Unit Weight (kN/m3)	Dead Load (kN)	Live Load (kN/m2	Live Load (kN)	Total Load (kN)
12DE & 23DE				0.14	25				
Environment Section	4.69	5.965	27.97			97.9154	2.5	17.4849	115.4003
Store	2.34	3.73	8.728			30.5487	5	21.8205	52.3692
Lobby			8.495			29.7358	4	16.9919	46.727725
23EF									
Store	3.34	1.8	6.012			21.042	5	15.03	36.072
Electricity Inspection			29.98			104.958	2.5	18.7425	123.7005
34EF & 34FG									
Store	2.85	1.8	5.13			17.955	5	12.825	30.78
Project Inspection			48.87			171.045	2.5	30.5437	201.58875
45EF & 45 FG									
PA	5.78	2.85	16.47			57.6555	2.5	10.2956	67.951125
Pantry	1.31	2.185	2.862			10.0182	2.5	1.78896	11.807193
Inspection Division			31.46			110.117	2.5	19.6638	129.78136
Toilet	1.5	2.135	3.202			11.208	2	1.6012	12.81
56EF & 56DE									
Meeting Hall	6.075	12.075	73.35			256.744	2.5	45.8472	302.59195
67DE									
Room	3	6	18			63	2.5	11.25	74.25
67CD									
Room			13.37			46.8125	2.5	8.3593	55.1718
Toilet	2.5	1.85	4.625			16.1875	2	2.3125	18.5
56CD									

Secretary	5.89	2.715	15.99			55.9697	2.5	9.9945	65.96431
Power Development Fund			20.00			70.0302	2.5	12.5054	82.53568
56BC									
w/c male	4.12	2.555	10.52			36.8431	2	5.2633	42.1064
w/c female	5.89	2.115	12.45			43.6007	2	6.2286	49.8294
Pantry	2.55	1.76	4.488			15.708	2.5	2.805	18.513
Lobby			8.528			29.848	4	17.056	46.904
23BC, 34BC, 45BC,34AB & 45AB									
Project office			136.1			476.373	2.5	85.0666	561.4401
lobby			7.893			27.6262	4	15.7864	43.4126
35CE									
Fire House Cabinet			90.34			316.204	4	180.688	496.892
Total									2687.0996

THIRD FLOOR:

S.N	Slab Id	Length (m)	Breadth (m)	Area (m^2)	Thickness (m)	Unit Weight (kN/m3)	Dead Load (kN)	Live Load (kN/m2	Live Load (kN)
1	12DE & 23DE								
a.	Office room	4.69	5.965	27.9758			97.9154	2.5	17.4849
b.	Store	2.34	3.73	8.7282			30.5487	5	21.8205
c.	Lobby			8.4959			29.7358	4	16.9919
2	23EF, 34EF & 34FG								
a.	Project Inspection			90	0.14	25	315	2.5	56.25
3	45EF & 45 FG								
a.	PA	5.78	2.85	16.473			57.6555	2.5	10.2956
b.	Pantry	1.31	2.185	2.86235			10.0182	2.5	1.78896
c.	Project Studies Division			31.46215			110.1175	2.5	19.6638

d.	Toilet	1.5	2.135	3.2025			11.20875	2	1.60125
4	56CD, 56DE, 56EF, 67EF & 67CD								
a.	Survey &feasibility study section-II			139.375			487.8125	2.5	87.1093
b.	Toilet	2.5	1.85	4.625			16.1875	2	2.3125
5	56BC								
a.	w/c male	4.12	2.555	10.5266			36.8431	2	5.2633
b.	w/c female	5.89	2.115	12.4573			43.600	2	6.2286
c.	Pantry	2.55	1.76	4.488			15.708	2.5	2.805
d.	Lobby			8.528			29.848	4	17.056
6	23BC, 34BC, 45BC,34AB & 45AB								
a.	Survey &feasibility study section-I			129.95			454.825	2.5	81.21875
b.	lobby			7.8932			27.6262	4	15.7864
c.	Store	3.03	2.03	6.1509			21.52815	5	15.377
7	35CE								
a.	Fire House Cabinet			90.344			316.204	4	180.688
	Total						2112.383		559.742
	Total load						2672.125419		

FORTH:

Slab ID	Length (m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness(m)	Dead Load (kN)	Live Load (kN/m2)	Live Load (kN)	Total Load (kN)
CSN	12	2.389	19.11	25	0.14	66.892	4	38.224	105.116
CSE	12	2.389	19.11	25	0.14	66.892	4	38.224	105.116
CSS	12	2.389	19.11	25	0.14	66.892	4	38.224	105.116
CSW	12	2.389	19.11	25	0.14	66.892	4	38.224	105.116
AB34, AB45	12	3	36	25	0.14	126	4	72	198
BC23, BC34	12	6	72	25	0.14	252	4	144	396
BC45	6	6	36	25	0.14	126	4	72	198
BC56	6	6	36	25	0.14	126	2	18	144
CE35	12	12	144	25	0.14	504	2.5	90	594

Deduction	7.325	7.325	-53.65	25	0.14	-187.7946	2.5	-33.534	-221.3294
CD56	6	6	36	25	0.14	126	3.8193	68.748	194.7482
CD67	6	3	18	25	0.14	63	3.4861	31.375	94.375
DE12	6	3	18	25	0.14	63	3.6441	32.797	95.797
DE23	6	6	36	25	0.14	126	4	72	198
Deduction	4.4	2	-8.8	25	0.14	-30.8	5	-22	-52.8
DE56	6	6	36	25	0.14	126	4	72	198
DE67	6	3	18	25	0.14	63	3.6441	32.797	95.7975
EF23	6	6	36	25	0.14	126	5	90	216
EF34	6	6	36	25	0.14	126	5	90	216
EF45	6	6	36	25	0.14	126	4	72	198
EF56	6	6	36	25	0.14	126	4	72	198
FG34	6	6	36	25	0.14	126	4	72	198
FG45	6	6	36	25	0.14	126	4	72	198
Total									3777.052

TOP FLOOR:

Slab ID	Length(m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness (m)	Dead Load (kN)	Live Load (kN/m2)	Live Load (kN)	Total Load (kN)
North curve part 3-4-5(Terrace)			57.34	25	0.14	200.69	4	114.68	315.382
2-3-4 B-C	12.075	5.965	72.02	25	0.14	252.09	4	144.05	396.1505
4-5 B-C						0		0	0
Project Office	3.915	6.075	23.78	25	0.14	83.242	4	47.567	130.8099
Lobby			6.702	25	0.14	23.458	4	13.404	36.8625
5-6 B-C						0		0	0
Pantry	2.55	1.76	4.488	25	0.14	15.708	2.5	2.805	18.513
Toilet(M)	4.12	2.555	10.52	25	0.14	36.843	2	5.2633	42.1064
Toilet(F)	5.89	2.115	12.45	25	0.14	43.600	2	6.2286	49.8294
Lobby			10.92	25	0.14	38.239	4	21.851	60.0902
Slab (fire exit)			8.336	25	0.14	29.177	4	16.673	45.850
1-2 C-D									

West curve part(Terrace)			8.502	25	0.14	29.757	5	21.255	51.012522
Stairs *									
D-E West curve part(Terrace)			27.48	25	0.14	96.192	5	68.709	164.902
2-3 C-D slab (Stairs)	2.575	5.725	14.74	25	0.14	51.596	4	29.483	81.08031
3-4-5 C-D-E	12	12	144	25	0.14	504	2.5	90	594
Deduction (Void)	-7.325	7.325	-53.65	25	0.14	-187.79	2.5	-33.537	-221.3294
5-6 C-D (Project Office)	6.075	5.89	35.78	25	0.14	125.23	4	71.563	196.799
6-7 C-D									
East curve part (Terrace)			21.06	25	0.14	73.727	4	42.130	115.8581
Disabled Toilet	2.5	1.85	4.625	25	0.14	16.187	2	2.3125	18.5
D-E East curve part (Terrace)			28.67	25	0.14	100.35	4	57.343	157.69411
2-3 D-E									
Store	4.14	3.09	12.79	25	0.14	44.774	5	31.981	76.7556
Multi-media center	3.13	2.575	8.059	25	0.14	28.209	2.5	5.0373	33.24646
Lobby			5.583	25	0.14	19.540	4	11.166	30.707187
Lift *						0		0	0
5-6 D-E-F (Project Office)	6.075	12.075	73.35	25	0.14	256.74	4	146.71	403.455
2-3-4 E-F (Auditorium with fixed seats)	11.965	5.955	71.25	25	0.14	249.38	4	142.50	391.883
4-5 E-F (Lobby)	5.944	6.275	37.29	25	0.14	130.5451	4	74.597	205.1423
3-4-5 South curve part (Terrace)			57.43	25	0.14	201.02	4	114.87	315.9009
									3711.2060

ROOF:

Slab ID	Length (m)	Breadth (m)	Area (m2)	Unit Weight (kN/m3)	Thickness (m)	Dead Load (kN)	Live Load (kN/m2)	Live Load (kN)	Total Load (kN)
Central Slab	24	24	576	25	0.14	2016	1.5	216	2232
Left Slab	6	3	18	25	0.14	63	1.5	6.75	69.75
Right Slab	6	3	18	25	0.14	63	1.5	6.75	69.75
Central Void	7.325	7.325	53.65	25	0.14	-187.794	1.5	20.1208	-167.673
Total						1954.20		249.6208	2203.8261

- **Column:**

FOR ALL FLOORS EXCEPT ROOF:

No. of Column	Width (m)	Depth (m)	Span (m)	Unit Weight (kN/m3)	Load(kN)
All Column 5*5+4*3-1=36	0.85	0.85	3.465	25	2253.11625
Total					2253.11625

FOR ROOF:

No. of Column	Width(m)	Depth(m)	Span(m)	Unit Weight(kN/m3)	Load(kN)	Remarks
5*5+2=27	0.85	0.85	1.7325	25	844.9185938	Span=half of storey height
Total					844.9185938	

- **Beam:**

UPPER BASEMENT & GROUND FLOOR:

Beam	Width(m)	Depth(m)	Span(m)	Unit Weight(kN/m3)	Load(kN)
B-to F 26	0.35	0.55	24	25	577.5
2 to 6 BF	0.35	0.55	24	25	577.5
Extended Beam	0.35	0.55	3	25	173.25

Total	1328.25

Secondary Beam Load					
Beam	Width(m)	Depth(m)	Span(m)	Unit Weight(kN/m3)	Load(kN)
B', C', D', E'	0.25	0.35	24	25	210
2',3',4',5'	0.25	0.35	24	25	210
Extended Beam	0.25	0.35	3	25	52.5
Deduction on Staircase	0.25	0.35	-9	25	-78.75
Deduction on Staircase	0.25	0.35	-6	25	-52.5
Total					341.25

FIRST, SECOND & THIRD FLOOR:

Beam		Width (m)	Depth (m)	Span (m)	Unit Weight (kN/m3)	Load(kN)
A.	**Primary**					
1	1CD	0.35	0.55	6		28.875
2	1DE	0.35	0.55	6		28.875
3	2BC	0.35	0.55	6		28.875
4	2DE	0.35	0.55	6		28.875
5	2EF	0.35	0.55	6		28.875
6	3AB	0.35	0.55	3		14.4375
7	3BC	0.35	0.55	6		28.875
8	3CD	0.35	0.55	6		28.875
9	3DE	0.35	0.55	6		28.875
10	3EF	0.35	0.55	6		28.875
11	3FG	0.35	0.55	3		14.4375
12	4AB	0.35	0.55	3		14.4375
13	4BC	0.35	0.55	6	25	28.875
14	4EF	0.35	0.55	6		28.875
15	4FG	0.35	0.55	3		14.4375
16	5AB	0.35	0.55	3		14.4375
17	5BC	0.35	0.55	6		28.875
18	5CD	0.35	0.55	6		28.875
19	5DE	0.35	0.55	6		28.875
20	5EF	0.35	0.55	6		28.875
21	5FG	0.35	0.55	3		14.4375
22	6BC	0.35	0.55	6		28.875
23	6CD	0.35	0.55	6		28.875
24	6DE	0.35	0.55	6		28.875
25	6EF	0.35	0.55	6		28.875

26	7CD	0.35	0.55	6		28.875
27	7DE	0.35	0.55	6		28.875
28	34A	0.35	0.55	6		28.875
29	45A	0.35	0.55	6		28.875
30	23B	0.35	0.55	6		28.875
31	34B	0.35	0.55	6		28.875
32	45B	0.35	0.55	6		28.875
33	56B	0.35	0.55	6		28.875
34	12C	0.35	0.55	3		14.4375
35	23C	0.35	0.55	6		28.875
36	34C	0.35	0.55	6		28.875
37	45C	0.35	0.55	6		28.875
38	56C	0.35	0.55	6		28.875
39	67C	0.35	0.55	3		14.4375
40	12D	0.35	0.55	3		14.4375
41	23D	0.35	0.55	6		28.875
42	56D	0.35	0.55	6		28.875
43	67D	0.35	0.55	3		14.4375
44	12E	0.35	0.55	3		14.4375
45	23E	0.35	0.55	6		28.875
46	34E	0.35	0.55	6		28.875
47	45E	0.35	0.55	6		28.875
48	56E	0.35	0.55	6		28.875
49	67E	0.35	0.55	3		14.4375
50	23F	0.35	0.55	6		28.875
51	34F	0.35	0.55	6		28.875
52	45F	0.35	0.55	6		28.875
53	56F	0.35	0.55	6		28.875
54	34G	0.35	0.55	6		28.875
55	45G	0.35	0.55	6		28.875
B.	Secondary				**Total**	**1414.875**
1	Between 2&3					
a	2'BC	0.2	0.3	6		9
b	2'DF	0.2	0.3	12		18
2	Between 3&4					
a	3'AC	0.2	0.3	9		13.5
b	3'EG	0.2	0.3	9		13.5
3	Between 4&5					
a	4'AC	0.2	0.3	9		13.5
b	4'EG	0.2	0.3	9	25	13.5
4	Between 5&6					
a	5'BF	0.2	0.3	24		36
5	Between B&C					
a	26B'	0.2	0.3	24		36
6	Between C&D					
a	57C'	0.2	0.3	9		13.5

7	Between D&E					
a	12D'	0.2	0.3	3		4.5
b	(2 to lift)D'	0.2	0.3	3.925		5.8875
c	57D'	0.2	0.3	9		13.5
8	Between E&F					
a	26E'	0.2	0.3	24		36
C.	Cantilever beam					
a	Inside 35CE	0.35	0.55	9.35		44.99688
D.	End Beams					
a	In void 35CE	0.2	0.3	29.3		43.95
					Grand Total	**1730.209**

FORTH:

Beam	Width (m)	Depth (m)	Span (m)	Unit Weight (kN/m3)	Load (kN)	Remarks
B-to F 26	0.35	0.55	24	25	577.5	X-spanning Beam
2 to 6 BF	0.35	0.55	24	25	577.5	Y-Spannin g
Extended Beam	0.35	0.55	3	25	173.25	12 beams in four sides
Beam‖curv e Axis	0.35	0.55	12	25	231	4 Beams in 4 Sides
Deduction	0.35	0.55	-12	25	-115.5	Beams on Void
Total					1443.75	

Secondary Beam Load						
Beam	Width (m)	Depth (m)	Span (m)	Unit Weight (kN/m3)	Load (kN)	Remarks
B', C', D', E'	0.25	0.35	24	25	210	Spanning along-X
2',3',4',5'	0.25	0.35	24	25	210	Spanning along-Y
Extended Beam	0.25	0.35	3	25	52.5	2 on each side
Deduction on Void	0.25	0.35	-7.325	25	- 64.0938	2 on each direction

Deduction on Staircase	0.25	0.35	-9	25	-78.75	Along X
Deduction on Staircase	0.25	0.35	-6	25	-52.5	Along Y
Total					277.1563	

TOP FLOOR:

Beam		Width(m)	Depth(m)	Span(m)	Unit weight (kN/m3)	Load(kN)
A.	**Primary**					
1	1CD	0.35	0.55	5.15	24	23.793
3	2BC	0.35	0.55	5.15	24	23.793
4	2DE	0.35	0.55	5.15	24	23.793
5	2EF	0.35	0.55	5.15	24	23.793
8	3CD	0.35	0.55	5.15	24	23.793
9	3DE	0.35	0.55	5.15	24	23.793
10	3EF	0.35	0.55	5.15	24	23.793
13	4BC	0.35	0.55	5.15	24	23.793
14	4EF	0.35	0.55	5.15	24	23.793
17	5BC	0.35	0.55	5.15	24	23.793
18	5CD	0.35	0.55	5.15	24	23.793
19	5DE	0.35	0.55	5.15	24	23.793
20	5EF	0.35	0.55	5.15	24	23.793
22	6BC	0.35	0.55	5.15	24	23.793
23	6CD	0.35	0.55	5.15	24	23.793
24	6DE	0.35	0.55	5.15	24	23.793
25	6EF	0.35	0.55	5.15	24	23.793
30	23B	0.35	0.55	5.15	24	23.793
31	34B	0.35	0.55	5.15	24	23.793
32	45B	0.35	0.55	5.15	24	23.793
33	56B	0.35	0.55	5.15	24	23.793
34	12C	0.35	0.55	2.15	24	9.933
35	23C	0.35	0.55	5.15	24	23.793
36	34C	0.35	0.55	5.15	24	23.793
37	45C	0.35	0.55	5.15	24	23.793
38	56C	0.35	0.55	5.15	24	23.793
39	67C	0.35	0.55	2.15	24	9.933
40	12D	0.35	0.55	2.15	24	9.933
41	23D	0.35	0.55	5.15	24	23.793
42	56D	0.35	0.55	5.15	24	23.793
45	23E	0.35	0.55	5.15	24	23.793
46	34E	0.35	0.55	5.15	24	23.793

47	45E	0.35	0.55	5.15	24	23.793
48	56E	0.35	0.55	5.15	24	23.793
50	23F	0.35	0.55	5.15	24	23.793
51	34F	0.35	0.55	5.15	24	23.793
52	45F	0.35	0.55	5.15	24	23.793
53	56F	0.35	0.55	5.15	24	23.793
54	4 C-D-E	0.35	0.55	10.3	24	47.586
55	D 3-4-5	0.35	0.55	10.3	24	47.586
56	Deduction (void)	0.35	0.55	-14.65	24	-67.683
57	(Curve part)				24	0
	3	0.35	0.55	6.66	24	30.7692
	4	0.35	0.55	9.66	24	44.6292
	5	0.35	0.55	6.66	24	30.7692
	D	0.35	0.55	4.83	24	22.3146
	E	0.35	0.55	6.66	24	30.7692
					Total	1049.2944

ROOF:

Beam	Width(m)	Depth (m)	Span (m)	Unit Weight (kN/m3)	Load(kN)	Remarks
B26, E26, F26	0.35	0.55	24	25	346.5	Horizontal Spanning Beams
C17	0.35	0.55	30	25	144.375	
D16	0.35	0.55	27	25	129.9375	
2,3,4,5,6FA	0.35	0.55	24	25	577.5	Vertically Spanning Beams
1DC	0.35	0.55	6	25	28.875	
Deduction	0.35	0.55	6	25	-57.75	Beam on Void
Total					1169.4375	

Secondary Beam Load						
Beam	Width (m)	Depth (m)	Span (m)	Unit Weight(kN/m3)	Load(kN)	Remarks
B', C', D'	0.25	0.35	72	25	157.5	Spanning Horizontally
C'	0.25	0.35	27	25	59.0625	
2',3',4', 5'	0.25	0.35	96	25	210	Spanning Vertically
Deduction	0.25	0.35	29.3	25	-64.0938	Beams on Void
Total					362.4688	

- **Wall Load:**

UPPER BASEMENT:

Walls	Length (m)	Width (m)	Height (m)	Unit Weight(kN/m3)	Load (kN/m)	Load (kN)
B35	9.7	0.11	1.7325	20	3.8115	36.97155
3CD Wall 270	1.525	0.27	1.7325	20	9.3555	14.2671375
3CD Wall 230	1.525	0.23	1.7325	20	7.9695	12.1534875
DE67	6.7	0.11	1.7325	20	3.8115	25.53705
C67	2.45	0.26	1.7325	20	9.009	22.07205
Total						111.001275

GROUND FLOOR:

Walls	Length(m)	Width(m)	Height(m)	Unit Weight (kN/m3)	Load (kN/m)	Load (kN)
110 Walls on BC56	24.218	0.11	1.7325	20	3.8115	92.306907
230 Wall on BC56	6.864	0.23	1.7325	20	7.9695	54.702648
D12 Wall 110	2.45	0.11	1.7325	20	3.8115	9.338175
C23 Wall 110	5.45	0.11	1.7325	20	3.8115	20.772675
3CD Wall 270	1.525	0.27	1.7325	20	9.3555	14.2671375
3CD Wall 230	1.525	0.23	1.7325	20	7.9695	12.1534875
D23 Wall 110	3.76	0.11	1.7325	20	3.8115	14.33124
CD56 Wall 230	1.257	0.23	1.7325	20	7.9695	10.0176615
CD56 Wall 110	4.04	0.11	1.7325	20	3.8115	15.39846
CD Wall 110	5.55	0.11	1.7325	20	3.8115	21.153825
AB34 Wall 110	7.2	0.11	1.7325	20	3.8115	27.4428
AB45 Wall 110	4.95	0.11	1.7325	20	3.8115	18.866925
BC45 Wall 110	1.965	0.11	1.7325	20	3.8115	7.4895975
CD67 Wall 110	4.915	0.11	1.7325	20	3.8115	18.7335225
Total						336.9750615

FIRST FLOOR & SECOND FLOOR & THIRD FLOOR:

<table>
<tr><th>S. N</th><th>Walls</th><th>Length (m)</th><th>Breadth (m)</th><th>Height (m)</th><th>Unit Weight (kN/m3)</th><th>Load (kN)</th><th>Load(kN/m or kN/m2)</th></tr>
<tr><td>1</td><td colspan="4">Grid 56BC</td><td rowspan="5">20</td><td colspan="2"></td></tr>
<tr><td>A</td><td>Slab 56BC</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>a.</td><td>1 brick wall</td><td>9.334</td><td>0.23</td><td>3.325</td><td>142.763 5</td><td rowspan="3">6.8319</td></tr>
<tr><td>b.</td><td>1/2 brick wall</td><td>17.327</td><td>0.11</td><td>3.325</td><td>126.747</td></tr>
<tr><td></td><td>Deduct 1-1.1m door</td><td>1.1</td><td>0.23</td><td>2.1</td><td>10.626</td></tr>
</table>

	Deduct 4-0.7m doors	2.8	0.11	2.1		12.936	
B	Beam 5BC						
a.	1/2 brick wall	2.455	0.11	3.325		17.9583 3	2.9930
C	Beam 56B						
a.	1/2 brick wall	2	0.11	3.325		14.63	2.4383
2	Grid 67CD						
A	Slab 67CD						
a	1/2 brick wall	3.602	0.11	3.325		26.348	2.9276
B	Beam 6CD						
a	1/2 brick wall	1.257	0.11	3.325		9.1949	3.0649
3	Grid 56CD						
A	Beam 5CD						
a	1/2 brick wall	1.257	0.11	3.325		9.1949	3.0649
4	Grid 45EF						
A	Beam 45E						
a	1 brick wall	1.06	0.23	3.325		16.2127	8.4570
b	1/2 brick wall	1.252	0.11	3.325		9.1583	
B	Beam 5EF						
a	1/2 brick wall	2.875	0.11	3.325		21.0306	7.0102
C	Slab 45EF						
a	1/2 brick wall	5.312	0.11	3.325		38.8572	3.9581
	Deduct 1-0.7m door	0.7	0.11	2.1		3.234	
5	Grid 45FG						
A	Beam 5FG						
a	1/2 brick wall	2.615	0.11	3.325		19.1287 3	6.3762
B	Beam 45G						
a	1/2 brick wall	1.257	0.11	3.325		9.1949	3.064
C	Slab 45FG						
a	1/2 brick wall	4.022	0.11	3.325		29.4209	2.9096
	Deduct 1-0.7m door	0.7	0.11	2.1		3.234	
6	Grid 23CD						
A	Beam 3CD						
a	1 brick wall	3.05	0.23	3.325		46.6497	7.7749
B	Beam 23D						
a	1/2 brick wall	4.11	0.11	3.325		30.0646	5.0107
C	Beam 23C						
a	1/2 brick wall	5.45	0.11	3.325		39.8667	6.6444
7	Grid 12CD						
A	Beam 12D						

a	1/2 brick wall	2.45	0.11	3.325		17.9215	5.9739
8	Outer Perimeter						
a	100 mm Glass	120	0.1	3.465	0.88 kN/m2	365.904	3.0492
	TOTAL						1020.278

FORTH FLOOR:

TOP

Walls	Length (m)	Width (m)	Height (m)	Unit Weight (kN/m3)	Load (kN/m)	Load (kN)
110 Walls on BC56	24.218	0.11	1.7325	20	3.8115	92.3069
230 Wall on BC56	6.864	0.23	1.7325	20	7.9695	54.70264
D12 Wall 110	2.45	0.11	1.7325	20	3.8115	9.338175
C23 Wall 110	5.45	0.11	1.7325	20	3.8115	20.77267
3CD Wall 270	1.525	0.27	1.7325	20	9.3555	14.26713
3CD Wall 230	1.525	0.23	1.7325	20	7.9695	12.15348
D23 Wall 110	3.76	0.11	1.7325	20	3.8115	14.33124
CD56 Wall 230	1.257	0.23	1.7325	20	7.9695	10.01766
CD56 Wall 110	4.04	0.11	1.7325	20	3.8115	15.39846
CD Wall 110	5.55	0.11	1.7325	20	3.8115	21.15382
DE 1	1.26	0.11	1.7325	20	3.8115	4.80249
E 12	2.59	0.11	1.7325	20	3.8115	9.871785
DE 2	1.914	0.11	1.7325	20	3.8115	7.295211
DE 12	1.517	0.11	1.7325	20	3.8115	5.782045
DE 7	1.26	0.11	1.7325	20	3.8115	4.80249
E 67	2.59	0.11	1.7325	20	3.8115	9.87175
DE 6	1.914	0.11	1.7325	20	3.8115	7.29521
DE 67	1.517	0.11	1.7325	20	3.8115	5.78205
E 56	1.99	0.11	1.7325	20	3.8115	7.58485
EF 6	1.64	0.11	1.7325	20	3.8115	6.25086
EF 56	3.614	0.11	1.7325	20	3.8115	13.774
DE 5	1.26	0.11	1.7325	20	3.8115	4.80249
E 45	2.59	0.11	1.7325	20	3.8115	9.871785
DE 4	1.914	0.11	1.7325	20	3.8115	7.295211
DE 45	1.517	0.11	1.7325	20	3.8115	5.78204
Total						375.3073

FLOOR:

Wall(brick)	Length (m)	Width (m)	Height (m)	unit weight (kN/m3)	Load (kN/m3)	Load (kN)
110 Walls B-C 5-6	25.326	0.11	3.325	20	7.315	185.2597

230Wall B-C 5-6	8.18	0.23	3.325	20	15.295	125.113 1
Deduction (door)	-4.5	0.11	2.1	20	4.62	-20.79
Deduction (door)	-1.2	0.23	2.1	20	9.66	-11.592
Wall 110 C 2-3	5.45	0.11	3.325	20	7.315	39.8667 5
Wall 270 3 C-D	1.525	0.27	3.325	20	17.955	27.3813 8
Wall 230 3 C-D	1.525	0.23	3.325	20	15.295	23.3248 8
Wall 110 D 2-3	3.76	0.11	3.325	20	7.315	27.5044
Wall 230 C-D 5-6	1.257	0.23	3.325	20	15.295	19.2258 2
Wall 230 C-D 6-7	2.322	0.23	3.325	20	15.295	35.5149 9
Wall 110 C-D 6-7	0.933	0.11	3.325	20	7.315	6.82489 5
E 4-5	1.06	0.27	3.325	20	17.955	19.0323
					total	476.666 2

ROOF:

Walls	Length (m)	Width(m)	Height(m)	Unit Weight(kN/m3)	Load (kN/m)	Load (kN)
230 Parapet wall	49.308	0.23	1.75	20	8.05	396.9294
230 Truss Supporting wall	44.76	0.23	1.5	20	6.9	308.844
	Wall Load From Top Floor=					375.31
Total						1081.083 4

- **Truss:**

Truss Load			
The Truss Load was calculated as per Duggal (Limit State Design of Steel Structure)			
Information about Truss			
Span =	7m	approx.	
Angle	40deg		

Self-Weight truss	=(span/3+5)*10 N/m^2	73.333333	
Self-Weight Bracing	N/m^2	15	As per Duggal
Total truss Area=	$24*24m^2$	576	
Roofing Material (Glazing)	N/m^2	250	***Hz weight=Sloping weight (Assumed) For conservative design***
Live Load(access not provided)	750-20(40-10)N/m^2	150	
Total Load	N/m^2	338.333333	
Total Load	kN	194.88	

- ## Staircase:

Staircase	Length(m)
1st Flight	1.712
1st Landing	0.6
2nd Fight	2.054
2nd Landing	2.421
3rd Flight	1.753
3rd Landing	0.6
4th Flight	1.816
Total Length	**10.956**

Width (m)	2.25
Thickness (m)	0.14
Volume Waist Slab	3.45114
Volume Of Each Rise-Tread	0.0556875
Number Of Rise-Tread	21
Total Rise Tread Vol(m^3)	1.1694375
Total Staircase Vol(m^3)	4.6205775
Unit Weight(kN/m^3)	25

Total Weight(kN)	115.514438

- **Lumped Mass:**

Floor ⇨	Upper Basement	Ground	First	Second	Third	Forth	Top	Roof
Mass (kN)	8030.5308	8474.2	7700.4822	7263.8572	7248.8830	8126.3825	6924.0038	5856.6144

5 STRUCTURAL ANALYSIS

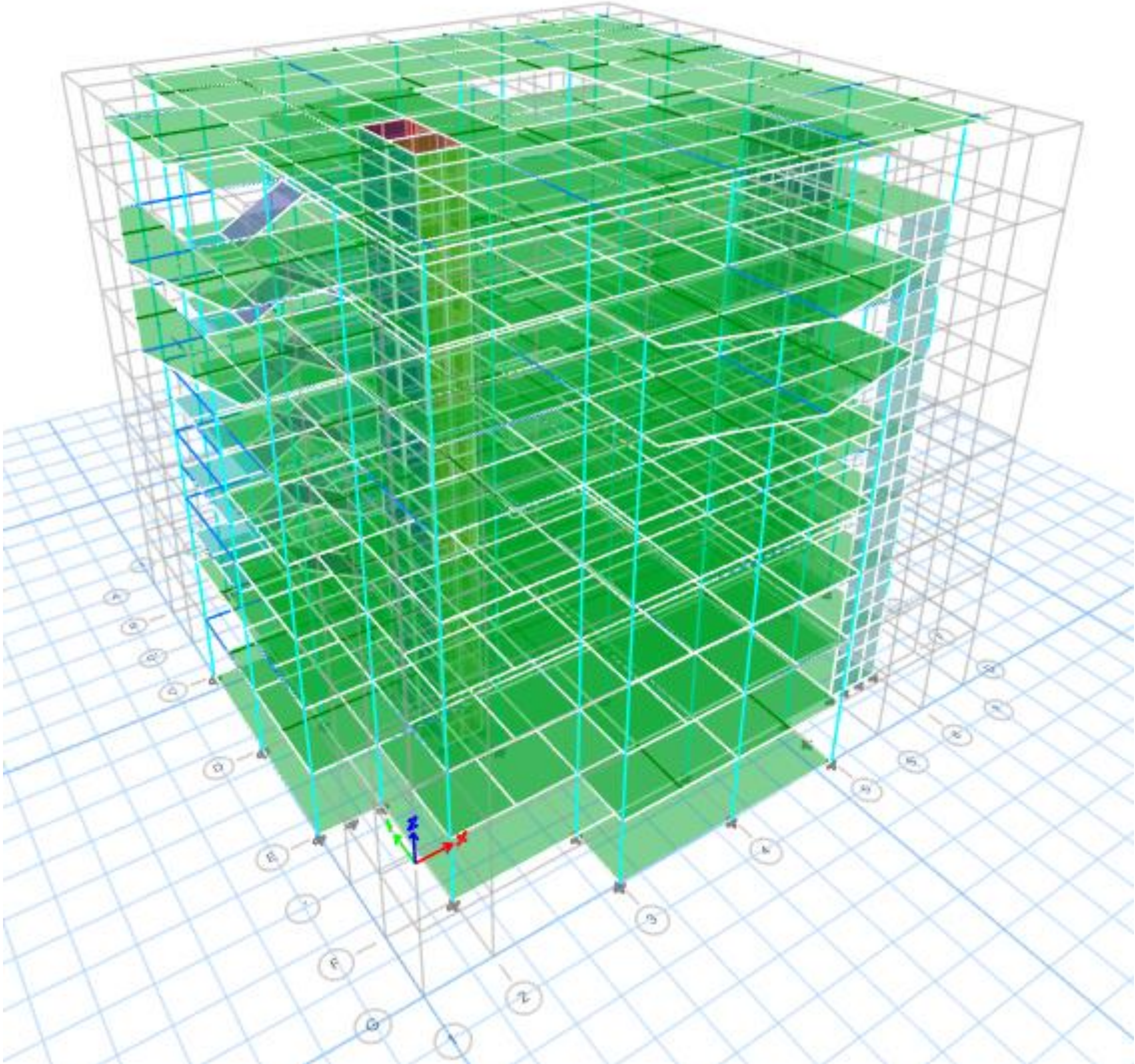


Figure 4 Building Model in ETABS

- ## Centre of mass and Centre of stiffness:

Centre of mass of each storey was calculated manually, along with the center of rigidity for the preliminary architectural plan. Additionally, these values were generated from the ETABS model, and compared. Eccentricity thus observed was used in determining changes, to structural configuration.

During an earthquake, the seismic forces that act throughout the body of structure can equivalently be analyzed as acting about the center of mass of each floor. This equivalence holds true only when the floor acts as a rigid body. Reinforced concrete slabs have very high in-plane rigidity and hence this condition is fulfilled. In response to the seismic forces, restoring forces are generated due to the stiffness of columns, and shear walls against relative drifts at their ends. These restoring forces acting at each column to slab and shear wall to slab connection can be equivalently analyzed as acting through the center of rigidity.

Any difference in location of these centers i.e. center of mass and center of rigidity, creates a torque that rotates the floor. The difference in the position of the centers is called eccentricity. And the effect seen in case of pronounced eccentricity is called Torsional irregularity.

Eccentricity can't be completely eliminated from the structure. And in design, an extra amount of eccentricity is added to calculated eccentricity to take into account minor oversight and approximations in assessing mass and stiffness distribution, as well as to take into account the unknown variations in configuration of live load, furniture and such.

Comparison of Results obtained from Manual Calculation and ETABS

Eccentricity Before addition of the counter-shear wall:

Table 2: Manual calculation of eccentricities

Floor Level	Centre of Mass		Centre of Stiffness		Eccentricity		Eccentricity	
	x (m)	y (m)	x(m)	y(m)	e_x (m)	e_y (m)	e_x(%)	e_y(%)
Roof Level	14.30671	15.2415	10.97773	15.13719	3.32898	0.104314	11.097	0.348
Top Floor	15.34241	15.22732	11.83156	14.8016	3.51085	0.425722	11.703	1.419
4th floor	15.21595	15.1725	11.83156	14.8016	3.38439	0.370902	11.281	1.236
3rd floor	15.21057	15.1894	11.83156	14.8016	3.37901	0.387802	11.263	1.293
2nd floor	15.25593	15.2137	11.83156	14.8016	3.42437	0.412102	11.415	1.374
1st floor	15.19438	15.42083	11.83156	14.8016	3.36282	0.619232	11.209	2.064

Table 3:Eccentricities obtained from ETABS model analysis

Floor Level	Centre of Mass		Centre of Stiffness		Eccentricity		Eccentricity	
	x (m)	y (m)	x(m)	y(m)	e_x (m)	e_y (m)	e_x(%)	e_y(%)
Roof Level	14.3235	15.118	11.2683	14.7931	3.0552	0.3249	10.184	1.083
Top Floor	15.5756	15.1864	11.1115	14.7271	4.4641	0.4593	14.88	1.531
4th floor	15.6299	14.7855	10.9349	14.6843	4.695	0.1012	15.65	0.337
3rd floor	16.0774	14.7599	10.8962	14.6421	5.1812	0.1178	17.271	0.393
2nd floor	16.0305	15.0899	11.1889	14.5975	4.8416	0.4924	16.139	1.641
1st floor	16.0145	14.8422	12.2616	14.6036	3.7529	0.2386	12.51	0.795

Table 4: Comparison of Manual and ETABS eccentricities calculation

Floor Level	Eccentricity from Manual Calculation		Eccentricity from ETABS		Difference		Difference	
	e_x (m)	e_y (m)	e_x (m)	e_y (m)	e_x (m)	e_y (m)	e_x(%)	e_y(%)
Roof Level	3.32898	0.104314	3.0552	0.3249	0.27378	-0.22059	0.913	-0.735
Top Floor	3.51085	0.425722	4.4641	0.4593	-0.95325	-0.03358	-3.178	-0.112
4th floor	3.38439	0.370902	4.695	0.1012	-1.31061	0.269702	-4.369	0.899
3rd floor	3.37901	0.387802	5.1812	0.1178	-1.80219	0.270002	-6.007	0.9
2nd floor	3.42437	0.412102	4.8416	0.4924	-1.41723	-0.0803	-4.724	-0.268
1st floor	3.36282	0.619232	3.7529	0.2386	-0.39008	0.380632	-1.3	1.269

Table 3 shows that the eccentricities obtained from manual calculations and ETABS analysis are nearly equal with maximum discrepancy of 6.0%.

From Table-1 and Table-2, it is clear that there is **high eccentricity** in the current model.

Hence the addition of shear wall to reduce thus obtained eccentricity is essential. Therefore, shear walls of length 3m are added at, right halves of grids **4-5-A & 4-5-G**, upper half of grid **7-C-D** and the lower half of grid **7-D-E** upto the top floor level.

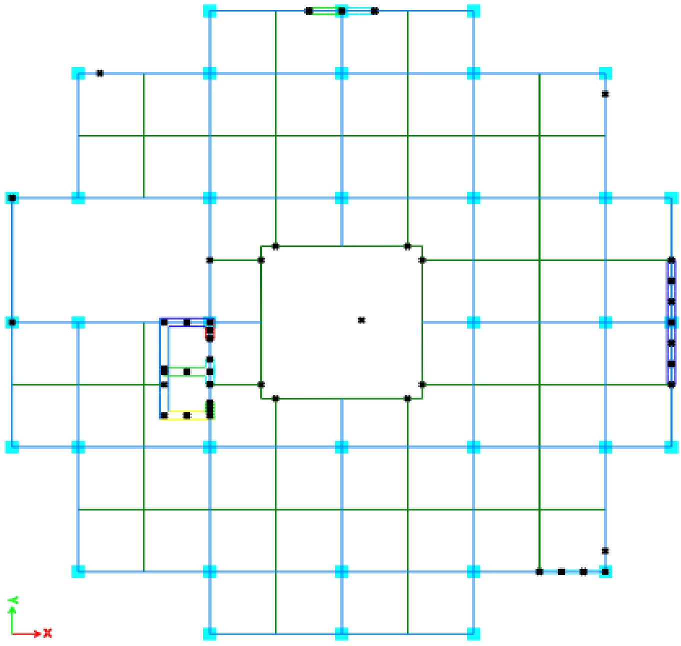


Figure 5 : Placing of Counter Shear Wall

Table 5: Eccentricities after addition of shear walls

Floor Level	Centre of Mass		Centre of Stiffness		Eccentricity		Eccentricity	
	x (m)	y (m)	x(m)	y(m)	e_x (m)	e_y (m)	e_x(%)	e_y(%)
Roof Floor	14.221	15.081	13.854	14.699	0.367	0.382	1.223	1.273
Top Floor	15.493	15.127	14.265	14.656	1.228	0.471	4.093	1.57
4th floor	15.648	14.744	14.418	14.667	1.23	0.077	4.1	0.257
3rd floor	16.075	14.716	14.705	14.665	1.37	0.051	4.567	0.17
2nd floor	16.03	15.031	15.263	14.611	0.767	0.42	2.557	1.4
1st floor	16.036	14.795	16.061	14.512	-0.025	0.283	-0.083	0.943

From the tabulated value for the eccentricities obtained after addition of shear walls, it is observed that the eccentricity which before the addition of shear wall ranging above 11% on x-direction has now been drastically reduced to less than 5% on all the floor even reaching nearly zero at the first floor.

- **Base Shear Comparison**

According to **IS 1893 (Part 1):2016 Cl. 6.4.2**:

The design horizontal seismic coefficient Ah for a structure shall be determined by the following expression

$$Ah = \frac{\left(\frac{Z}{2}\right) x \left(\frac{Sa}{g}\right)}{\frac{R}{I}}$$

Where,

Z = Zone factor given by IS 1893 (Part I): 2016 Table 3

I = Importance Factor

R = Response reduction factor given by IS 2016 (Part I): 2016

S_a/g = Average response acceleration coefficient which depends on approximate fundamental natural period of vibration (Ta).

The base shear is calculated manually and from ETABS. Manual Calculation was as per follows: For the building in this study, following data were adopted:

Table 6: Parameter adopted for base shear calculation

Z=	0.36	Zone V
I	1.5	
R	5	SMRF with structural wall
Site Condition	Medium Soil	

Table 7: Base shear calculation

Direction	T	Sa/g	Ah=(z/2)*(Sa/g)*(I/R)	Base Shear =Ah*W	From ETABS	Errors (%)
X	1.07	1.2710	0.068636	3541.416	3752.278	5.6195
Y	0.96	1.4166	0.0765	3947.203	4131.983	4.4719

The Base shears from manual Calculation and from ETABS are approximately similar. The minor errors may due to **cantilever slab and beam** on top and roof floors which weren't considered during manual calculation of seismic weight.

- ## Time Period Calculation

The time period in ETABS model and in manual calculation was determined as per IS 1893:2016 **clause 7.6.2 (b)**

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$$

$$A_w = \sum_{i=1}^{N_w} [A_{wt}\{0.2 + (\frac{L_{wi}}{h})^2\}]$$

Table 8: Time period calculation in X- direction

		X-direction				
		h	20.79			
Wall no	x-dim	y-dim	A_{wi}	L_{wi}	L_{wi}/h	A_w
a	1.600	0.200	0.32	1.600	0.07696	0.065895
b	1.600	0.200	0.32	1.600	0.07696	0.065895
c	1.600	0.200	0.32	1.600	0.07696	0.065895
c wall 1	3.000	0.200	0.6	3.000	0.1443	0.132494
c wall 2	3.000	0.200	0.6	3.000	0.1443	0.132494
Aw						0.462673
T					1.073532	

Table 9: Time period calculation in Y-direction

		Y-direction				
		h	20.79			
Wall no	x-dim	y-dim	A_{wi}	L_{wi}	L_{wi}/h	A_w
d	0.200	4.400	0.88	4.400	0.21164	0.215417
e	0.200	0.700	0.14	0.700	0.03367	0.028159
f	0.200	1.200	0.24	1.200	0.05772	0.0488

g	0.200	0.500	0.1	0.500	0.02405	0.020058
c wall 3	0.200	3.000	0.6	3.000	0.1443	0.132494
c wall 4	0.200	3.000	0.6	3.000	0.1443	0.132494
Aw						0.57742
T						0.960962

- **Check for Mass Participation Ratio**

The building is modeled without basement and the results for modal mass participation ratio are as follows:

Table 10 Modal Mass participation Factors

Model	**Period sec**	**U_X**	**U_Y**	**SumU_X**	**SumU_Y**	**R_Z**	**SumR_Z**
1	0.616	0.7313	2.18E-05	0.7313	2.18E-05	0.0017	0.0017
2	0.529	0	0.713	0.7313	0.7131	0.025	0.0267
3	0.472	0.0017	0.0272	0.733	0.7402	0.7103	0.737
4	0.164	0.1472	0.0017	0.8801	0.7419	0.0014	0.7384
5	0.15	0.0045	0.1064	0.8846	0.8483	0.0247	0.7631
6	0.134	0.0001	0.0434	0.8847	0.8917	0.0997	0.8628
7	0.084	0.0095	0.0234	0.8942	0.9151	0.04	0.9028
8	0.078	0.0504	0.0055	0.9447	0.9206	0.0062	0.909
9	0.068	0.0003	0.0282	0.9449	0.9488	0.0247	0.9336
10	0.049	0.0294	0.0014	0.9744	0.9502	0.0027	0.9363
11	0.047	0.0021	0.0262	0.9765	0.9764	0.0073	0.9435
12	0.042	0.0009	0.0019	0.9774	0.9783	0.0306	0.9741

- ## Irregularity Check

Irregularities must be eliminated as far as possible to ensure better performance of buildings during seismic events. In our building some irregularities were not avoidable, but since we have used Modal Response Spectrum Methods, the effects of those irregularities have been considered and thus their presence is acceptable. Check for plan irregularities and vertical irregularities were done as per IS 1893 Part 1(2016).

5..1 Vertical Irregularity

a. Stiffness Irregularity (Soft Storey):

A soft storey is the one whose lateral stiffness is less than that of the storey above. Soft Storey check was done using lateral stiffness obtained from ETABS, and our building conformed to this check.

Table 11: Soft storey check

Storey	Soft Storey Check in X direction		Soft Storey Check in Y direction	
	Lateral Stiffness (kN/m)	Check (K_i>K_{i+1})	Lateral Stiffness (kN/m)	Check (K_i>K_{i+1})
Roof Floor	169834		313736	
Top Floor	316322	**Regular**	634520	**Regular**
4th floor	408943	**Regular**	825156	**Regular**
3rd floor	475198	**Regular**	977983	**Regular**
2nd floor	551005	**Regular**	1154996	**Regular**
1st floor	683253	**Regular**	1436240	**Regular**
Ground floor	1042421	**Regular**	2198460	**Regular**

b. Mass Irregularity:

When the seismic weight of any floor is more than 150% of that of the floors below, then mass irregularity is considered to exist. A roof that is lighter than the floor below need not be considered.

Table 12: Mass irregularity check

Storey	Mass irregularity check for X and Y direction		
	Mass	M_i/M_{i-1}	Check (<1.5)
Roof Floor	671808.13	-	-
Top Floor	865134.56	0.78	**Regular**
4th floor	884964.76	0.98	**Regular**
3rd floor	807576.48	1.10	**Regular**
2nd floor	820255.57	0.98	**Regular**
1st floor	823609.82	1.00	**Regular**
Ground floor	904513.98	0.91	**Regular**

c. **Vertical Geometric Irregularity:**

Vertical geometric irregularity shall be considered to exist if the horizontal dimension of lateral force resisting system in any Storey is more than 125% of the storey below.

Table 13: Vertical Geometric Irregularity check

Storey	Vertical Geometric Irregularity Check		
	Horizontal Dimension (mm)	D_i/D_{i-1}	Check (<1.25)
Roof Floor	28528	0.82	**Regular**
Top Floor	34786	1.00	**Regular**
4th floor	34786	1.16	**Regular**
3rd floor	30000	1.00	**Regular**
2nd floor	30000	1.00	**Regular**
1st floor	30000		

d. **In-Plane Discontinuity in Vertical Elements Resisting Lateral Force**:

It is considered to exist when the in-plane offset of the lateral force resisting system is greater than 20% of the plan length of those elements.

Since the columns and shear walls are continuous without in-plane offset throughout the vertical dimension, the building conforms to this check.

e. **Strength Irregularity (Weak Storey):**

A weak storey is a storey whose lateral strength is less than that of storey above. Since all columns and shear walls extend to the top floor in each block, the lateral force resisting system is same in each floor. Moreover, the design forces on column and shear wall are always greater on lower floor than in upper floors, and since strength is provided as required by design forces, the designed strength is also greater in lower stories compared to upper ones. Thus, weak Storey doesn't exist in our buildings, and no further elaborate checks are performed.

f. **Floating or Stub Columns:**

The columns are continuous throughout the vertical dimension and hence conform to this check.

g. **Irregular Modes of Oscillation in Two Principal Plan Directions**:

A building is said to have lateral storey irregularity in a principal plan direction if:

a. the first three modes together contribute less than 65% mass participation factor in each principal plan direction, and
b. the fundamental lateral natural periods of the building in the two principal plan directions are away from each other by at least 10% of the larger value.

Both cases (a) and (b) are passed so as to confirm the lateral storey regularity of our building.

5..2 Plan Irregularity

a. Torsional Irregularity:

Torsion irregularity is considered to exist where the maximum horizontal displacement of any floor in the direction of the lateral force (applied at the centre of mass) at one end of the Storey is more than 1.5 times its minimum horizontal displacement at the far end of the same Storey in that direction. Moreover, the natural period corresponding to fundamental torsional mode of oscillation being more than those of first two translational modes along each principal plan direction also indicates torsional irregularity.

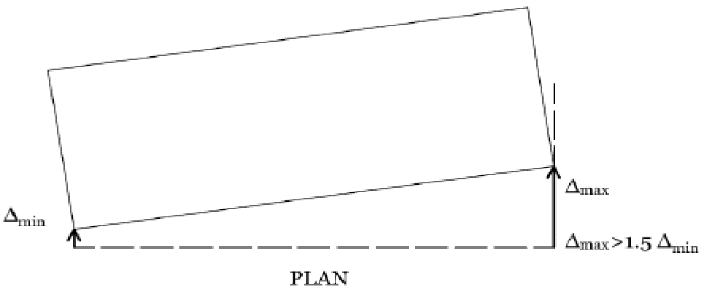


Figure 6: Torsional Irregularity

Table 14: Torsional Irregularity check X- direction

Storey	Torsional Irregularity Check Along X direction			
	Minimum displacement(Δ_{min})	Maximum Displacement(Δ_{max})	(Δ_{max})/(Δ_{min})	Check (<1.5)
Roof Floor	49.940	47.013	0.94	**Regular**
Top Floor	43.309	39.452	0.91	**Regular**
4th floor	35.584	32.407	0.91	**Regular**
3rd floor	27.492	25.358	0.92	**Regular**
2nd floor	19.741	18.200	0.92	**Regular**
1st floor	12.541	11.549	0.92	**Regular**
Ground floor	6.413	5.888	0.92	**Regular**

Table 15: Torsional Irregularity Check Along Y direction

Storey	Torsional Irregularity Check Along Y direction			
	Minimum displacement(Δ_{min})	Maximum Displacement(Δ_{max})	$(\Delta_{max})/(\Delta_{min})$	Check (<1.5)
Roof Floor	22.648	25.446	1.12	**Regular**
Top Floor	19.347	21.139	1.09	**Regular**
4th floor	15.806	17.168	1.09	**Regular**
3rd floor	12.313	13.120	1.07	**Regular**
2nd floor	8.860	9.310	1.05	**Regular**
1st floor	5.711	5.848	1.02	**Regular**
Ground floor	3.045	2.966	0.97	**Regular**

Also, the natural periods for first two translational modes (1.205 sec and 0.849 sec) are greater than the natural period for torsional mode of oscillation (0.795 sec).

Hence, the building conforms to this check.

b. Re-entrant Corners:

A structure is said to have re-entrant corner in a direction if its structural configuration has a projection of greater than 15% of its overall dimension in that direction.

As no such condition exists, the building conform to this check.

c. Floor Slabs having Excessive Cut-Outs or Openings:

Excessive openings in slabs result in flexible diaphragm behavior, and hence lateral force is not shared in proportion to the lateral translational stiffness of frames/columns. A building is said to have discontinuity in their in-plane stiffness when floor slabs have cut-outs or openings of area more than 50% of full area of the floor slab.

Table 16: Flexible Diaphragm Check

Flexible Diaphragm Check				
Storey	Storey Area(A) m^2	Opening Area (A_o) m^2	A_o/A (%)	Check (≤50%)
Roof Floor	594	53.839	9.06	**Regular**
Top Floor	798.864	99.125	12.41	**Regular**
4th floor	798.864	99.125	12.41	**Regular**
3rd floor	720	99.125	13.77	**Regular**
2nd floor	720	99.125	13.77	**Regular**
1st floor	720	99.125	13.77	**Regular**

d. Out of Plane Offsets in Vertical Elements:

Out of Plane offset irregularity is said to exist where there is a discontinuity in a lateral force resisting path i.e. when structural walls or frames are moved out of plane in any storey along the height of the building.

Since no such offsets are significant in our building, the building conforms to this check.

e. Non- Parallel Lateral Force System:

Non-parallel systems do not have the vertically oriented structural systems resisting lateral forces oriented along the two principal orthogonal axes in plan.

Since no such conditions prevail in our building, the building conforms to this check.

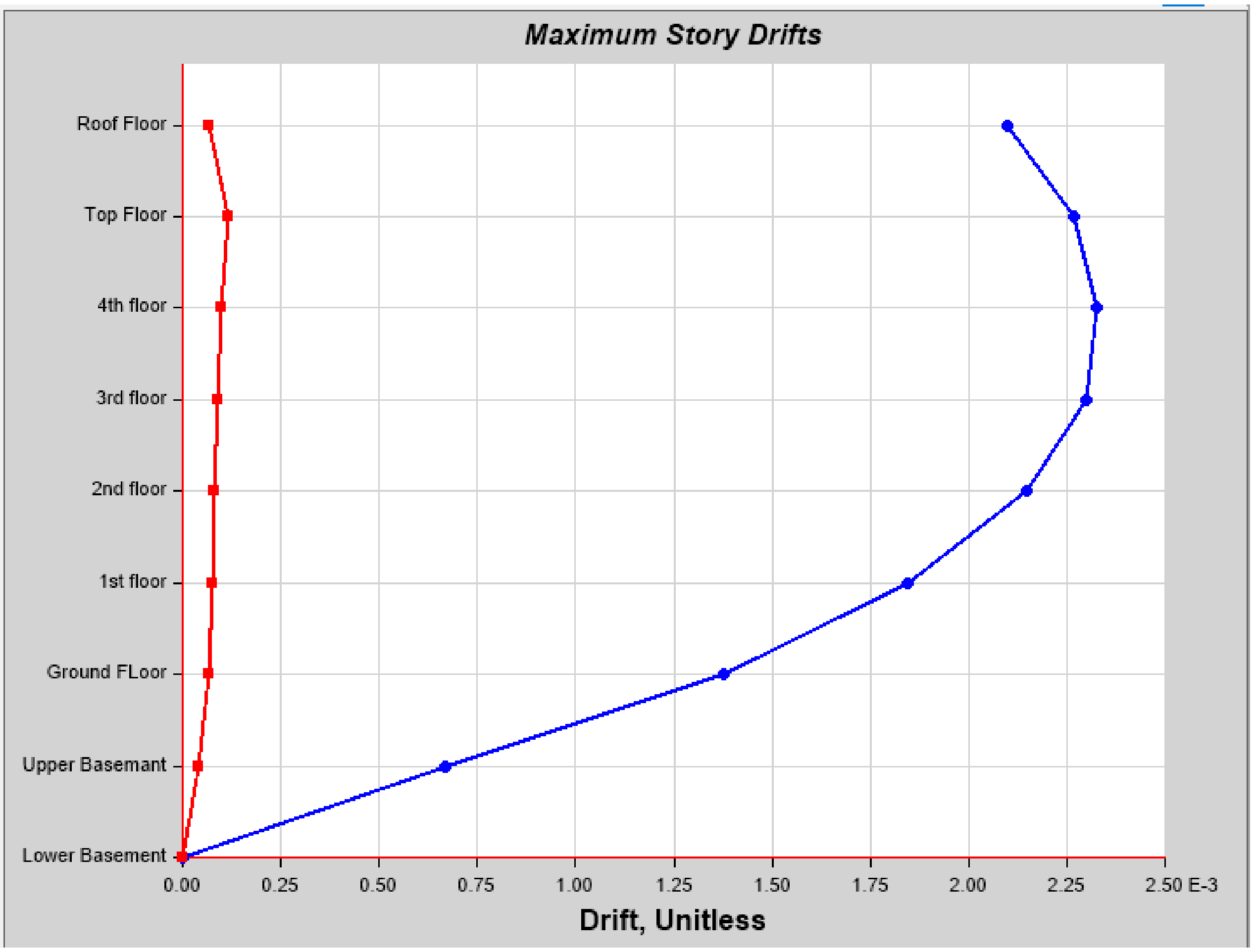
- ## Storey Drift

Storey drift is the relative displacement between the floors above and/or below the storey under consideration. As per **IS 1893:2016** Storey drift in any Storey shall not exceed 0.004 times the storey height. The limitation on storey drift is necessary to avoid discomfort to occupants of the building and to save non-structural elements from damage. ETABS analysis directly generates the drift in form ratio so that the result can be directly compared with the permissible drift ratio of **0.004**.

Table 17: Storey Drift Ratio

Storey	Elevation (m)	X-Direction	<0.004	Y-Direction	<0.004
Roof Floor	27.72	0.002098	**OK**	0.001339	**OK**
Top Floor	24.255	0.002268	**OK**	0.001222	**OK**
4th floor	20.79	0.002328	**OK**	0.001216	**OK**
3rd floor	17.325	0.002298	**OK**	0.001177	**OK**
2nd floor	13.86	0.002146	**OK**	0.001078	**OK**
1st floor	10.395	0.001846	**OK**	0.000912	**OK**
Ground Floor	6.93	0.001378	**OK**	0.000676	**OK**
Upper Basement	3.465	0.000669	**OK**	0.000351	**OK**
Lower Basement	0	0	**OK**	0	**OK**

Figure 7: Storey Drift Ratio for EQx in ETABS



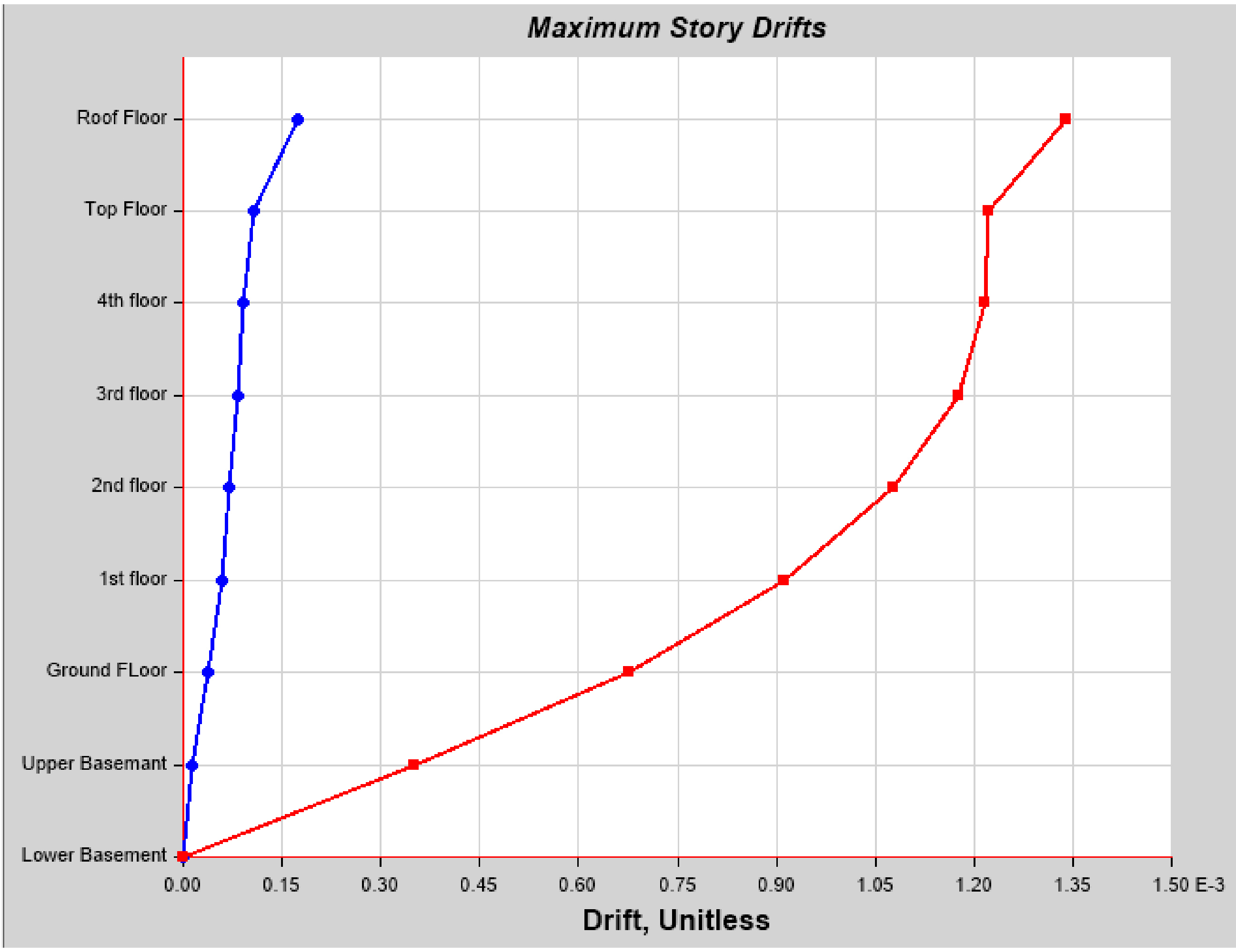


Figure 8: Storey Drift Ratio for EQy in ETABS

6 DESIGN AND DETAILING

• Design of Slab:

Slabs are plate elements forming floors and roof of a building and carrying distributed load primarily by flexure. A slab may be supported by beams or wall and may be used as the flange of a T or L beam. Moreover, a slab may be simply supported or continuous over one or more supports and classified accordingly.

a) One-way slab spanning in one direction – Length is more than twice the breadth.
b) Two-way slab spanning on both direction
c) Circular slabs
d) Flat slabs (Resting directly on column)
e) Grid floor and Ribbed slab

Slab is an important structural element which rests on a beam and supports various dead loads (self-weight, floor finishing) and live loads. The main function of slab is to transfer gravity loads to the beams either in one way or in two-way action. The slab is considered two ways when the ratio of longer side to smaller side does not exceed 2.

In one-way slabs, the slab deflects in shorter direction alone in a cylindrical fashion, hence main reinforcement is provided in shorter direction. It is because substantial bending moment develops on only one direction (i.e. shorter direction) as the utmost load will be transmitted by the larger side. However, for two slabs, the contribution of longer span in carrying load becomes substantial. The load is transmitted in two directions in two-way slab, which when loaded will deflect into a dished surface.

IS 456:2000 Clause 24 gives the provision for slab design. Design forces are calculated according to type and extent of loadings. The moments developed depends upon the edge's conditions. IS 456(Annex D, Table 26 and 27) provides moment coefficient to find positive and negative moments developed in slabs with various edges conditions.

Slab are designed using same theories of beam i.e. theories of bending and shear. Following are the method that can be used for analysis of slab:

a) Elastic analysis (Idealization on strip as Beam)
b) Semi-empirical coefficients (As given in Code)
c) Yield line theory

Reinforcement provided is least in slab among three structural members: slab, beam and column. A slab can be distinguished from a beam as follows:

a) Minimum span of slab should not be less than four times the overall depth and slab are much thinner than beam.
b) Slab are analyzed and designed as having unit width i.e. 1 m.
c) Compression reinforcement is used only in exceptional case.
d) Shear stress is very low in slab hence shear reinforcement is not provided. If needed, depth is increased rather than providing shear reinforcement.

✓ **Design of two-way slab**

❖ **Slab BC'5'6**

Step 1: Depth or Thickness Calculation

Overall Depth of Slab, D = 140 mm

Clear Long Span = 3 - $\frac{0.2}{2} - \frac{0.35}{2}$ = 2.725 m

Clear Short Span = 3 - $\frac{0.2}{2} - \frac{0.35}{2}$ = 2.725 m

Diameter of the bar, ϕ = 12 mm

Considering bars in two layers

Effective depth, d = 140 − 15 −$\frac{12}{2}$ = 119 mm

Step 2: Effective Span

Along X-X axis:

• c/c of supports = 3 m

• clear span + effective depth = 2.725 + 0.119 = 2.844 m

L_x = 2.844 m

Along Y-Y axis:

• c/c of supports = 3 m

• clear span + effective depth = 2.725 + 0.119 = 2.844 m

L_y = 2.844 m

L_x/L_y = 1 < 2 i.e. two-way slab

Step 3: Load Calculation

Load definition	Value
Live Load	4 kN/m^2
Floor Finish	1 kN/m^2
Self-weight	3.5 kN/m^2
Dead load of wall	8.2 kN/m^2
Total Load	16.7 kN/m^2
Factored load	1.5* 16.5 =25.05 kN/m^2

Step 4: Bending Moment

$$\frac{Ly}{Lx} = \frac{2.844}{2.844} = 1$$

& Interior Panel

$\alpha_x^- = 0.032$	$\alpha_x^+ = 0.024$
$\alpha_y^- = 0.032$	$\alpha_y^+ = 0.024$

$M_x+ = \alpha_x^+ * w * L_x^2$ = 4.86 kN-m	$M_x- = \alpha_x^- * w * L_x^2$ = 6.48 kN-m
$M_y+ = \alpha_y^+ * w * L_x^2$ = 4.86 kN-m	$My- = \alpha_y^- * w * L_x^2$ = 6.48 kN-m

$X_{u,lim}$ = 0.46d = 0.46*119 = 54.74 mm

$M_{u,lim} = 0.36*f_{ck}*b_{xu,lim}(d-0.42_{xu,lim})$

= 0.36 *25* 1000*54.74*(119 - 0.42*54.74)

= 47.3 kN-m > M_{max} (6.48 kN-m)

Therefore, singly reinforced section can be designed.

Step 5: Design for Reinforcement

$M_x+ = 0.87 f_y \mathbf{Ast^+} d \left(1- \frac{\boldsymbol{Ast+fy}}{bdfck}\right)$	$\mathbf{(Ast+)_x}$ = 95.47 mm2
$M_x- = 0.87 fy \mathbf{Ast^-} d \left(1-\frac{\boldsymbol{Ast-fy}}{bdfck}\right)$	$\mathbf{(Ast-)_x}$ = 128 mm2
$M_y+ = 0.87 fy \mathbf{Ast^+} d \left(1-\frac{\boldsymbol{Ast+fy}}{bdfck}\right)$	$\mathbf{(Ast+)_y}$ = 95.47 mm2
$M_y- = 0.87 fy \mathbf{Ast^-} d \left(1-\frac{\boldsymbol{Ast-fy}}{bdfck}\right)$	$\mathbf{(Ast-)_y}$ = 128 mm2

The mild steel reinforcement in either direction in slabs shall not be less than 0.15% of the total cross sectional. However, this value can be reduced to 0.12% when high strength deform bars are used. **(IS 456:2000 Cl. 26.5.2.1)**

$(A_{st})_{min}$ = 0.12% of bD = 0.12% *1000*140 = 168 mm^2 > (A_{st}) required, so minimum rebar(168mm^2) is provided.

a. Reinforcement in x-direction

Use 12mm Ø bars then,

Spacing in y-direction taking maximum moment at x-direction

$$=\frac{\pi * 12^2/4}{168} * 1000 = 673.2 \text{ mm}$$

But, as per codal provision, spacing ≤ 300mm

spacing ≤ 3x119mm = 357mm

So, we adopt spacing of bars = 300 mm

(A_{st}, provided)$_x$ = $\frac{\pi*12^2/4}{300}$ $*1000$ = 376.99 mm2

Provide 12 diameter bar at 300 c/c.

b. Reinforcement in y-direction

Use 12mm Ø bars then,

Spacing in x-direction taking maximum moment at y-direction

$=\frac{\pi*12^2/4}{168}*1000$ = 673.2 mm

But, as per codal provision, spacing ≤ 300mm

spacing ≤ 3x119mm = 357mm

So, we adopt spacing of bars = 300 mm

(A_{st}, provided)$_y$ = $\frac{\pi*12^2/4}{300}$ $*1000$ = 376.99 mm2

Provide 12 diameter bar at 300 c/c.

Step 6: Check for Shear

V_u =w*$\frac{r^4}{1+r^4}$ *$\frac{Lx}{2}$ = 25.05*$\frac{1^4}{1+1^4}$ *$\frac{2.844}{2}$ = 17.81 kN

τ_v=$\frac{V_u}{bd}$ = $\frac{17.81*1000}{1000*119}$ = 0.15 N/mm2

P_t = $\frac{376.99}{1000*119}$$*100$= 0.32 %

From **Table 19 IS 456:2000** for Pt = 0.32% and M25 Concrete;

τ_c= 0.3964 N/mm2

Again, from **Clause 40.2.1.1 of IS 456** for slab thickness ≤ 150 mm; k = 1.30

Therefore, Permissible shear stress (τ'_c) = k× τ_c = 1.30×0.3964 = 0.515 N/mm2

Also from **Table 20 of IS 456**; $\tau_{c,max}$ = 3.1 N/mm2 (for M25) i.e. τv < τc' < $\tau c_{,max}$, hence shear reinforcement is not required.

Step 7: Check for Deflection Criteria

$$\frac{l}{d} = \frac{2844}{119} = 23.9$$

α=26 (For Continuous Slab; **IS456:2000 Cl. 23.2.1**)

β=1 ($\beta = \frac{Span}{10}$ for span > 10m, otherwise 1)

fs = 0.58×fy×(A_{st} required/A_{st} Provided) = 0.58×500 × (128/376.99) = 98.46

Hence, for fs = 98.46 and %A_{st} Provided = 0.32%, from **Fig. 4 IS456:2000;** γ= 2

δ=1 (From **Fig. 5 IS456:2000)**

λ=1 (From **Fig. 6 IS456:2000)**

$\alpha\beta\gamma\delta\lambda$= 26×1×2×1×1 = 52

Hence, $\frac{l}{d} \leq \alpha\beta\gamma\delta\lambda$ hence the design is safe in deflection control criterion. **OK**

Step 8: Check for Development Length

The development length (L_d) is given by **(IS 456: 2000, Cl. 26.2)**;

$$Ld = \frac{0.87 f_y \,\emptyset}{4\tau bd}$$

$$= \frac{0.87 * 500 * 12}{4 * 1.6 * 1.4}$$

= 582.589 mm

Also, from **IS456:2000 Cl. 26.2.3.3**

$L_d < 1.3 \frac{M_1}{V_u}$ + Lo

Here M_1 = moment of resistant of the section assuming all the reinforcement at the section to be stressed to f_d.

M_1 = 0.87×fy×Ast provided × d×(1$-\frac{\text{Ast provided}*\text{fy}}{\text{bdfck}}$)

= 0.87 X 500 X 376.99 X 119 X (1$-\frac{376.99*500}{1000*119*25}$)

= 18.278 kN-m

Assuming L_o=0

$1.3 \frac{M_1}{V_u} + L_o = \frac{1.3*18.278*10^3}{17.81}$ = 1334.16 mm > L_d (Ok)

Summary of the design:

Table 18: Slab Design Summary

Parameters	X Direction		Y Direction	
	Top reinforcement	Bottom reinforcement	Top reinforcement	Bottom reinforcement
Design Moment (kNm)	6.48	4.86	6.48	4.86
A_{st},req (mm^2)	128	95.47	128	95.47
Required spacing for 12 Ø (mm)	673.2	1184.6	673.2	1184.6
Provided spacing (mm)	300	300	300	300
A_{st},provided (mm^2)	376.99	376.99	376.99	376.99

• Design of beam

Beams are structural members assigned to transmit the loads from slab to the column through it. Specially, flexure is more dominant than shear in the beam.

There are three types of reinforced concrete beams:

1. Singly reinforced beams

2. Doubly reinforced beams

3. Singly or doubly reinforced flanged beams

In singly reinforced simply supported beams, reinforcements are placed at the bottom of the beam whereas on top in case of cantilever beams.

A doubly reinforced concrete beam is reinforced in both compression and tension regions. The necessity of using steel in compression region arises when depth of the section is restricted due to functional or aesthetic requirements.

A complete design of beam involves consideration of safety under ultimate limit state in flexure, shear, torsion and bond as well as consideration of serviceability limit states of deflection, crack width, durability etc.

Basically, two types of works are performed namely, analysis of section and design of section. In the analysis of a section, it is required to determine the moment of resistance knowing the cross section and reinforcement details.in the design of sections, it is required to determine the cross section and amount of reinforcement knowing the factored design loads.

Concrete Grade = M25

Steel Grade = Fe500

1. **Main beam**

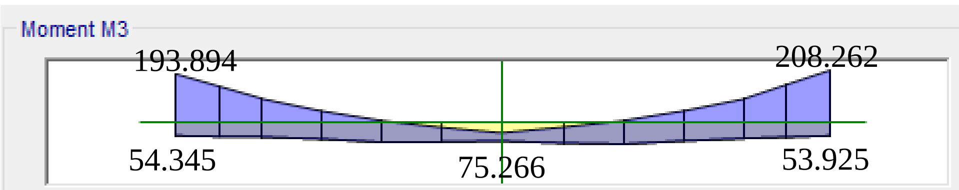


Figure 9: Main beam moment

Main Beam – B18 Unique No: 492 Floor: Second Floor

Known Data

Concrete Grade= **25 MPa**

Steel Grade= **415 MPa**

Overall depth of beam, **D=550mm**

Width of beam, **B= 350mm**

Clear Cover **= 25mm**

Bar Diameter= **25mm**

Effective depth, **d**= 550-25-10-25/2=**502.5 mm**

Table 19: Main beam design

Reference	Step	Calculations				Remarks
IS 13920 2016 Cl. 6.1.2, Cl. 6.1.1, Cl. 6.1.4 and Cl. 6.1.3	**1**	**Check for member size**				
		Width of beam,b=	350 mm		**>200mm**	**OK**
		Depth of beam,D=	550 mm			
		B/D=0.63			**>0.3**	**OK**
		C/C distance =6000 mm				
		Column Size= 600 mm				
		Clear Length = 6000-2*600/2=5400mm				
		Length/depth= 9.82			**>4**	**OK**
IS 13920 2016 Cl.6.2.2	**2**	**Check For Limiting longitudinal reinforcement**				
		Min. Reinforcement**, $A_{st,min}$ =** $0.24\frac{\sqrt{f_{ck}}}{fy}bD$ **= 556.63 mm^2**				
		Max. Reinforcement**, $A_{st,max}$ =** 0.025bd= **4397 mm^2**				
IS 456 2000 Cl. 26.5.1		Min. Reinforcement**,** Ast,min= $\frac{0.87}{fy}bd$ **= 368.70 mm^2**				
		Max. Reinforcement**,** Ast,max= 0.04bd= 7035mm^2				

	3	**$M_{u,lim}/f_{ck}bd^2$ =0.138 for Fe415**	
		$M_{u,lim}$=0.138$f_{ck}bd^2$ = **304.90** kNm	
		At left end(top)	
		Hogging (Negative) moment, M_u = 193.9 KNm	
		Torsional Moment, T_u = 0 KN-m	
		Bending moment equivalent to torsion,	
IS 456:2000 Cl. 41.4.2		M_t = $\frac{T_u(1+\frac{D}{b})}{1.7}$ = 0.000KNm	
		$M=M_u+M_t$= 193.894 kNm	
(IS 456-2000 ANNEX G. G.1.1 b)		Since M< $M_{u,lim}$, provide singly reinforced section	
		$$M = 0.87\, f_y\, A_{st} d\left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$$	
		$$A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6\frac{M}{f_{ck} bd^2}}\right) bd$$	
		A_{st}(top)= 1206.68mm^2	
		Percentage of tension steel (%) = 0.63%	
IS 13920:2016 Cl. 6.2.4		Also,	A_{sc}= 50% of A_{st} or $A_{st,min}$ whichever is greater
		Area of compression steel(A_{sc})= **603.339** mm^2	
		Percentage of compression steel= 0.31%	
		At left end(bottom)	
		Sagging (Positive) moment, M_u = 54.345 kNm	
		Torsional Moment, T_u = 2.515 kNm	
		Bending moment equivalent to torsion,	

		$Mt = \frac{T_u(1+\frac{D}{b})}{1.7}$	= 3.80 kNm		
		$M = M_u + M_t$ = 58.149KNm			
		Since M< $M_{u,lim}$, provide singly reinforced section			
		$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$			
		$A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6\frac{M}{f_{ck} b d^2}}\right) bd$			
		A_{st}(bottom)=	331.01 mm^2	<$A_{st,min}$	
		So, provide A_{st}(bottom)=$A_{st,min}$=556.63 mm2			
		Percentage of tension steel (%) = 0.29%			A_{sc}= 50% of A_{st} or $A_{st,min}$ whichever is greater
		Also,			
		Area of compression steel(A_{sc}) = **556.63** mm^2			
		Percentage of compression steel= 0.29%			
		At mid(top)			
		Hogging (Negative) moment, M_u = 0 KNm			
		So, provide A_{st}(top)= $A_{st,min}$=	556.63 mm^2		
		Percentage of tension steel (%) = 0.29 %			
		Also,			
		Area of compression steel(A_{sc})= **556.627** mm^2 (Bottom)			
		Percentage of compression steel (%) = 0.29%			
		At mid(bottom)			
		Sagging (Positive) moment, M_u=	**75.266 kNm**		
		Torsional Moment, T_u =	0.634 kNm		
		Bending moment equivalent to torsion,			

		$M_t = \dfrac{T_u(1+\frac{D}{b})}{1.7}$ = 0.96 kNm				
		$M=M_u+M_t$=	76.225 kNm			
		Since M< $M_{u,lim}$, provide singly reinforced section				
		$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$				
		$A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6 \frac{M}{f_{ck} b d^2}}\right) bd$				
		A_{st}(bottom)=	438.50 mm^2	<$A_{st,min}$		
		So, provide A_{st}(bottom)=$A_{st,min}$=556.63 mm2				
		Percentage of tension steel (%) =	0.29 %			
		Also,				
		Area of compression steel(A_{sc})= 556.63 mm^2				
		Percentage of compression steel=	0.29	%		
		At right end(top)				
		Hogging (Negative) moment, M_u =		**208.6**	**KNm**	
		Torsional Moment, T_u =		2.832	KN-m	
		$M_t = \dfrac{T_u(1+\frac{D}{b})}{1.7}$=	4.284 kNm			
		$M=M_u+M_t$=	212.904	kNm		
		Since M< $M_{u,lim}$, provide singly reinforced section				
		$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$ $A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6 \frac{M}{f_{ck} b d^2}}\right) bd$				
		A_{st}(top)=	1344.76	mm^2		
		Percentage of tension steel (%) =	0.70	%		
		Also,				A_{sc}= 50% of A_{st} or $A_{st,min}$
		Area of compression steel(A_{sc})= 672.381 mm^2				

						whichever is greater
		Percentage of compression steel=	0.35 %			
		At right end(bottom)				
		Sagging (Positive) moment, M_u=	**53.925**	**kNm**		
		Torsional Moment, T_u =	0 KNm			
		Bending moment equivalent to torsion,				
		$$Mt = \frac{T_u(1+\frac{D}{b})}{1.7} = 0.00\ kNm$$				
		$M=M_u+M_t$=	53.925	kNm		
		Since $M< M_{u,lim}$, provide singly reinforced section				
		$$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$$				
		$$A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6 \frac{M}{f_{ck} bd^2}}\right) bd$$				
		A_{st}(bottom)=	298.35	mm^2	$<A_{st,min}$	
		So, provide A_{st}(bottom)=$A_{st,min}$=556.63 mm2				
		Percentage of tension steel (%) =	0.29%			
		Also,				
		Area of compression steel(A_{sc})= 556.63 mm^2				
		Percentage of compression steel=	0.29%			

Longitudinal Detailing

Table 20: Longitudinal rebar detail in Main beam

Position	Area of Steel(mm^2)	Bars(mm)	Area Provided(mm^2)	Provided % steel
Left(Top)	1206.68	2-25φ+2-20φ	1610	0.84
Left(Bottom)	603.34	2-25φ	981.7	0.51
Mid(Top)	556.63	2-25φ	981.7	0.51
Mid(Bottom)	556.63	2-25φ	981.7	0.51
Right(Top)	1344.76	2-25φ+2-20φ	1610	0.84
Right(Bottom)	672.38	2-25φ	981.7	0.51

Check for Shear

Table 21: Main beam shear check

Reference	Step	Calculations				Remark
		From Analysis Results, Shear force at :				
		Left end=	146.07 kN			
		Mid span=	54.63 kN			
		Right end=	168.82 kN			
		Torsion at:				
		Left end=	2.515 kNm			
		Mid span=	0.634 kNm			
		Right end=	2.832 kNm			
		From flexure design				
		Percentage of tensile steel at left end=	0.51	%		
		Percentage of tensile steel at mid span=	0.51	%		
		Percentage of tensile steel at right end=	0.51	%		
		Shear force due to formation of plastic hinges at both ends:				
		$V_{a,D+L}$=	85.566	kN		
		$V_{b,D+L}$=	106.848	kN		
		$M_{u,AH}$=	247.7195	kNm		
		$M_{u,BH}$=	247.7195	kNm		
		$M_{u,AS}$=	161.6115	kNm		
		$M_{u,BS}$=	161.6115	kNm		
		Due to formation of Plastic hinges at both ends of beam				
IS 13920:2016		**For sway to right:**				
Cl. 6.3.2		$V_{u,a} = V_{u,a}^{D+L} - 1.4\dfrac{M_u^{As} + M_u^{Bh}}{L_{AB}}$	-9.94 kN			
		$V_{u,b} = V_{u,b}^{D+L} - 1.4\dfrac{M_u^{As} + M_u^{Bh}}{L_{AB}}$	202.36 kN			
		For sway to left:				

		$V_{u,a} = V_{u,a}^{D+L} - 1.4\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$	9.94 kN			
		$V_{u,b} = V_{u,b}^{D+L} - 1.4\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$	-202.36 KN			
		Hence, Design Shear Force at:				
		Left end, V_u=	146.07	kN		
		Mid span, V_u=	54.63	kN		
		Right end, V_u=	202.36	kN		
	1	**At left End**				
		Tensile Steel provided=	981.747	mm^2		
		Percentage of steel provided=	0.51	%		
IS 456:2000		τ_c=	0.493	N/mm^2		
Table 19, 20		$\tau_{c,max}$ =	3.1	N/mm^2		
		V_u=	146.07	kN		
		T_u=	2.515	kNm		
		Equivalent Shear:				
		V_e= V_u+1.6 T_u/b=	**146.0815**	**kN**		**τ_c<τ_{ve} <$\tau c_{,max}$**
		τ_{ve}=V_e/bd=	**0.831**	**N/mm^2**		
		So, Shear reinforcement needs to be designed.				
IS 456:2000		V_{us} = V_e-τ_c*bd=	59.375	kN		
Cl. 41.4.3		Assuming 2-legged 10 mm stirrups,				
		A_{sv} =	157.08	mm^2		
		$A_{sv} = \frac{T_u s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$				
		S_v=	377.86	mm		
IS 456:2000, Cl. 26.5.1.5, 26.5.1.6		Minimum shear reinforcement				
		$s_{v,max} = \frac{0.87 f_y A_{sv}}{(\tau_{ve} - \tau_c) b}$ =	479.98	mm		
IS 13920:2016, Cl. 6.3.5		The spacing of stirrups as confining reinforcement over a length of 2d= 1005mm should be:				
		i. d/4= 125.6 mm				
		ii. 8 times minimum diameter of longitudinal bar= 200mm				

		iii. However, it need not be less than 100mm				
		Provide 10.00mm diameter 2-legged vertical stirrups @100.00mm c/c up to a length of 2d= 1005mm from left end.				
	2	**At mid**				
		Tensile Steel provided=	981.74	mm^2		
		Percentage of steel provided= 0.51%				
IS 456:2000 Table 19, 20		τ_c=	0.493	N/mm^2		
		$\tau_{c,max}$ =	3.1	N/mm^2		
		V_u=	54.63	kN		
		T_u=	0.634	kNm		
		Equivalent Shear:				
		$V_e= V_u+1.6\ T_u/b$=	**54.632**	**kN**		
		$\tau_{ve}=V_e/bd$=	**0.311**	**N/mm^2**		**$\tau_c>\tau_{ve}$**
		So, Shear reinforcement need not be designed.				
		Assuming 2-legged 10 mm stirrups,				
		A_{sv} =	157.08	mm^2		
		$A_{sv}=\frac{T_u s_v}{b_1 d_1 (0.87 f_y)}+\frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$				
		S_v=	1060.254	mm		
IS 456:2000, Cl. 26.5.1.5, 26.5.1.6		Minimum shear reinforcement				
		$s_{v,max} = \frac{0.87 f_y A_{sv}}{(\tau_c - \tau_{ve}) b}$ =	888.54	mm		
IS 13920:2016 Cl. 6.3.5.2		The spacing of stirrups shall not be greater than d/2=251.25 mm.				
		Provide 10.00mm diameter 2-legged vertical stirrups @250.00mm c/c at mid span				
	3	**At right End**				
		Tensile Steel provided=	981.747	mm^2		

		Percentage of steel provided=	0.51	%		
IS 456:2000		τ_c=	0.493	N/mm^2		
Table 19, 20		$\tau_{c,max}$ =	3.1	N/mm^2		
		V_u=	202.36	kN		
		T_u=	2.832	kNm		
		Equivalent Shear:				
		V_e= V_u+1.6 T_u/b=	**202.371**	**kN**		
		τ_{ve}=V_e/bd=	**1.151**	**N/mm^2**		**τ_c<τ_{ve}< τ_{max}**
		So, Shear reinforcement needs to be designed.				
IS 456:2000		V_{us} = V_e-τ_c*bd=	115.665	kN		
Cl. 41.4.3		Assuming 2-legged 10 mm stirrups,				
		A_{sv}=$\frac{T_u s_v}{b_1 d_1 (0.87 f_y)}+\frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$				
		A_{sv}=157.08 mm^2				
		S_v=	280.336	mm		
		$s_{v,max}=\frac{0.87 f_y A_{sv}}{(\tau_{ve}-\tau_c)b}$ =	246.39	mm		
IS 13920:2016, Cl. 6.3.5		The spacing of stirrups as confining reinforcement over a length of 2d= 1005mm should be:				
		i. d/4= 125.63mm				
		ii. 8 times minimum diameter of longitudinal bar =	200 mm			
		iii. However, it need not be less than 100mm				
		Provide 10.00mm diameter 2-legged vertical stirrups @100.00mm c/c up to a length of 2d= 1005mm from left end.				
IS 456-2000 Cl.26.5.1.7 b		**Design of side reinforcement**				
		Since d< 750mm, side face reinforcement is not needed				

Check for deflection

Table 22: Deflection check in Main beam

Reference	Step	Calculations				Remarks
IS 456-2000 Cl.22.2		Clear span=	5450	mm		
		Width of the support=	350.00	mm		
		1/12 of clear span=	454.17	mm	<350	

		So, effective span shall be taken lesser of 6000mm or 5450+502.5=5952.5mm				
IS 456-2000 Cl.23.2.1		Effective Length(L_x)=	5952.5	mm		
		α=	26			
		β=	1			Span<10m
		For λ,				
		$fs = 0.58 f_y \frac{Ast_{required}}{Ast_{provided}}$ =	136.47	N/mm^2		
		% A_{st}=	0.51			
		λ=	2			
		γ=	1.15			
		δ=	1			
		So,				
		αβλγδ=	59.8			
		$\frac{Lx}{d}$ =	11.87			≤αβλγδ **(OK)**

Check for development length

Table 23: Development length check in Main beam

Reference	**Calculations**		**Remarks**
IS456-2000 Cl.26.2.3.3	$L_d = \frac{\emptyset \sigma_s}{4\tau_{bd}} = \frac{25 * 0.87 * 415}{4 * 1.6 * 1.4} =$ $L_d \leq 1.3\frac{M}{V} + L_o$ $M = 0.87 f_y A_{st,prov}(d - \frac{f_y A_{st,prov}}{f_{ck} b})$	1007.39 mm (For tension) 805.915 mm(For compression)	
	$A_{st,prov}$=	981.747 mm^2	
	M=	166.042 kNm	
	V =	153.21 kN	
	$L_0 = \frac{350}{2} - 25$ = 150 mm		
	$= 1.3\frac{M}{V} + L_o$ =1558.88 mm	>L_d	
	Extend the bars 150mm from the center of the support.		

2. **Secondary beam**

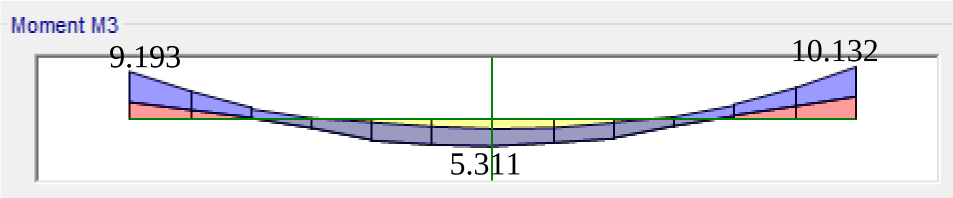


Figure 10: Secondary beam moment

Main Beam – B59 Unique No: 492 Floor: First Floor
Known Data
Concrete Grade= **25 MPa**
Steel Grade= **415 MPa**
Overall depth of beam, **D=300mm**
Width of beam, **B= 200mm**
Clear Cover **= 25mm**
Bar Diameter= **20mm**
Effective depth, **d**= 300-25-8-20/2=**257 mm**

Table 24: Secondary beam design

Reference	**Step**	**Calculations**					**Remark**
	1	**Check for member size**					
IS 13920 2016 Cl. 6.1.2, Cl. 6.1.1, Cl. 6.1.4 and Cl. 6.1.3		Width of beam, b=	200 mm			>200 mm	**OK**
		Depth of beam, D=	300 mm				
		B/D=	0.67			>0.3	**OK**
		C/C distance =6000 mm					
		Column Size= 600 mm					
		Clear Length =	6000-2*600/2=	5400	mm		
		Length/depth=	9.82			>4	**OK**
	2	**Check For Limiting longitudinal reinforcement**					
IS 13920 2016 Cl.6.2.2		Min. Reinforcement, $A_{st,min} = 0.24\frac{\sqrt{f_{ck}}}{fy}bD = 173.49\ mm^2$					
		Max. Reinforcement, $A_{st,max}$= 0.025bd=	1285	mm^2			
IS 456 2000 Cl. 26.5.1		Min. Reinforcement, $A_{st,min} = \frac{0.87}{fy}bd = 107.75\ mm^2$					
		Max. Reinforcement, $A_{st,max}$= 0.04bd=	2056	mm^2			

	3	$M_{u,lim}/f_{ck}bd^2$ =0.138 for Fe415	
		$M_{u,lim}$=0.138$f_{ck}bd^2$ = 45.57 kNm	
		At left end(top)	
		Hogging (Negative) moment, M_u = 9.193 kNm	
		Torsional Moment, T_u = 0	
		Bending moment equivalent to torsion,	
IS 456:2000 Cl. 41.4.2		Mt=$\frac{T_u(1+\frac{D}{b})}{1.7}$ = 0.000kNm	
		M=M_u+M_t= 9.193kNm	
		Since M< $M_{u,lim}$, provide singly reinforced section $M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$	
		Ast(top)=102.52<Ast,min	
		Ast(top)= 173.49mm^2	
		Percentage of tension steel (%) = 0.29 %	
		Also,	
		Area of compression steel(A_{sc})(bottom) = 173.5 mm^2	
		Percentage of compression steel= 0.29%	
	4	**At left end(bottom)**	
		Sagging (Positive) moment, M_u = 0 kNm	
		Torsional Moment, T_u = 0 kNm	
		So, provide A_{st} (bottom)=$A_{st,min}$=173.49 mm^2	
		Percentage of tension steel (%) = 0.29 %	A_{sc}= 50% of A_{st} or $A_{st,min}$ whichever is greater
		Also,	
		Area of compression steel(A_{sc})(Top) = 173.5 mm^2	
		Percentage of compression steel= 0.29 %	
	5	**At mid(top)**	
		Hogging (Negative) moment, M_u = 0 KNm	
		So, provide A_{st}(top)= $A_{st,min}$= 173.5 mm^2	
		Percentage of tension steel (%) = 0.29 %	
		Also,	
		Area of compression steel(A_{sc})= 173.49 mm^2 (Bottom)	
		Percentage of compression steel (%) = 0.29%	

	6	**At mid(bottom)**			
		Sagging (Positive) moment, M_u=	5.311	kNm	
		Torsional Moment, T_u =	0		
		Bending moment equivalent to torsion,			
		$Mt = \frac{T_u\left(1+\frac{D}{b}\right)}{1.7} = 0$			
		$M=M_u+M_t$=	5.311 kNm		
		Since M< $M_{u,lim}$, provide singly reinforced section			
		$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$			
		$A_{st} = 0.5\, \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - 4.6\frac{M}{f_{ck} b d^2}}\right) bd$			
		A_{st}(bottom)=	58.37 mm^2	<$A_{st,min}$	
		So, provide A_{st}(bottom)=$A_{st,min}$=173.49 mm2			
		Percentage of tension steel (%) =	0.29%		
		Also,			
		Area of compression steel(A_{sc})(Top) = 173.49 mm^2			
		Percentage of compression steel=	0.29%		
	7	**At right end(top)**			
		Hogging (Negative) moment, M_u =	10.132	kNm	
		Torsional Moment, T_u =	0		
		Bending moment equivalent to torsion,			
		$Mt = \frac{T_u(1+\frac{D}{b})}{1.7} = 0.000$ kNm			
		$M=M_u+M_t$=	10.132 kNm		
		Since M< $M_{u,lim}$, provide singly reinforced section			
		$M = 0.87\, f_y\, A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b\, d}\right)$			
		A_{st}(top)=	113.40mm^2		
		Percentage of tension steel (%) =	0.19 %	<$A_{st,min}$	
		So, provide A_{st}(bottom)=$A_{st,min}$=173.49 mm2			
		Percentage of tension steel (%) =	0.29	%	

		Also,			A_{sc}= 50% of A_{st} or $A_{st,min}$ whichever is greater
		Area of compression steel(A_{sc})(Top) = 173.49 mm^2			
		Percentage of compression steel=	0.29 %		
		At right end(bottom)			
		Sagging (Positive) moment, M_u=	0	kNm	
		Torsional Moment, T_u =	0	kNm	
		So, provide A_{st}(top)= $A_{st,min}$=	173.49	mm^2	
		Percentage of tension steel (%) =	0.29	%	
		Also,			
		Area of compression steel(A_{sc})=	173.49	mm^2 (Bottom)	
		Percentage of compression steel (%) = 0.29%			

Longitudinal Detailing

Table 25: Longitudinal detail in secondary beam

Position	Area of Steel	Bars	Area Provided	Provided % steel
Left(Top)	173.49	2-20φ	628.32	1.05
Left(Bottom)	173.49	2-20φ	628.32	1.05
Mid(Top)	173.49	2-20φ	628.32	1.05
Mid(Bottom)	173.49	2-20φ	628.32	1.05
Right(Top)	173.49	2-20φ	628.32	1.05
Right(Bottom)	173.49	2-20φ	628.32	1.05

Check for shear

Table 26: Shear check secondary in beam

Reference	**Step**	**Calculations**				**Remark**
		From Analysis Results, Shear force at:				
		Left end=	8.771	kN		
		Mid span=	1.067	kN		
		Right end=	9.102	kN		
		Torsion at:				
		Left end=	0	kNm		
		Mid span=	0	kNm		
		Right end=	0	kNm		
		From flexure design				

		Percentage of tensile steel at left end=	1.05	%		
		Percentage of tensile steel at mid span=	1.05	%		
		Percentage of tensile steel at right end=	1.05	%		
		Shear force due to formation of plastic hinges at both ends:				
		$V_{a,D+L}$=	7.953	kN		
		$V_{b,D+L}$=	8.575	kN		
		$M_{u,AH}$=	107.234	kNm		
		$M_{u,BH}$=	107.234	kNm		
		$M_{u,AS}$=	107.234	kNm		
		$M_{u,BS}$=	107.234	kNm		
IS 13920:2016		**For sway to right:**				
Cl. 6.3.2		$V_{u,a} = V_{u,a}^{D+L} - 1.4\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}$	-42.09	kN		
		$V_{u,b} = V_{u,b}^{D+L} - 1.4\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}$	58.62	kN		
		For sway to left:				
		$V_{u,a} = V_{u,a}^{D+L} - 1.4\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$	= 58.00kN			
		$V_{u,b} = V_{u,b}^{D+L} - 1.4\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$	=-41.47	kN		
		Hence, Design Shear Force at:				
		Left end, V_u=	58.00	kN		
		Mid span, V_u=	1.067	kN		
		Right end, V_u=	58.62	kN		
	1	**At left End**				
		Tensile Steel provided=	628.3	mm^2		
		Percentage of steel provided=	1.05	%		
IS 456:2000		τ_c=	0.645	N/mm^2		
Table 19, 20		$\tau_{c,max}$ =	3.1	N/mm^2		

		V_u=	58.00	kN		
		T_u=	0	kNm		
		Equivalent Shear:				
		$V_e= V_u+1.6\ T_u/b$=	57.99	kN		
		$\tau_{ve}=V_e/bd$=	0.645	N/mm^2		$\tau_c=\tau_{ve}$
		So, Shear reinforcement need not be designed.				
	Assuming 2-legged 8 mm stirrups, A_{sv} =100.53 mm^2					
IS 456:2000, Cl. 26.5.1.5, 26.5.1.6		Minimum shear reinforcement				
		$s_{v,max} = \dfrac{0.87\ fy\ A_{sv}}{0.4b}$	259.26	mm		
		Maximum spacing=0.75d=	192.75	mm		
IS 13920:2016, Cl. 6.3.5		The spacing of stirrups as confining reinforcement over a length of 2d= 514mm should be:				
		i. d/4= 64.25 mm				
		ii. 8 times minimum diameter of longitudinal bar=160 mm				
		iii. However, it need not be less than 100mm				
		Provide 8.00mm diameter 2-legged vertical stirrups @100.00mm c/c up to a length of 2d= 514mm from left end.				
	2	**At mid**				
		Tensile Steel provided=	628.318	mm^2		
		Percentage of steel provided=	1.05	%		
IS 456:2000 Table 19, 20		τ_c=	0.645	N/mm^2		
		$\tau_{c,max}$ =	3.1	N/mm^2		
		V_u=	1.07	kN		
		T_u=	0	kNm		
		Equivalent Shear:				
		$V_e= V_u+1.6\ T_u/b$=	1.067	kN		
		$\tau_{ve}=V_e/bd$=	0.012	N/mm^2		$\tau_c>\tau_{ve}$
		So, Shear reinforcement need not be designed.				
IS 456:2000 Cl. 40.4						
		Assuming 2-legged 8 mm stirrups,				
		A_{sv} =	100.53	mm^2		
		Minimum shear reinforcement				

IS 456:2000, Cl. 26.5.1.5, 26.5.1.6		$s_{v,max} = \frac{0.87\,fy\,A_{sv}}{0.4b}$	259.26	mm		
		Maximum spacing=0.75d=	192.75	mm		
IS 13920:2016 Cl 6.3.5.2		The spacing of stirrups shall not be greater than d/2=128.5 mm.				
		Provide 8.00mm diameter 2-legged vertical stirrups @120.00mm c/c at mid span				
	3	**At right End**				
		Tensile Steel provided=	628.318	mm^2		
		Percentage of steel provided=	1.05	%		
IS 456:2000 Table 19, 20		τ_c=	0.645	N/mm^2		
		$\tau_{c,max}$ =	3.1	N/mm^2		
		V_u=	58.62	kN		
		T_u=	0	kNm		
		Equivalent Shear:				
		$V_e= V_u+1.6\ T_u/b$=	58.6175	kN		
		$\tau_{ve}=V_e/bd$=	0.652	N/mm^2		$\tau_c<\tau_{ve}<\tau_{c,max}$
		So, Shear reinforcement needs to be designed.				
IS 456:2000 Cl. 40.4		$V_{us} = V_e-\tau_c*bd$=	0.600	kN		
		Assuming 2-legged 8 mm stirrups,	= 100.53	mm^2		
		$S_v= \frac{0.87 f_y A_{sv} d}{Vus}$ =	24300.7	mm		
IS 456:2000, Cl. 26.5.1.5, 26.5.1.6		Minimum shear reinforcement				
		$s_{v,max} = \frac{0.87\,fy\,A_{sv}}{0.4b}$	405.10	mm		
		Maximum spacing=0.75d=	192.75	mm		
IS 13920:2016, Cl. 6.3.5		The spacing of stirrups as confining reinforcement over a length of 2d= 514mm should be:				
		i. d/4= 64.25 mm				

		ii. 8 times minimum diameter of longitudinal bar=	200 mm	
		iii. However, it need not be less than 100mm		
		Provide 10.00mm diameter 2-legged vertical stirrups @100.00mm c/c up to a length of 2d= 514mm from left end.		
IS 456-2000 Cl.26.5.1.7b		**Design of side reinforcement**		
		Since d< 750mm, side face reinforcement is not needed		

Check for deflection

Table 27: Deflection check in secondary beam

Reference	Step	Calculations				Remarks
IS 456-2000 Cl.22.2		Clear span=	5400	mm		
		Width of the support=	350.00	mm		
		1/12 of clear span=	450.00	mm	>350	
IS 456-2000 Cl.23.2.1		Effective Length(L_x)=	5400	mm		
		α=	26			
		β=	1	Span<10m		
		For λ,				
		$fs = 0.58 f_y \frac{Ast_{required}}{Ast_{provided}}$=	199.66	N/mm^2		
		% A_{st}=	1.02			
		λ=	1.1			
		γ=	1.25			
		δ=	1			
		So, αβλγδ=	35.75			
		$\frac{Lx}{d}$ =	21.01			≤αβλγδ **(OK)**

Check for development length

Table 28: Development length check in secondary beam

Reference	Step	Calculations			Remarks
IS456-2000 Cl.26.2.1		$L_d = \frac{\emptyset \sigma_s}{4\tau_{bd}} = \frac{25 * 0.87 * 415}{4 * 1.6 * 1.4}$	=1007.394	mm (For tension)	

		$L_d = \frac{\emptyset\sigma_s}{4\tau_{bd}} = \frac{25 * 0.87 * 415}{4 * 1.6 * 1.4 * 1.25}$	=805.9152	mm (For compression)	
		Also, $L_d \leq 1.3\frac{M}{V} + L_o$			
IS456-2000 Cl.26.2.3.3		$M = 0.87f_y A_{st,prov}(d - \frac{f_y A_{st,prov}}{f_{ck}b})$			
		$A_{st,prov}$=	628.3185	mm^2	
		M=	46.4710	kNm	
		V=	153.21	kN	
		$L_0 = \frac{200}{2}$ - 25=	75	mm	
		$1.3\frac{M}{V} + L_o$	469.310	mm	>L_d
		Extend the bars 75mm from the center of the support.			

• Design of Column

Columns are vertical structural members which are predominantly subjected to axial forces. Columns support the beams and slabs and transfer the load to the foundation.

On the basis of whether slenderness effects are considered insignificant or not, the column may be classified as either a short column or a long column respectively. A short column generally fails by direct compression whereas a long column fails by buckling. Slenderness expressed in terms of the slenderness ratio, which is the ratio of the effective length to the least lateral dimension of the column.

Based on the nature of loading, columns may be classified as either axially loaded, uniaxially loaded or biaxially loaded columns. Axially loaded columns are under pure axial compression whereas uniaxially and biaxially loaded columns have eccentric loading in one or both directions respectively. Columns in framed structures under seismic loads are generally biaxially loaded and hence designed accordingly.

The following steps were followed in the design of columns.

Step 1: Preliminary Design

The preliminary sizing of the column members is carried out by using the tributary area method. The dead load and live load acting within the influence area of the column is calculated and factored by 1.5 and the size of column is determined by designing the column as an axially loaded column with a certain amount of longitudinal reinforcement (2%).

Step 2: Calculate effective length of column and check for long/short column.

The effective length of column is determined based on the end supports and the sway/no sway condition. Depending on the ratio of the effective length to the least lateral dimension of the column, the column is classified as either a short column or a long column. A column is long if the ratio of its effective length to its least lateral dimension is greater than 12.

Step 3: Check for eccentricity

In columns, eccentricity refers to the deviation of the point of application of axial load from the center of the column. Eccentric load causes the column to bend towards loaded point and hence generates a bending moment in the column.

Step 4: Design of longitudinal reinforcement

The amount of longitudinal reinforcement required is then determined as a percentage of the gross sectional area.

Step 5: Design of Lateral Ties

The diameter and spacing of lateral ties is then determined based on the design shear strength. The diameter of the ties shall not be lesser than the greatest of the following two values

Column Design for: Interior Column

Table 29: Interior column design

Column	C29 (CC6)	Grid Point 3-C
Storey	Upper Basement	
Load Combination	1.5(DL+LL)	

S.N.	Reference	Calculations	Values	Units	Remarks
1.		**Known Data**			
		Square Column			Depth of column, D_y=600mm
		Width of column, D_x	600	mm	
		Beam width	350	mm	
		Beam depth	550	mm	
		Height of floor	3.465	mm	
		Unsupported length	2.915	m	
		Assumptions			
		Clear cover	40	mm	min for column
		Longitudinal bar diameter, φ_l	25	mm	
		Effective cover, d'	54	mm	
		Grade of concrete, f_{ck}	25	MPa	
		Grade of steel, f_y	415	MPa	
2.		**Check for Maximum Axial Stress**			
	IS 13920:2016 Cl.7.1	0.1 f_{ck}	2.5	MPa	
		Factored Axial Load	6358.68	kN	
		Factored Axial Stress	17.66	MPa	>0.1 f_{ck} **(Ok)**
3.	**IS 13920:2016 Cl.7.1.2**	**Member size**			
		Dimension of column (D)	600	mm	>300mm **(Ok)**
		Since it is square column, B/D=1	1	>0.4	**(Ok)**
4.	**IS 13920:2016 Cl.7.1.1**	**Check for Minimum Member Size**			
		Dimension of column	600	mm	
		Area=	360000	mm^2	
		Minimum dimension=20d_b or 300mm	500	mm	d_b=25 mm
5.		**Check for Slenderness**			
	IS 456: 2000	Moment of Inertia of Section	With Modifiers		Modifiers

	Clause 25.2 Annex E	MOI of column=$bd^3/12$	0.00756	m^4	0.7	
		MOI of beam section=$bd^3/12$	0.0016984	m^4	0.35	
		Length of adjoining beam	5.4	m		
		Height of column	2.915	m		
		Flexural stiffness:				
		For columns, k_c=MOI*E/L	0.002593			
		For beams, k_b=MOI*E/L	0.0003145			
		So, $\beta_1=\beta_2=\Sigma K_c/(\Sigma K_c+\Sigma K_b)$ $=2*K_c/(2K_c+4K_b)$	0.80479	0.7 and 0.35 taken as modifier for cracked section		
		Type of Frame (Sway or Non-sway)				
		Stability index (Q_x=0.012019, Q_y=0.00669)		<0.04		
		It is non-sway frame as per calculations.				
		For non-sway frame				
	Fig 26	Effective length ratio for both axes	0.83			
	IS 456: 2000 Cl.25.1.2	Slenderness ratio=kL/d	4.0324166	<12	**Column is short.**	
6.		**Limiting Longitudinal Reinforcement**				
		Min. Reinforcement=0.8%of Area	2880	mm^2		
		Max. Reinforcement=4% of Area	14400	mm^2		
		But in extreme cases;				
		Max. Reinforcement=6% of Area	21600	mm^2		
7.		**Design Forces in Section**				
	From ETABS	P_u	4964.9491	kN	Maximum axial load at load combination: **1.5(DL+LL)**	
		M_{u2}	11.99	kNm		
		M_{u3}	-5.9704	kNm		
		V_{u2}	0.4959	kN		
		V_{u3}	-8.21	kN		
8.		**Check for Eccentricity**				
	IS 456: 2000	Minimum eccentricity(e_{min}) $=l/500+D/30\geq20mm$	25.83		>20mm	
		Moment due to accidental ecc.=Pu*e_{min}	128.24463 53	kNm	>M_{u3} and >M_{u2}	
		Design moment=M_{ux}=	128.2446	kNm		
		Design moment=M_{uy}=	128.2446	kNm		
		Column is designed as bi-axially loaded column.				

9.	**IS 456:2000 Cl 39.6**	**Longitudinal Reinforcement**				
		d'/D=	0.09	Take 0.1		
		$P_u/(f_{ck}bD)$=	0.552			
		Trial 1: take p=	1.560	%		
		p/f_{ck}	0.062			
		Referring to chart 44, SP-16				
		$M_{ux}/f_{ck}D^3$=	0.041		from table	
		$M_{uy}/f_{ck}D^3$=	0.041		from table	
		$M_{ux,lim}$=	221.4			
		$M_{uy,lim}$=	221.4	kNm	$>M_{u,design}$ **(Ok)**	
		Check for biaxial bending				
		$P_{uz}=0.45f_{ck}A_c+0.75f_yA_{sc}$	5734.8	kNm		
		P_u/P_{uz}=	0.865758	>0.8		
		αn=	2			
		$(M_{ux}/Mu_{x1})^{\alpha n}+(M_{uy}/M_{uy1})^{\alpha n}$=	0.671046	<1.0	**(Ok)**	
		Therefore, required longitudinal steel=1.56%.				
		Area of steel required =	5616	mm^2		
		Provide 12-25mmΦ bars as longitudinal steel bars.				
		Provided A_{st} =	5892.857	mm^2	>required **(Ok)**	
10.	**IS 456:2000**	**Design for shear**				
		Percentage steel provided in tension=	0.818	%		
		Design shear strength of concrete, τ_c=	0.589	N/mm^2		
	Cl 40.2.2	Shear strength of members under axial compression				
		$\delta=1+3P_u/(Agf_{ck})$	2.65498	>1.5	Take 1.5	
		Actual τc=	0.8835			
		Shear capacity of the section, Vcx=Vcy=	318.06	kN		
		Shear Force due to Formation of Plastic Hinge				
	End moment of the beam is tabulated below		$\mathbf{M_R^{LS}}$	$\mathbf{M_R^{LH}}$	$\mathbf{M_R^{RS}}$	$\mathbf{M_R^{RH}}$
	X-axis	Moment from ETABS kNm	84.84	148.55	84.84	151.40
	Y-axis	Moment from ETABS kNm	73.18	142.13	62.48	121.92
	IS 13920:2016 Cl 7.5	$V_p=1.4(M_R^{LS}+M_R^{RH})/H$ or $1.4(M_R^{RS}+M_R^{LH})/H$				
		V_{px1}	95.454	kN	$>V_u$ from ETABS	

		V_{px2}	94.302	kN	$>V_u$ from ETABS
		V_{py1}	78.833	kN	$>V_u$ from ETABS
		V_{py2}	82.672	kN	$>V_u$ from ETABS
		Design shear forces			
		V_{dx}=	95.45	kN	
		V_{dy}=	82.672	kN	
	IS 456:2000 Table 20	**Maximum shear stress:**			
		τ_{cmax} for M25=	3.1	N/mm^2	
		$\tau_{vx}=V_{dx}/A$	0.265	N/mm^2	
		$\tau_{vy}=V_{dy}/A$	0.229	N/mm^2	
		τ_{cdx}	0.8835	N/mm^2	
		τ_{cdy}	0.8835	N/mm^2	
	IS 456:2000 Clause 40.2.3 & 40.3	$\tau_{cd} > \tau_v$ also $\tau_{cd}/2 > \tau v$ theoretically shear reinforcement is not provided but minimum shear reinforcement is provided.			
		Minimum shear reinforcement			
	IS 456:2000 Cl 40.3 & Cl 26.5.1.6	$A_{svmin}/S_v=0.4*b/0.87*f_y$	664.73	mm^2/m	Which is similar to the results from ETABS 20
		f_y is taken as $415N/mm^2$			
11.		**Design of Transverse Reinforcement**			
		Diameter of ties			
		φ_t must be greater than or equal			
	IS 13920:2016 Cl.7.4.2	a) 8mm			
	IS 456:2000 Cl 26.5.3.2.c	b) 1/4*25mm (we used 25mm for longitudinal bars)	6.25	mm	
		Hence, adopt ties of 8mm φ	8	mm	
		Area of tie (two legged) =	100.5714	mm^2	
		Spacing of the ties			
	IS 13920:2016 Cl.7.4.2	S_v <= 1/2 least lateral dimension of the compression member	300	mm	

	IS 456:2000 Cl 26.5.3.2.c	S_v <= 16 times the dia of smallest longitudinal bar	400	mm		
		S_v <=	300	mm		
		Spacing of the ties= Area/A_{svmin}/S_v	151.2971	mm		
		Thus, provide 8mm φ lateral ties @ 150mm c/c in the central part.				
	IS 456:2000	**Spacing between longitudinal bars**				
	Cl.26.5.3.2 b	(600-40*2-8*2-25)/3	159.67	mm	>75mm	
		So, additional ties need to be provided requires smaller spacing of stirrups for higher ductility.				
12.	**IS 13920:2016 Cl. 8.1.a)**	**Special confining reinforcement**				
		Special confining reinforcement has to be provided over the length l_o from each joint face towards mid span, and on the either side of any section, where flexural yielding may occur under the effect of earthquake.				
		l_o should be more than				
		1.Lateral dimension of column	600	mm		
		2. 1/6 clear span of member	485.83	mm		
		3. 450mm				
		Provide l_o =	**650**	**mm**		
	Cl. 8.1.b)	Spacing should not be more than				
		1. 1/4 of minimum member dimension of column or beam	150	mm		
		2. 6 times diameter of smallest longitudinal rebar	150	mm		
		3. 100mm				
	Cl. 8.1.c)	Area A_{sh} of cross section of bar forming links of at least				
		A_{sh}= pi*8^2/4	50.28	mm2		
		A_{sh}= Maximum of				
		0.18 s_v h f_{ck}/f_y(Ag/Ak-1)				
		0.05 s_v h f_{ck}/f_y				
		where,				
		Ak= area of confined concrete core in the rectangular hoop measured to its outer dimension = $(600-80)^2$	270400	mm^2		
		h=(600-40*2)/3	173.33	mm		
		then, 0.18*s_v*173.33*25/415(360000/270400-1) = 50.286				

		s_v=90.65 mm or, 0.05*s_v*173.33*25/415=50.286 s_v=101.56mm **Provide spacing 100mm**.				
13.		**Splicing of vertical bars**				
	IS 13920:2016 Cl.7.3.2.1	Maximum of 50% bars should be spliced at a section and the clear overlap of the spliced section should be more than development length (L_d) of the largest bars				
	IS 456:2000 Cl. 26.2.1	$L_d=0.87f_y\varphi/(4\tau_{bd})$	805.9151	mm		
		Therefore, provide the overlap of 820 mm in spliced section.				
14.	**IS 13920:2016 Cl.7.4.1**	**Hooks on Transverse Reinforcement**			Amendment	
		Hook length=8φ or 75mm (larger)				
		or, Hook length=8x8 or 75mm				
		or, Hook length=64mm or 75mm				
		Hook length= 75mm				

Column Design for: Corner Column

Column	**C1 (CC13)**	**Grid Point 3-G**
Storey	**Upper Basement**	
Load Combination	**1.5(DL-EQ$_y$-e%)**	

Table 30: Corner column design

S.N.	Reference	Calculations	Values	Units	Remarks
1.		**Known Data**			
		Square Column			Depth of column, D_y=600mm
		Width of column, D_x	600	mm	
		Beam width	350	mm	
		Beam depth	550	mm	
		Height of floor	3.465	mm	
		Unsupported length	2.915	m	
		Assumptions			
		Clear cover	40	mm	
		Longitudinal bar diameter, φ_l	20	mm	
		Effective cover, d'	54	mm	

		Grade of concrete, f_{ck}	25	MPa	
		Grade of steel, f_y	415	MPa	
2.		**Check for Maximum Axial Stress**			
	IS 13920:2016 Cl.7.1	0.1 f_{ck}	2.5		
		Factored Axial Load	6358.68	kN	>0.1 fck
		Factored Axial Stress	17.66	MPa	**(Ok)**
3.	**IS 13920:2016 Cl.7.1.2**	**Member size**			
		Dimension of column (D)	600	mm	>300mm **(Ok)**
		Since it is square column, B/D=1	1	>0.4	**(Ok)**
4.	**IS 13920:2016 Cl.7.1.1**	**Check for Minimum Member Size**			
		Dimension of column	600	mm	
		Area=	360000	mm^2	
		Minimum dimension=$20d_b$ or 300mm	500	mm	d_b=25 mm
5.		**Check for Slenderness**			
	IS 456: 2000 Cl.25.2 Annex E	Moment of Inertia of Section	With Modifier		Modifiers
		MOI of column=$bD^3/12$	0.00756	m^4	x0.7
		MOI of beam section=$bD^3/12$	0.0016984	m^4	x0.35
		Length of adjoining beam	5.4	m	
		Height of column	2.915	m	
		Flexural stiffness:			
		For columns, k_c=MOI*E/L	0.0025934		
		For beams, k_b=MOI*E/L	0.0003145		
		So, $\beta_1=\beta_2=\Sigma K_c/(\Sigma K_c+\Sigma K_b)$ $=2*K_c/(2K_c+2K_b)$	0.8918430	0.7 and 0.35 taken as modifier for cracked section	
		Type of Frame (Sway or Non-sway)			
		Stability index (Q_x=0.012019 ,Q_y=0.00669)		<0.04	
		It is no- sway frame as per calculation.			
		For non-sway frame,			
	Fig 26	Effective length ratio for both axes	0.918		
	Cl.25.1.2	Slenderness ratio=kL/d	4.45995	<12	**Column is short.**
6.		**Limiting Longitudinal Reinforcement**			

		Min. Reinforcement=0.8%of Area	2880	mm^2	
		Max. Reinforcement=4% of Area	14400	mm^2	
		But in extreme cases;			
		Max. Reinforcement=6% of Area	21600	mm^2	
7.		**Design Forces in Section**			
	From ETABS	P_u	1471.2151	kN	
		M_{u2}	94.635	kNm	
		M_{u3}	38.0015	kNm	
		V_{u2}	4.80	kN	
		V_{u3}	32.05	kN	
8.		**Check for Eccentricity**			
	IS 456: 2000	Minimum eccentricity(e_{min}) $=l/500+D/30\geq 20mm$	25.83		>20mm
		Moment due to accidental ecc.=P_u*e_{min}	38.001486	kNm	<Mu_3 and <Mu_2
		Design moment=M_{ux}=	94.635	kNm	
		Design moment=M_{uy}=	38.0015	kNm	
		Column is designed as bi-axially loaded column.			
9.	**IS 456:2000**	**Longitudinal Reinforcement**			
	Cl 39.6	d'/D=	0.09	Take 0.1	
		$P_u/(f_{ck}bD)$=	0.163		
		Trial 1: take p=	1.000	%	
		p/f_{ck}	0.040		
		Referring to chart 44, SP-16			
		$M_{ux}/f_{ck}D^3$=	0.09		from table
		$M_{uy}/f_{ck}D^3$=	0.09		from table
		$M_{ux,lim}$=	486	kNm	
		$M_{uy,lim}$=	486	kNm	>$M_{u,design}$ **(OK)**
		Check for biaxial bending			
		$P_{uz}=0.45fckAc+0.75fyAsc$	5130	kNm	
		P_u/P_{uz}=	0.2867865	<0.8	
		αn=	1.1446442		
		$(M_{ux}/M_{ux1})^{\alpha n}+(M_{uy}/M_{uy1})^{\alpha n}$=	0.2077695	<1.0	**(Ok)**
		Therefore, required longitudinal steel = 1%			
		Area of steel required=	3600	mm^2	

		Provide 12-20mmΦ bars as longitudinal steel bars.				
		Provided A_{st}=		3771.4285	mm^2	>required (**Ok**)
10.	**IS 456:2000**	**Design for shear**				
		Percentage steel provided in tension=		0.52%	<3%	
		Design shear strength of concrete, τ_c=		0.4964	N/mm^2	
	Cl 40.2.2	Shear strength of members under axial compression				
		$\delta=1+3P_u/(A_g*f_{ck})$		1.4904050	<1.5	
		Actual τ_c=		0.7398370		
		Shear capacity of the section, $Vc_x=Vc_y$=		266.34134	kN	
		End moment	**M_R^{LS}**	**M_R^{LH}**	**M_R^{RS}**	**M_R^{RH}**
	Xaxis	**Moment from ETABS kNm**	0	0	84.842	90.8497
	Yaxis	**Moment from ETABS kNm**	0	0	84.842	84.842
	IS 13920:2016 Cl 7.5	**Shear Force due to Formation of Plastic Hinges**				
		$Vp=1.4(M_R^{LS}+M_R^{RH})/H$ or $1.4(M_R^{RS}+M_R^{LH})/H$				
		V_{px1}	43.633	kN	>V_u from ETABS	
		V_{px2}	40.747	kN	>V_u from ETABS	
		V_{py1}	40.747	kN	>V_u from ETABS	
		V_{py2}	40.747	kN	> V_u from ETABS	
		Design Shear Forces				
		V_{dx}	43.63	kN		
		V_{dy}	40.75	kN		
		Maximum shear stress:				
	IS 456:2000 Table 20	τ_{cmax} for M25=	3.1	N/mm^2		
		$\tau_{vx}=V_{dx}/A$	0.102	N/mm^2		
		$\tau_{vy}=V_{dy}/A$	0.095	N/mm^2		
		τ_{cdx}	0.739	N/mm^2		
		τ_{cdy}	0.739	N/mm^2		
	IS 456:2000 Clause 40.2.3	$\tau_{cd} > \tau_v$ also $\tau_{cd}/2 > \tau_v$ theoretically shear reinforcement is not provided but minimum shear reinforcement is provided.				
	IS 456:2000 Cl 40.3 Cl 26.5.1.6	**Minimum shear reinforcement**				
		$A_{svmin}/S_v=0.4*b/0.87*f_y$	664.73	mm^2/m		

		f_y is taken as 415N/mm^2 as per Cl 26.5.1.6			
11.		**Design of Transverse Reinforcement**			
	IS 13920:2016 Cl.7.4.2	**Diameter of ties**			
		φ_t must be greater than or equal to			
		a)8mm			
	IS 456:2000 Cl 26.5.3.2c	b)1/4*20mm=5mm (we used 20mm for longitudinal bars)			
		Hence adopt ties of 8mm φ	8	mm	
		Area of tie (2 legged)	100.5714	mm2	
		Spacing of the ties			
	IS 13920:2016 Cl.7.4.2	S_v <= 1/2 least lateral dimension of the compression member ie =	300	mm	
	IS 456:2000 Cl 26.5.3.2c	S_v <= 16 times the dia of smallest longitudinal bar	320	mm	
		S_v <=	300	mm	
		Spacing of the ties= Area/Asv_{min}/Sv	151.2971	mm	
		Thus, provide 8mm φ lateral ties @ 150mm c/c in the central part.			
	IS 456:2000	**Spacing between longitudinal bars**			
	Cl.26.5.3.2 b	(600-40*2-8*2-20)/3	161.34	mm>75mm	
		So, additional ties need to be provided requires smaller spacing of stirrups for higher ductility.			
12.	**IS 13920:2016 Cl. 8.1.a)**	**Special confining reinforcement**			
		Special confining reinforcement has to be provided over the length l_o from each joint face towards mid span, and on the either side of any section, where flexural yielding may occur under the effect of earthquake.			
		l_o should be more than			
		1.Lateral dimension of column	600	mm	
		2. 1/6 clear span of member	485.83	mm	
		3. 450mm			
		Provide lo =	**650**	**mm**	

	Cl. 8.1.b)	Spacing should not be more than			
		1. 1/4 of minimum member dimension of column or beam	150	mm	
		2. 6 times diameter of smallest longitudinal rebar	120	mm	
		3. 100mm			
	Cl. 8.1.c)	4.Area A_{sh} of cross section of bar forming links of at least			
		A_{sh}= pi*8^2/4	50.28571	mm2	
		A_{sh}= Maximum of			
		$0.18\ s_v\ h\ f_{ck}/f_{y*}(A_g/A_k-1)$			
		$0.05\ s_v\ h\ f_{ck}/f_y$			
		where,			
		A_k= area of confined concrete core in the rectangular hoop measured to its outer dimension = $(600-80)^2$=270400 mm^2			
		h=(600-40*2)/3	173.33	mm	
		then,			
		$0.18*s_v*173.33*25/415(360000/270400-1)=50.286$ s_v=90.65 mm or, $0.05*s_v*173.33*25/415=50.286$ s_v=101.56 mm			
		Provide spacing 100mm which is not less than 75mm.			
13.	**IS 13920:2016 Cl.7.3.2.1**	**Splicing of vertical bars**			
		Maximum of 50% bars should be spliced at a section and the clear overlap of the spliced section should be more than development length (L_d) of the largest bars			
	IS 456:2000 Cl.26.2.1	$L_d=0.87f_y\varphi/(4\tau_{bd})$	644.732	mm	
		Therefore, provide the overlap of 700 mm in spliced section.			
14.	**IS 13920:2016**	**Hooks on Transverse Reinforcement**			Amendment
		Hook length=8φ or 75mm (larger)			

	Cl.7.4.1	or, Hook length=8x8 or 75mm	
		or, Hook length=64mm or 75mm	
		Hook length= 75mm	

• Design of staircase

A staircase is a component of a structure which provides access to different levels of the structure and also serves as an emergency escape route in case of a fire. We shall learn how to design a staircase manually. To learn how to design, we need to know different components of it.

There are various types of staircases used in structure based on their functionality and space available for construction.

The following some of the important types of stairs generally used in structures:

- Geometric Stair
- Quarter turn stair
- Bifurcated stair
- Circular stair
- Spiral stair
- Open newel stair
- Dog legged stair

1. Geometrical properties

- Floor height = 3.465 m
- Tread width (T) = 300 mm
- Riser height (R) = 157.5 mm
- Number of riser = 22 (2*11)
- Number of tread = 2*10 (for two storey)
- Length of flights in each floor = 3.00m
- Width of flight = 2.25m

$$\text{Cos}\alpha = \frac{3}{(3^2+1.7325^2)^{\frac{1}{2}}} = 0.8659$$

Some assumed data

- Depth of waist slab (D) = 150 mm
- Clear cover (cc) = 20mm
- Diameter of bar (ϕ) = 10mm
- Effective depth = 150-20-10/2 = 125 mm

2. Effective Span

Flight: c/c of supports = 3.5 m

Landing 1: Lesser of 3 m and (2.725+0.125) m =2.85 m

Landing 2: Clear span between supports =2.225 m

3. Load Calculation

Load calculation for landing

Dead load from slab = 0.150m * 25kN/m^3

= 3.75kN/m^2

Floor finish = 1kN/m2

Total dead load = 3.75+1 = 4.75 kN/m^2

Imposed load = 4kN/m2

Load calculation for going

Wt. of waist slab on horizontal plane = 25*0.15*$\frac{338.83}{300}$ = 4.24 kN/m^2

Wt. of steps = 25*0.5*0.1575 = 1.97 kN/m^2

Floor finish = 1 kN/m^2

Total dead load = 4.24+1.97+1 = 7.21 kN/m^2

Imposed load = 4 kN/m^2

4. Analysis

Consider 1m width of flight.

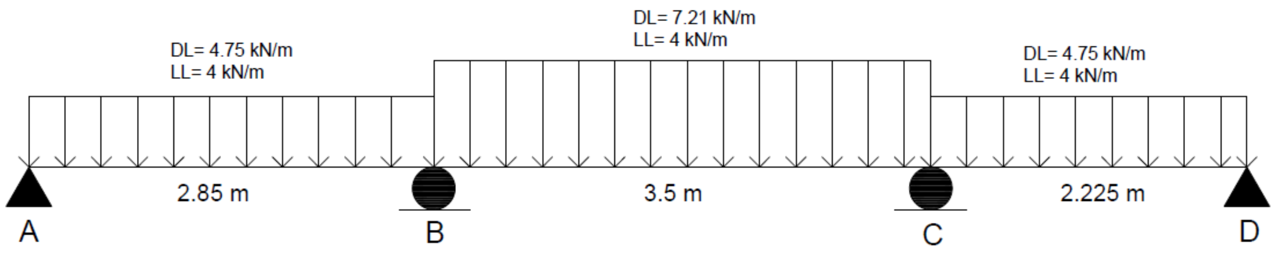


Figure 11: Load analysis of staircase

Moment in span AB = $1.5*(\frac{1}{12}*4.75*2.85^2 + \frac{1}{10}*4*2.85^2)$ = 9.696 kNm

Moment in span BC = $1.5*(\frac{1}{16}*7.21*3.5^2 + \frac{1}{12}*4*3.5^2)$ = 14.40 kNm

Moment at B = Avg. of $1.5*(-\frac{1}{10}*4.75*2.85^2 - \frac{1}{9}*4*2.85^2)$ and $1.5*(-\frac{1}{10}*7.21*3.5^2$ $-\frac{1}{9}*4*3.5^2)$

= Avg. of -11.20 and -21.42

= - 16.31 kNm

Moment at C = Avg. of $1.5*(-\frac{1}{10}*4.75*2.225^2 - \frac{1}{9}*4*2.225^2)$

and $1.5*(-\frac{1}{10}*7.21*3.5^2 - \frac{1}{9}*4*3.5^2)$

= Avg. of -6.83 and -21.42

= - 14.12 kNm

$M_{u,lim} = 0.133*f_{ck}*b*d^2$

$= 0.133*25*1000*125^2 = 51.95$ kNm > M_{max} (16.31 kNm)

Hence section can be designed as singly reinforced.

5. Design of reinforcement

$Mu = 0.87 f_y Astd\,(1-\frac{Astfy}{bdfck})$

$16.31*10^6 = 0.87*500*Ast*125*(1-\frac{Ast*500}{1000*125*25})$

Solving, Ast = 315.92 mm^2

Use 10mm Ø bars then,

Spacing = $\frac{\pi*10^2/4}{315.92}*1000 = 248.61$ mm

Provide 10 dia bar at 225 c/c.

Ast, provided = $\frac{\pi*10^2/4}{225}*1000 = 349.06$ mm2

Distribution steel = 0.12% of bD = 180 mm^2

Use 8mm Ø bars then,

Spacing = $\frac{\pi*8^2/4}{180}*1000 = 279.25$ mm

Provide 8 dia bar at 250 c/c.

6. Check for Shear

Vu = 1.5*(0.6*7.21*3.5+0.6*4*3.5) = 35.31 kN

$\tau_v = \frac{V_u}{bd} = \frac{35.31*1000}{1000*125} = 0.282$ N/mm2

$Pt = \frac{349.06}{1000*125}*100 = 0.28$ %

From **Table 19 IS 456:2000** for Pt = 0.28% and M25 Concrete;

τ_c= 0.37 N/mm2

Again, from **Clause 40.2.1.1 of IS 456** for slab thickness ≤ 150 mm; k = 1.30

Therefore, Permissible shear stress (τ'c) = k× τc = 1.30×0.37 = 0.481 N/mm2

Also from **Table 20 of IS 456**; τc, max = 3.1 N/mm2 (for M25) i.e. τv < τc' < τc,max, hence shear capacity is sufficient.

7. Check for deflection

$\frac{l}{d}$= $\frac{3500}{125}$ = 28

α=26 (For Continuous Slab; **IS456:2000 Cl. 23.2.1**)

β=1 (β = $\frac{Span}{10}$ For span > 10m, 1 Otherwise)

fs = 0.58×fy×(A_{st}required/A_{st}Provided) = 0.58×500 × (315.92/349.06) = 262.5

Hence for fs = 262.5 and %A_{st}Provided = 0.28% From **Fig. 4 IS456:2000;** γ= 1.4

δ=1 (From **Fig. 5 IS456:2000)**

λ=1 (From **Fig. 6 IS456:2000)**

$\alpha\beta\gamma\delta\lambda$= 26×1×1.4×1×1 = 36.4

Hence, $\frac{l}{d} \leq \alpha\beta\gamma\delta\lambda$ hence the design is safe in deflection control criterion. **OK**

8. Check for development length

The development length (Ld) is given by **(IS 456: 2000, Cl. 26.2)**;

Ld = $\frac{0.87 f_y \emptyset}{4\tau bd}$ = $\frac{0.87*500*10}{4*1.6*1.4}$ = 485.49 mm

Also, from **IS456:2000 Cl. 26.2.3.3**

Ld < 1.3 $\frac{M_1}{V_u}$ + Lo

Here M_1 = moment of resistant of the section assuming all the reinforcement at the section to be stressed to fd.

M_1 = 0.87×fy×Ast provided × d×(1−$\frac{\text{Ast provided}*\text{fy}}{\text{bdfck}}$)

= 0.87 X 500 X 349.06 X 125 X (1−$\frac{349.06*500}{1000*125*25}$)

= 17.92 kN-m

Assuming L_o=0,

1.3 $\frac{M_1}{V_u}$ + L_o = $\frac{1.3*17.92*10^3}{35.31}$ + 0 = 659.75 mm > L_d (Ok)

• Design of basement wall

Basement wall is constructed to retain the earth and to prevent moisture from seeping into the building. Since the basement wall is supported by the mat foundation, the stability is ensured and the design of the basement wall is limited to the safe design of vertical stem.

Basement walls are exterior walls of underground structures (tunnels and other earth sheltered buildings), or retaining walls must resist lateral earth pressure as well as additional pressure due to other type of loading. Basement walls carry lateral earth pressure generally as vertical slabs supported by floor framing at the basement level and upper floor level. The axial forces in the floor structures are, in turn, either resisted by shear walls or balanced by the lateral earth pressure coming from the opposite side of the building.

The basement wall is designed as the cantilever wall with the fixity provided by the mat foundation.

Basement wall is idealized as propped cantilever wall assuming that basement wall is rigidly fixed as base and supported hingedly at floor slab.

Modeling of Basement wall

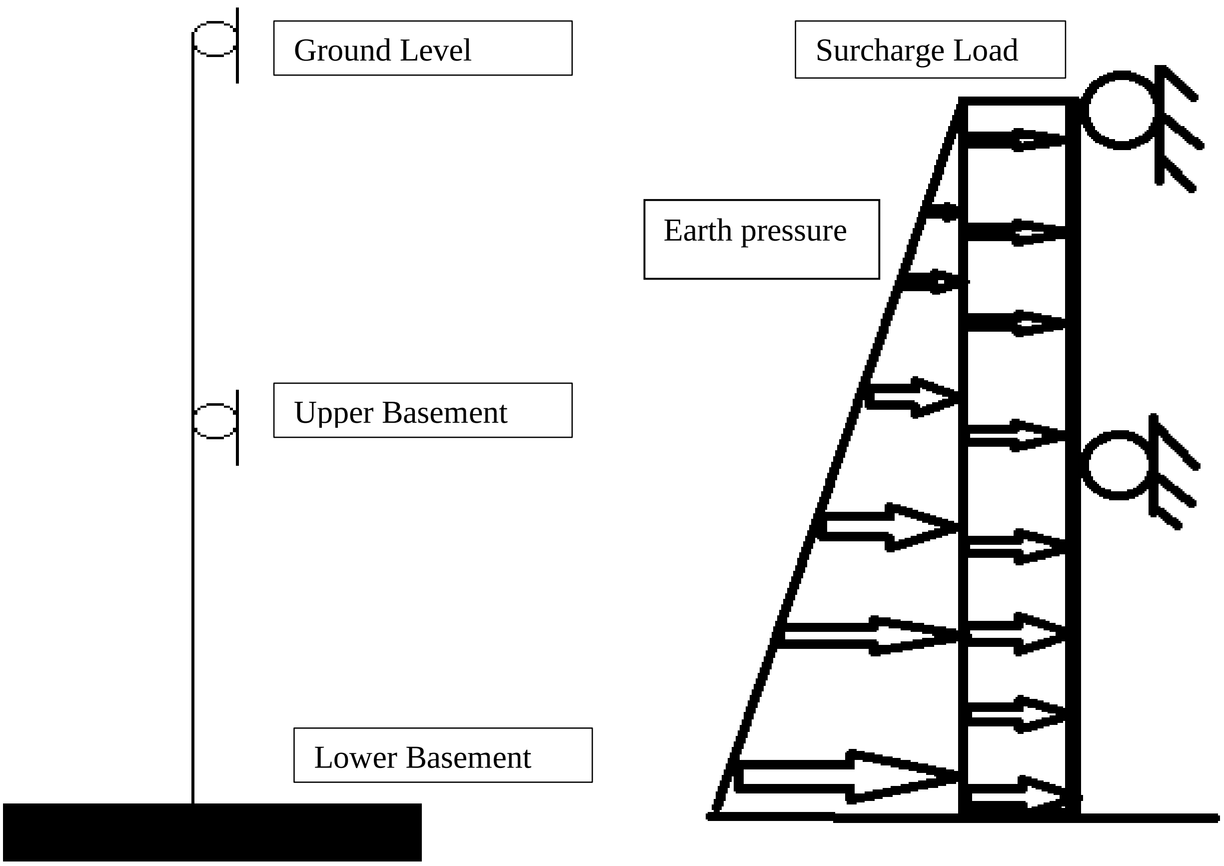


Figure 12: Idealization of basement

Basement walls are commonly used in building construction to provide support and stability to the structure. These walls are designed to resist various types of loads such as earth pressure, surcharge loads, and lateral loads due to wind or seismic forces. In this case, the surcharge load due to vehicular movement and earth pressure is considered as the main load acting on the basement wall.

To design the basement wall, out of plane bending moment is considered as the critical design parameter. The wall is analyzed and designed for moment and shear from earth pressure loads and surcharge loads. The loading diagram is generated considering the surcharge load and earth pressure.

The pressure at the base of the basement wall can be calculated using the Rankine's theory of earth pressure. The Rankine's theory assumes that the soil exerts a pressure perpendicular to the wall, which can be calculated using the following formula:

$$P = Ka * \gamma * h$$

Unit Weight of soil =20 kN/m^2

Assuming at active pressure,

K_a=(1-sin(Ø))/(1+sin(Ø))=0.333

Assuming angle of internal friction of soil=30^0

K_a=0.33

P=k_a* gamma*h

P=6.6*h

At the base of basement wall, P=45.74kN/m2

The basement wall is modeled and analyzed as beam of 1m width,

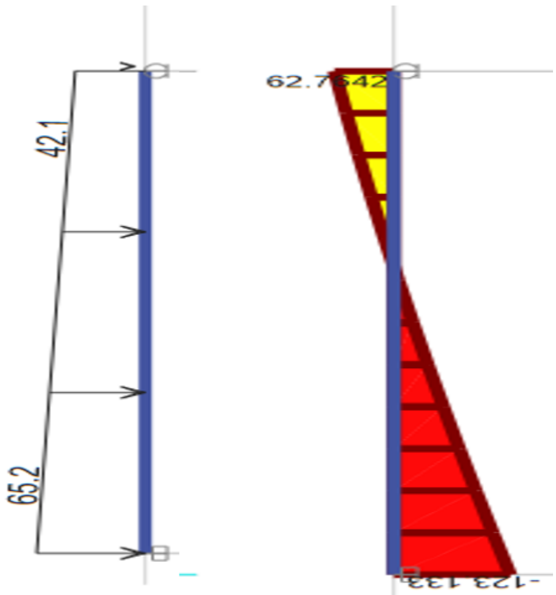


Figure 13: Loading and Shear force diagram of Basement wall

Maximum positive moment = 43.4kNm

Maximum Negative moment = 84.48kNm

Shear Force = 123.12kNm

Maximum Deflection obtained from ETABS = 2.489mm

So Design M(+) = 1.5M(+)

= 65.1kNm

Design M(-) = 122.22kNm

Design V = 184.68KN

As per clause 32.3.2 walls subjected to horizontal forces perpendicular to the wall and for which the design axial load doesn't exceed 0.04fckAg, shall be designed as slabs

$M_{u,lim}$=0.138*fck*b*d^2

d=188mm

clear cover=25mm

assuming Ø16mm is provided

overall depth = 188 + 25 + 8 = 221mm

provide depth **D = 225mm**

d = 225-33 = 192mm

Now Reinforcement is provided at outer face for negative moment and inner side for positive moment.

Calculation of main(vertical) Reinforcement

Table 31: Vertical Rebar requirement for basement

Location	Moment	Mu_lim=0.138fck*bd^2	Reb Req
Outer face	122.22	127.18	2180.19478
Inner Face	65.1	127.18	1032.669343

Checks

Table 32: Design checks for basement

Min.Rebar	0.12%bD	270	Okay	**[IS 456:2000, Cl. 32.5.a.1,26.5.2.1]**
D_{max}	D/8	28.125	Okay	
Max.Spacing	3D or 300	675 or 300	Okay	

The minimum thickness of wall should not be less than 100 mm.[clause 11.2.1 sp 34]

Thickness provided(225mm)>100mm **Okay**

Table 33: Design summary basement

Design Summary		
Location	Provided	Rebar(%)
Outer face	Ø16@85	1.1%
Inner Face	Ø16@190	0.6%

As per clause 11.2.1.1 IS SP 34, following consideration should be taken regarding minimum reinforcement:

Reinforcement -The minimum reinforcement for walls shall be provided as given below:

a. The minimum ratio of vertical reinforcement to gross concrete area shall be 0.004 (irrespective of type and grade of steel).

b. Vertical reinforcement shall be spaced not farther apart than three times the wall thickness or 450 mm, whichever is less.

c. The minimum ratio of horizontal reinforcement to gross concrete area shall be:

- **i.** 0.0020 for deformed bars not larger than 16 mm in diameter and with a characteristic strength of 415 N/mm* or greater.
- **ii.** 0.0025 for other types of bars.

Reinforcement Horizontal:

a) Area of Hor. Steel reinforcement 0.0020*bD=450mm^2

Reinforcement is distributed considering temperature changes in front face

Hor reinforcement in Outer face=450*2/3

=300mm^2

In inner face, A=150mm2

Minimum spacing of Hor. Rebars=3D=675mm or 300mm

Table 34: Horizontal rebar summary of basement

Min.Rebar	0.0020*b*D	450	
Location	**Reb Req mm^2**	**Spacing mm**	**Provided mm**
Outer face	300	255	255
Inner Face	150	520	300
Max.Spacing	3D or 300	675 or 300	
Design Summary			
Location		**Provided**	
Outer face		Ø10@255	
Inner Face		Ø10@300	

Check for shear:

V_u	184.68
τ_v	0.961875
Rebar(%)	1.05
K	1.15
τ_c	0.656113
k τ_c	0.75453

Since Tv >kTc shear reinforcement should be provided.

Design for Shear Reinforcement

$V_{us}=V_u$-Tc*bd **[Clause 40.4 Is 456:2000]**

$$V_{us} = \frac{0.87 f_y A_{sv}}{S_v}$$

Table 35: Shear reinforcement in basement

V_{us}	39810.24 N	$Ø_{shear}$	10mm
Spacing	130	A_{sv}	78.53981634

So provide shear reinforcement of Ø10@130 mm of spacing.

According to the calculations, the shear stress at the support sections is greater than the shear capacity of concrete, while the shear stress at the middle span sections is less than the shear capacity of concrete. Therefore, shear reinforcement will be provided at the support sections only. The shear reinforcement will consist of shear stirrups of Ø10@130 mm. The stirrups will be provided at a spacing of 130 mm and will extend up to 500 mm from the lower basement support on both sides of the structure. The shear stirrups will be placed perpendicular to the longitudinal reinforcement and will be adequately anchored into the support columns. So provide shear stirrups of Ø10@130 mm upto 500 mm from lower basement support as shown in fig ***Figure 14***:

Figure 14 : Region of shear Reinforcement in basement

• Design of shear wall

Shear wall, also called structural wall, is an important structural unit for a multistoried building. For tall buildings, it is necessary to provide sufficient stiffness to resist the lateral loads caused by earthquake and limit the drift within the codal requirements. When buildings are not provided with such adequate stiffness, excessive vibrations and sway can occur during seismic event, or non-structural components can fail, which is uncomfortable to the occupants. From structural point of view, it is most beneficial to provide shear wall along the outer periphery and in symmetrical position so as to provide maximum resistance against torsional forces. However, shear walls are also usually provided around lift wells, stairwells, utility shafts because those locations are convenient from architectural point of view to position the walls. A very important property of shear wall is that, it should have good ductility under reversible and repeated loads.

The design of shear wall involves providing of adequate cross-section and reinforcements to resist bending (in plane and out of plane), shear (in plane and out of plane), and Axial forces due to gravity and lateral loads. The design forces are taken from ETABS analysis and design is done per the requirements of IS 13920-2016. In addition to the requirements in codes for design of in-plane moment and shear force, we have also checked the reinforcement and redesigned them if necessary for out of plane moment, and axial force and checked the concrete capacity for out of plane shear force. Since walls are thin and deep they are subjected to substantial axial forces too especially at lower stories.

Design of wall located in between grids 3 and 4 along grid A at lower basement level is shown.

a. Geometry of Design loads

Table 36: Design parameter for shear walls

Length of wall L_w	1500mm
Thickness of wall t_w	300mm
Floor height, h	3465mm
$\frac{h}{l_w}=\frac{24255}{1500}$	16.17>2. So the wall is slender as per Cl 10.1.4 of IS 13920:2016
Axial Force (compressive) P_-	3600 kN
Axial Force (tensile) P_+	2210 kN
In-plane Moment M_3	1527 kN-m
Out of Plane Moment M_2	22.11 kN-m
In Plane Shear V_2	452.01 kN
Out of Plane Shear V_3	26.27 kN

b. General Requirements

(As per IS 13920-2016 Cl. 10.1)

c. Wall Thickness

The minimum thickness of special shear wall (i.e. walls that follow ductile detailing provision) is 150 mm in general case and not less than 300 mm for coupled shear wall. The minimum thickness is necessary to confirm to the fire resistance requirements and also to avoid thin sections. Thin walls are susceptible to instability (buckling) at regions of high compressive Strain. (IS 13920-2016 Cl. 10.1.2)

d. Reinforcement Limits

The minimum amount of vertical and horizontal reinforcement should be 0.25 per cent of the gross concrete area. And this reinforcement should be distributed uniformly across the cross section of the wall. Uniform distribution helps to control the width of inclined cracks that are caused by shear.

When the factored shear stress in the wall exceeds 0.25 $\sqrt{fck}$ MPa or when the thickness of the wall exceeds 200 mm, reinforcements should be provided in two curtains, each having bars running in the longitudinal and transverse directions. The use of two curtains of reinforcement will reduces fragmentation and premature deterioration of the concrete under cycling loading into the inelastic range. It is also mentioned that, all vertical bars shall be contained within horizontal steel bars. (IS 13920, Cl. 10.1.7)

Also, the diameter of bars used in any part of the wall should not exceed one-tenth of the thickness of the wall. The helps to prevent the use of very large diameter bars in thin wall sections. So, for our wall, the maximum size of bar is $\frac{300}{10}$= 30mm.(IS 13920, Cl. 10.1.8)

The maximum spacing of reinforcement in either direction should not exceed the smallest of $l_w/5$, $3t_w$, and 450 mm (IS 13920, Cl. 10.1.9)

i.e. $\frac{1500}{5} = 300mm, 3\ x\ 300 = 900mm\ and\ 450mm$. So provided spacing should be less than 300mm.

e. Provided Reinforcements

Table 37: Reinforcement detail in shear walls

Reinforcement	Value	Remarks
Vertical Bar φ	28 mm	<30mm
Horizontal Bar φ	10mm	

Vertical Bar Spacing	110mm	<300 mm
Horizontal Bar Spacing	200mm	<300 mm
Horizontal Reinforcement %	0.26%	≥0.25%
Vertical Reinforcement %	3.73%	≥0.25%

f. Design and Checks

i. Design for In-Plane Moment

The bending strength of thin sections of rectangular structural wall containing evenly distributed vertical reinforcement and subjected to axial and lateral load can be obtained using the same assumptions as for columns. The moment capacity of the slender wall with uniformly distributed vertical reinforcements is given in IS 13920 Annex A (As per Clause 10.3.1. Two equations are given for calculating the flexural strength of the section, depending on the position of neutral axis.

Critical non-dimensional depth of neutral axis:

$$\frac{x_u{}^*}{l_w} = \frac{0.0035}{0.0035 + 0.002 + \frac{0.87 f_y}{E_s}} = 0.479$$

$$\varphi = \frac{0.87\, f_y \rho}{f_{ck}} = \frac{0.87 * 415 * 0.0373}{25} = 0.538$$

Where, ρ is percentage of vertical reinforcement

$$\lambda = \frac{P_u}{f_{ck} t_w l_w} = \frac{3600\ X\ 10^3}{25\ X\ 300\ X\ 1500} = 0.32$$

where P_u is the compressive axial load

So, non-dimensional depth of neutral axis for Case I:

$$\frac{x_u}{l_w} = \frac{\varphi\ +\ \lambda}{2\varphi\ +\ 0.36} = 0.597$$

Hence, Case I is not applicable.

For case II:

$$\beta = \frac{0.002 + 0.87\ f_y/E_s}{0.0035} = \frac{0.002 + 0.87 X 415/200000}{0.0035} = 1.087$$

$$\alpha_1 = 0.36 + \ \varphi\left(1 - \frac{\beta}{2} - \frac{1}{2\beta}\right) = 0.358$$

$$\alpha_2 = 0.15 + \frac{\varphi}{2}\left(1 - \ \beta + \frac{\beta^3}{3} - \frac{1}{3\beta}\right) = 0.15$$

$$\alpha_4 = \frac{\varphi}{\beta} - \lambda = 0.175$$

$$\alpha_5 = \frac{\varphi}{2\,\beta} = 0.247$$

Now for $\frac{x_u}{l_w}$, solving : $\alpha_1\,(\frac{x_u}{l_w})^2 + \alpha_4\left(\frac{x_u}{l_w}\right) - \alpha_5 = 0$

We get, $\frac{x_u}{l_w} = 0.621$

$$\alpha_3 = \frac{\varphi}{6\,\beta}\left(\frac{1}{\frac{x_u}{l_w}} - 3\right) = -0.115$$

So,

$$\frac{M_u}{f_{ck} t_w {L_w}^2} = \alpha_1\left(\frac{x_u}{l_w}\right) - \alpha_2\left(\frac{x_u}{l_w}\right)^2 - \alpha_3 - \frac{\lambda}{2}$$

$$\frac{M_u}{f_{ck} t_w {L_w}^2} = 0.119$$

M_u=2016.07 kNm >M_3=1527kNm (OK)

Similarly, for tensile axial force P_t = $2210kN$, M_u=2227.068kNm >M_3=1527kNm (OK)

ii. Design for In-Plane Shear

The horizontal reinforcement is provided from shear design. The shear design is done as per Cl 40 of IS 456:2000.

Nominal Shear stress on wall is:

$$\tau_v = \frac{V_u}{t_w d_w} = \frac{452.01\ X\ 10^3}{300\ X\ 0.8\ X1500} = 1.255 N/mm^2$$

Where, V_u is factored shear force V_2 = 452.01 kN

d_w is effective depth = $0.8 l_w$

Shear strength of concrete, $\tau_{c,max}$=3.1N/mm^2 (Table 20, IS 456:2000)

Design shear strength of concrete, τ_c=0.92 N/mm^2 (Table 19, IS 456:2000)

If $\tau_v > \tau_{c,max}$ then wall needs to be re-designed. However, here

$\tau_c < \tau_v < \tau_{c,max}$

So, total capacity of shear reinforcement and concrete is checked from:

$$V_u = \frac{0.87 f_y A_{sv} d}{s_v} + \tau_c t_w l_w = 753.52 \text{ kN} > V_2 = 360.4 \text{ kN (OK)}$$

iii. Check for Out of Plane Moment

Out of plane moment capacity is calculated as done in beam taking width and overall depth corresponding to clear length of wall and thickness of wall respectively.

Overall Depth (D) = $t_w = 300mm$

Effective Depth (d) = D – clear cover – Diameter of horizontal bars – Diameter of vertical bars/2

$$d = 300 - 20 - 10 - \frac{28}{2} = 256\text{mm}$$

For, 28Φbars at 110 mm c/c, $A_{st} = \pi \times \frac{28^2}{4} \times \frac{1000}{110}$ mm^2/m= 5598.74 mm^2/m

Moment Capacity= $0.87\ f_y\ A_{st}\ (d - 0.416 \times \frac{0.87\ fy\ Ast}{0.36\ fck\ bd})$

So, M=516.65 kN-m >M_2 =22.11 kN-m (OK)

iv. Check for Axial Force

For tension force

$P_t = 2210kN$

$\%A_{st} = 3.73\%$

So, $A_{st} = \%A_{st} \times l_w t_w = 16{,}785mm^2$

Tension force capacity

$0.87 f_y A_{st} = 0.87 \times 415 \times 16785 = 6060.2kN > P_t = 2210$(OK)

v. For Compressive Force

Compressive Force P_c= 3600kN

Area of Concrete $A_c = l_w t_w = 450{,}000mm^2$

So, Compressive force capacity= $0.4 f_{ck} A_c + 0.67 f_y A_{st}$

$= 0.4 \times 25 \times 450000 + 0.67 \times 415 \times 16{,}785$

= 9167.1kN> P_c =3600kN (OK)

vi. Check for Out of Plane Shear

$V_3 = 26.27\ kN < V_c = \tau_c t_w l_w = 129.6$(OK)

g. Boundary elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcements. According to IS 13920 Clause 10.4.1 boundary elements should beprovided along the vertical boundaries of walls, when the extreme fiber compressive stress in walls exceeds $0.2f_{ck}$ due to factored gravity forces plus earthquake forces. The boundary elements can be discontinued at elevations where the extreme fibre compressive stress becomes less than 0.15 f_{ck}.

The stress at extreme compressive fiber is given by:

$$\sigma = \frac{P_u}{A_g} + \frac{M_u(\frac{l_w}{2})}{I}$$

Where, P_u is the factored axial load,

M_u is the factored moment acting on the wall (in plane moment),

A_g is the gross area of the wall

$I = \frac{t_w l_w{}^3}{12}$ is the moment of inertial of wall

Here,

$I = 8.4375 \times 10^{10}$ mm^4; A_g=450000mm^2

So stress, σ=18.46 N/mm^2 > $0.2f_{ck}$=5 N/mm^2

At one end, the shear wall is connected to column so the column acts as the boundary elementof the wall and at the other end boundary element of thickness $t_w = 300mm$ and length = 600mm is provided.

(As per NBC 105 Annex A Cl. 5.3, the length of boundary zone in each side shall be the maximum of 2 times the wall thickness and 0.2 times the wall length i.e., maximum of $2 \times 300 = 600$& $0.2 \times 1500 = 300mm$)

The reinforcement of boundary element should not be less than 0.8 percent and not greater than 6 percent (practical upper limit is 4%). Area of the special confining reinforcement is providedas per the NBC 105 Annex A Cl. 5.3 and IS 13920 Cl. 10.4.4. During a severe earthquake, boundary elements may be subjected to stress reversals, Hence, they have to confined adequately to sustain the cyclic loading without a large degradation in strength.

Spacing of reinforcement (s_v) should not be more than:

a) 1/3rd of minimum member dimension of boundary element = 100mm
b) 6 times Diameter of smallest longitudinal bars = $6 \times 28 = 168mm$
c) 100 mm

. So, taking $s_v = 100mm$

From Cl. 10.4.4 area of reinforcement is given by

$A_{sh} = 0.05\ s_v\ h \frac{f_{ck}}{f_y} = 0.05 \text{ X } 100 \text{ X } 300 \text{ X} \frac{25}{415} = 90.36 mm^2$

Where, h is longer dimension of rectangular confining link measured to its outer face whichdoes not exceed 300 mm (NBC 105 Annex A, Cl. 4.3 c ii)

So, provide 2 legged 8 mm @ 100 c/c.

The vertical reinforcement provided in shear wall is sufficient to resist extra axial force. So extra vertical reinforcement is not provided in the boundary element. If extra vertical reinforcement was required, it can be provided as in lift shear wall as shown in upcoming section.

h. Design Summary

Similarly, the design reinforcement for all the shear walls is obtained. The summary of the design forces and provided reinforcement are shown for all outer shear walls in each floor.

Table 38: Design of shear wall 43-A

Location	Grid 43-A
Length of wall , L_w	1500mm
Thickness of wall, t_w	300mm
Pier id	P4

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor
Vertical Reinforcement Dia. Φv (mm)	28	25	20	16	16	12	12
Vertical Reinforcement spacing (mm)	110	110	110	110	110	110	110

Horizontal Reinforcement Dia. Φh (mm)	10	10	10	10	10	10	10
Horizontal Reinforcement spacing (mm)	200	200	200	200	200	200	200
Boundary Element tie spacing (sv) (mm)	100	100	100	90	80	70	Discontinued as σ<0.15fck
Boundary element Ties dia (mm)	8	8	8	8	8	8	
Boundary Element size (mm x mm)	600 X 300	600 X 300	600 X 300	600 X 300	600 X 300	600 X 300	

Force/Moment		M3 (kNm)	M2 (kNm)	V2 (kN)	V3 (kN)	Pc (kN)	Pt (kN)
Lower Basement	Design Forces	1527	22.11	452.01	26.27	3600	2210
	Provided Capacity	2020.474	516.6555	753.5292	413.2476	9169.361	6063.2
	Ratio	1.323166	23.3675	1.667063	15.73078	2.547045	2.743529
Upper Basement	Design Forces	784.49	27.82	455.78	14.61	3476	1957
	Provided Capacity	1551.189	322.8763	725.3605	385.0789	7382.615	3743.098
	Ratio	1.977322	11.6059	1.591471	26.35721	2.123882	1.912671
Ground Floor	Design Forces	566.38	32.89	380.31	16.97	2461	1154
	Provided Capacity	1422.377	217.8732	677.5497	337.2681	6429.685	2505.71
	Ratio	2.511348	6.6243	1.781572	19.87437	2.612631	2.171326
1st Floor	Design Forces	421.35	31.83	335.28	17.31	1813	718.94
	Provided Capacity	1349.649	172.8275	650.6345	310.3529	6024.689	1979.82
	Ratio	3.203154	5.429704	1.940571	17.92911	3.32305	2.753805
2nd Floor	Design Forces	332.35	34.15	287.48	21.45	1393.2	535.44
	Provided Capacity	1220.463	132.8403	621.4996	281.218	5667.34	1515.8
	Ratio	3.672223	3.889905	2.161888	13.11039	4.067858	2.830943
3rd Floor	Design Forces	265.25	26.82	276.57	11.46	1065	454.03
	Provided Capacity	1049.068	97.97698	589.9749	249.6933	5357.638	1113.649
	Ratio	3.955018	3.653131	2.133185	21.78825	5.030646	2.452809
4th Floor	Design Forces	232.58	26.18	285.89	21.52	672.06	383.15
	Provided Capacity	946.3275	97.97698	589.9749	249.6933	5357.638	1113.649
	Ratio	4.068826	3.742436	2.063643	11.60285	7.971963	2.906561

Table 39: Design of shear wall 45-A

Location	Grid 45-A

Length of wall ,L_w	1500mm
Thickness of wall, t_w	300mm
Pier id	P5

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor
Vertical Reinforcement Dia. Φv (mm)	28	25	20	16	16	12	12
Vertical Reinforcement spacing (mm)	110	110	110	110	110	110	110
Horizontal Reinforcement Dia. Φh (mm)	10	10	10	10	10	10	10
Horizontal Reinforcement spacing (mm)	200	200	200	200	200	200	200
Boundary Element tie spacing (sv) (mm)	100	100	100	90	80	70	70
Boundary element Ties dia (mm)	8	8	8	8	8	8	8
Boundary Element size (mm x mm)	600 X 300	600 X 300	600 X 300	600 X 300	600 X 300	600 X 300	600 X 300

Force/Moment		M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c (kN)	P_t (kN)
Lower Basement	Design Forces	1653.16	23.31	267.89	25.55	3513.12	2278.26
	Provided Capacity	2035.30	516.655	753.529	413.247	9169.36	6063.2
	Ratio	1.23116	22.1645	2.81283	16.1740	2.61003	2.66132
Upper Basement	Design Forces	758.28	30.21	418.03	16.18	3390	2000.99
	Provided Capacity	1564.86	322.876	725.360	385.078	7382.61	3743.09
	Ratio	2.06369	10.6877	1.73518	23.7996	2.17776	1.87062
Ground Floor	Design Forces	534.44	35.54	339.91	19.56	2374.7	1199.29
	Provided Capacity	1430.08	217.873	677.549	337.268	6429.68	2505.71
	Ratio	2.67585	6.13036	1.99332	17.2427	2.70757	2.08932
1st Floor	Design Forces	399.6	33.89	292.85	18.73	1736.28	758.39
	Provided Capacity	1351.97	172.827	650.634	310.352	6024.68	1979.82
	Ratio	3.3833	5.09966	2.22173	16.5698	3.46988	2.61055
2nd Floor	Design Forces	303.44	36.36	239.81	22.35	1343.09	557.96
	Provided Capacity	1215.71	132.840	621.499	281.218	5667.34	1515.8
	Ratio	4.0064	3.65347	2.5916	12.5824	4.21962	2.71668
3rd Floor	Design Forces	348.55	28.84	209.16	13.31	1050.86	456.02

	Provided Capacity	1046.03	97.9769	589.974	249.693	5357.63	1113.64
	Ratio	3.00109	3.39726	2.82068	18.7598	5.09833	2.44210
4th Floor	Design Forces	402.16	27.41	160.63	23.64	674.36	376.99
	Provided Capacity	947.039	97.9769	589.974	249.693	5357.63	1113.64
	Ratio	2.35483	3.57449	3.67288	10.5623	7.94477	2.95404

Table 40 :Design of shear wall 65-F

Location	Grid 65-F
Length of wall ,L_w	3000mm
Thickness of wall, t_w	200mm
Pier id	P1

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor
Vertical Reinforcement Dia. Φv (mm)	20	20	16	12	12	12	12
Vertical Reinforcement spacing (mm)	90	90	90	90	90	90	90
Horizontal Reinforcement Dia. Φh (mm)	12	12	12	12	12	12	12
Horizontal Reinforcement spacing (mm)	200	200	200	200	200	200	200
Boundary Element tie spacing (sv) (mm)	60	60	60	60	60	60	Discontinued as σ<0.15fck
Boundary element Ties dia (mm)	8	8	8	8	8	8	
Boundary Element size (mm x mm)	600 X 200	600 X 200	600 X 200	600 X 200	600 X 200	600 X 200	

Force/Moment		M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c (kN)	P_t (kN)
Lower Basement	Design Forces	4448.6	50.29	1137.73	35.725	4751.22	2355.94
	Provided Capacity	5181.591	198.6631	1531.008	550.9968	11823.47	7561.814
	Ratio	1.164769	3.95035	1.345669	15.42328	2.488511	3.20968
Upper Basement	Design Forces	3509.39	65.23	1025.06	47.24	3171.5	784.83
	Provided Capacity	5036.262	162.0114	1526.722	546.7111	10717.01	6125.069
	Ratio	1.435082	2.483694	1.489398	11.57305	3.37916	7.804326
Ground Floor	Design Forces	2334.8	60.33	799.5	44.32	2199.5	125.62

	Provided Capacity	4133.231	99.31576	1445.875	465.8642	8853.498	3705.289
	Ratio	1.770272	1.646208	1.808474	10.51138	4.025232	29.49601
1st Floor	Design Forces	1599.4	62.55	665.22	46.98	1666.99	0
	Provided Capacity	3614.88	73.44209	1398.423	418.4115	8096.448	2722.253
	Ratio	2.260147	1.174134	2.102196	8.906162	4.856926	-
2nd Floor	Design Forces	1131.2	63.11	545.32	47.99	1420.22	169.25
	Provided Capacity	3569.182	73.44209	1398.423	418.4115	8096.448	2722.253
	Ratio	3.155218	1.163716	2.564407	8.718722	5.70084	16.08421
3rd Floor	Design Forces	889.3	63.54	430.57	49.37	1188.7	244.95
	Provided Capacity	3509.243	73.44209	1398.423	418.4115	8096.448	2722.253
	Ratio	3.946074	1.15584	3.24784	8.475015	6.811178	11.1135
4th Floor	Design Forces	686.1	104.08	439.9	86.98	803.3	206.3
	Provided Capacity	3372.825	73.44209	1398.423	418.4115	8096.448	2722.253
	Ratio	4.915938	0.705631	3.178955	4.810433	10.07898	13.1956

Table 41:Design of shear wall 7-DE

Location	Grid 7-DE
Length of wall ,L_w	3000mm
Thickness of wall, t_w	350mm
Pier id	P2

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor
Vertical Reinforcement Dia. Φv (mm)	30	25	25	20	16	12	12
Vertical Reinforcement spacing (mm)	110	110	110	110	110	110	110
Horizontal Reinforcement Dia. Φh (mm)	12	12	12	12	12	12	12
Horizontal Reinforcement spacing (mm)	150	150	150	150	150	150	150
Boundary Element tie spacing (sv) (mm)	100	10	100	100	80	Discontinued as σ<0.15fck	
Boundary element Ties dia (mm)	8	8	8	8	8		
Boundary Element size (mm x mm)	700 X 350	700 X 350	700 X 350	700 X 350	700 X 350		

Force/Moment		M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c (kN)	P_t (kN)
Lower Basement	Design Forces	6117.6	142.2	1667.7	42.14	11284.6	8629.07

	Provided Capacity	8212.889	702.1697	1857.678	964.2444	21220.47	13920.61
	Ratio	1.342502	4.937902	1.113916	22.88193	1.880481	1.613223
Upper Basement	Design Forces	4277	121.4	1662.2	39.4	9159.3	6516.32
	Provided Capacity	7596.05	531.0493	2256.344	949.6622	18552.26	10455.93
	Ratio	1.776023	4.374377	1.357444	24.1031	2.025511	1.604575
Ground Floor	Design Forces	3089.67	106.1	1431	40.7	6634.89	4374.3
	Provided Capacity	7138.968	382.8093	2161.82	855.1387	16265.23	7486.195
	Ratio	2.310593	3.608004	1.510706	21.01078	2.45147	1.711404
1st Floor	Design Forces	2144.39	97.4	1235.5	40.65	4576.5	2704.48
	Provided Capacity	6442.332	257.9838	2052.268	745.5866	14359.37	5011.42
	Ratio	3.004272	2.648704	1.661083	18.34161	3.137631	1.853007
2nd Floor	Design Forces	1449.09	92.31	1012.67	45.3	3029.04	1557.9
	Provided Capacity	5355.869	157.1	1925.383	618.7016	12834.68	3031.6
	Ratio	3.696023	1.701874	1.901294	13.65787	4.237211	1.945953
3rd Floor	Design Forces	861.1	69.03	787.6	31.4	1672.7	611.1
	Provided Capacity	4189.401	115.7991	1854.659	547.9777	12215.28	2227.298
	Ratio	4.865173	1.677518	2.354824	17.45152	7.302729	3.644735
4th Floor	Design Forces	709.75	114.19	546.01	52.94	787.6	289.55
	Provided Capacity	3529.961	115.7991	1854.659	547.9777	12215.28	2227.298
	Ratio	4.973528	1.014091	3.396749	10.35092	15.50949	7.692274

Table 42:Design of shear wall 7-DC

Location	Grid 7-DC
Length of wall ,L_w	3000mm
Thickness of wall, t_w	350mm
Pier id	P3

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor
Vertical Reinforcement Dia. Φv (mm)	32	25	25	20	16	12	12
Vertical Reinforcement spacing (mm)	110	110	110	110	110	110	110

<table>
<tr><td>Horizontal Reinforcement Dia. Φh (mm)</td><td>12</td><td>12</td><td>12</td><td>12</td><td>12</td><td>12</td><td>12</td></tr>
<tr><td>Horizontal Reinforcement spacing (mm)</td><td>150</td><td>150</td><td>150</td><td>150</td><td>150</td><td>150</td><td>150</td></tr>
<tr><td>Boundary Element tie spacing (sv) (mm)</td><td>100</td><td>100</td><td>100</td><td>100</td><td>80</td><td colspan="2" rowspan="3">Discontinued as σ<0.15fck</td></tr>
<tr><td>Boundary element Ties dia (mm)</td><td>8</td><td>8</td><td>8</td><td>8</td><td>8</td></tr>
<tr><td>Boundary Element size (mm x mm)</td><td>700 X 350</td><td>700 X 350</td><td>700 X 350</td><td>700 X 350</td><td>700 X 350</td></tr>
</table>

Force/Moment		M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c (kN)	P_t (kN)
Lower Basement	Design Forces	6135.5	151.3	1676.6	42.5	11328.5	8615.5
	Provided Capacity	8193.242	702.1697	2270.926	964.2444	21220.47	13920.61
	Ratio	1.335383	4.64091	1.354483	22.6881	1.873193	1.615764
Upper Basement	Design Forces	4291.3	124.8	1656	41.82	9226.4	6492.3
	Provided Capacity	7570.995	531.0493	2256.344	949.6622	18552.26	10455.93
	Ratio	1.764266	4.255203	1.362526	22.70833	2.01078	1.610512
Ground Floor	Design Forces	3091	118	1431.5	48	6691.5	4357.5
	Provided Capacity	7124.921	382.8093	2161.82	855.1387	16265.23	7486.195
	Ratio	2.305054	3.244146	1.510178	17.81539	2.43073	1.718002
1st Floor	Design Forces	2134	105.4	1241.8	46.8	4614	2685.8
	Provided Capacity	6437.879	257.9838	2052.268	745.5866	14359.37	5011.42
	Ratio	3.016813	2.447664	1.652656	15.93134	3.11213	1.865895
2nd Floor	Design Forces	1445.7	104.1	1022.3	52.3	3056.6	1545.9
	Provided Capacity	5363.163	157.1	1925.383	618.7016	12834.68	3031.6
	Ratio	3.709734	1.509126	1.883384	11.82986	4.199006	1.961058
3rd Floor	Design Forces	858.1	76.4	784.7	32.2	1692.1	603.3
	Provided Capacity	4201.939	115.7991	1854.659	547.9777	12215.28	2227.298
	Ratio	4.896794	1.515695	2.363526	17.01794	7.219003	3.691858
4th Floor	Design Forces	744	110.8	537.5	57.2	836.5	260.3
	Provided Capacity	3570.858	115.7991	1854.659	547.9777	12215.28	2227.298

	Ratio	4.799541	1.045118	3.450529	9.58003	14.60284	8.556657

• Design of lift shear wall

Sample Calculation

The lift shear wall design of wall spanning along Y-axis at lower basement level is shown below.

Table 43: Design parameter for lift shear wall

Length of wall	4475 mm
Thickness of wall, tw	400 mm
Pier id	L1
Axial Force (compressive) P_-	16974.9kN
Axial Force (tensile) P_+	11335.23 kN
Inplane Moment M_3	11407.6kN-m
Out of Plane Moment M_2	506.2 kN-m
In Plane Shear V_2	4038.1kN
Out of Plane Shear V_3	410.63kN

The Vertical reinforcement, Horizontal reinforcement and Horizontal reinforcement in boundary element is designed as in Shear wall design section above and the result is shown below as:

Table 44: Reinforcement detail in Lift shear wall

Vertical Reinforcement Dia. Φv (mm)	32
Vertical Reinforcement spacing (mm)	120
Horizontal Reinforcement Dia. Φh (mm)	20
Horizontal Reinforcement spacing (mm)	100
Boundary Element tie spacing (sv) (mm)	100
Boundary element Ties dia (mm)	10
Boundary Element size (mm x mm)	895 X 400

As per above reinforcement the provided resistance capacity of wall would be

Force/Moment	M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c(kN)	P_t(kN)
Design Forces	11407.6	506.2	4038.1	410.63	16974.9	11335.23
Provided Capacity	19292.34	737.383	8222.13	1643.81	32558.75	19034.5

Vertical reinforcement in Boundary element:

The boundary element shall have adequate axial load carrying capacity and should be designed assuming short column action (IS 13920:2016 Cl 10.4.2). The axial load carrying capacity of a short axially loaded column is given as:

$P_u = 0.4\ f_{ck}A_c + 0.67\ f_y\ A_{sc}$ (IS 456:2000 Cl 39.3)

Where,

Pu= axial load on member

f_{ck} = characteristic compressive strength of concrete

A_c = Area of concrete

f_y= Characteristic strength of the compression reinforcement

A_{sc}= area of longitudinal reinforcement for columns

But, as per IS 13920:2016 Cl 10.4.2.1 the load factor for gravity loads shall be taken as 0.8.

So, $P_u = 0.8\ (0.4\ f_{ck}A_c + 0.67\ f_y\ A_{sc})$ is used to calculate the load carrying capacity of the boundary element.

The vertical reinforcement in boundary elements shall not be less than 0.8% and not greater than 6% (4% practically). (IS 13920:2016 Cl 10.4.3)

Moment of resistance from web section (without boundary element) (i.e. by taking $l_w = 4475 - 2 \times 895 = 2685mm$) can be obtained as in Section i.). So, new

$M_{u,\text{web}} = 2632.479$ kNm

Remaining Moment $M_3 - M_{3u,\text{web}} = 11406.7 - 2632.479 = 8774.221\ kNm$

Distance between centroids of two boundary elements ,C_w= Length of wall – Size of Boundary Element1 / 2 – Size of Boundary Element2 /2

=4475 - 895/2 - 895/2

=3580 mm

So, Tension and Compression force due to remaining Moment

$= \frac{M_3 - M_{u,web}}{C_w} = \frac{8774.221\ X\ 1000}{3580}$ =2451.1 kN

In addition to the above tensile and compressive force, boundary element must also share someaxial load, in proportion to its area.

So, Compression due to axial load Pu on each boundary element

$=\frac{Axial\ compressive\ load}{Total\ area\ of\ wall}\ X\ Area\ of\ Boundary\ element$

$= \frac{16974.9}{4475\ X\ 400}\ X\ 895\ X\ 400$

= 3394.98 kN

So, Total Axial Compressive Load on boundary element

$P_{c,} = 2451.1+3394.98 = 5846.08kN$

Total Axial Tensile Load $P_t = 0kN$

Now, Let's provide 3.9% reinforcement on boundary wall, such that

A_{sc} =3.9% X 895 X 400=13962 mm^2

A_c = 895 X 400 – 13962 = 344038 mm^2

Then Axial Compression Capacity:

$P_u = 0.8\ (0.4\ f_{ck}A_c + 0.67\ f_y\ A_{sc}) = 5858.011$ kN > $P_{c,be}$ (OK)

Axial Tension Capacity

$P_{ut} = 0.87 f_y A_{st} = 5040.98$ kN > $P_{t,}$ (OK)

So provide 14 – 36φ bars ($A_{st,provided}$ = 14243.04 mm^2)

Design Summary

Now, a summary of the design forces and provided reinforcement is shown for the same lift shear wall section in each floor.

Table 45: Design of Lift shear wall

Location: Left wall Along Y-axis	
Length of wall	4475 mm
Thickness of wall, t_w	400 mm
Pier id	L1

Reinforcement	Lower Basement	Upper Basement	Ground floor	1st floor	2nd floor	3rd floor	4th floor	Top Floor
Vertical Reinforcement Dia. Φ_v (mm)	32	28	25	20	16	16	16	16
Vertical Reinforcement spacing (mm)	120	120	120	120	120	120	120	120

Horizontal Reinforcement Dia. Φ_h (mm)	20	20	16	16	16	16	16	16
Horizontal Reinforcement spacing (mm)	100	150	150	150	150	150	150	150
Hz. Boundary Element tie spacing (s_v) (mm)	100	100	90	90	90	90	90	90
Hz. Boundary element Ties dia (mm)	10	10	8	8	8	8	8	8
Boundary Element size (mm x mm)	895 X 400	895 X 400	895 X 400	895 X 400	895 X 400	895 X 400	895 X 400	895 X 400
Vertical rebar at Boundary element dia(mm)	36	28	Not needed. Provide same as in Vertical reinforcement in wall					
No. of V_z rebar at Boundary element	14	14						

Force/Moment		M_3 (kNm)	M_2 (kNm)	V_2 (kN)	V_3 (kN)	P_c (kN)	P_t (kN)
Lower Basement	Design Forces	11407.6	506.2	4038.1	410.63	16974.9	11335.23
	Provided Capacity	19292.34	737.3832	8222.131	1643.807	32558.75	19034.5
	Ratio	1.691183	1.456703	2.036139	4.003134	1.918053	1.679234
Upper Basement	Design Forces	10352.9	307.9	4276.1	545	14987.4	9256
	Provided Capacity	18875.06	644.2643	5969.38	1583.83	30669.4	16581.17
	Ratio	1.823167	2.092447	1.395987	2.90611	2.046346	1.791397
Ground Floor	Design Forces	7933	226	3528	545	10787	5570
	Provided Capacity	17975.67	478.8732	4900.809	1435.684	27281.6	12182.08
	Ratio	2.265936	2.118908	1.389118	2.634283	2.529119	2.187088
1st Floor	Design Forces	5865	250.8	2962	547.2	7495.5	3347
	Provided Capacity	16377.04	334.4936	4731.738	1266.613	24415	8459.779
	Ratio	2.792334	1.333706	1.597481	2.314716	3.257288	2.527571
2nd Floor	Design Forces	3941	258	2527	524	5611	2387.1
	Provided Capacity	15176.31	271.7236	4638.477	1173.352	23177.15	6852.421
	Ratio	3.850878	1.053192	1.835567	2.239221	4.130663	2.870605
3rd Floor	Design Forces	2358	157	2069	517	4383	2118

	Provided Capacity	12259.56	165.3207	4432.61	967.4847	21092.35	4145.292
	Ratio	5.199133	1.052998	2.142393	1.871344	4.812309	1.957173
4th Floor	Design Forces	1386	151	1361	426	3231	1933
	Provided Capacity	11400.63	165.3207	4432.61	967.4847	21092.35	4145.292
	Ratio	8.225564	1.094839	3.256877	2.271091	6.528119	2.144486
Top Floor	Design Forces	995	131.1	803.3	548.1	1966	1465.5
	Provided Capacity	10170.65	165.3207	4432.61	967.4847	21092.35	4145.292
	Ratio	10.22176	1.261028	5.518001	1.765161	10.72856	2.828585

Similarly, other sections of Lift Shear wall can also be designed.

• Design of Foundation:

Foundation are structural elements that transfer load from the building or individual column to the earth below. If these loads are to be transmitted properly, foundations should be designed to prevent excessive settlement and rotation, to minimize differential settlement and to provide adequate safety against sliding and overturning. Foundation can be classified as:

(1) Isolated footing under individual columns. These may be rectangular, square of circular in plan.
(2) Strip foundation or Wall foundation
(3) Combined footing supporting two or more column load.
(4) Mat or Raft foundation
(5) Plie Foundation

Soil Type used: Medium stiff soils

N values: 10-30(table 2 IS 1893:2016)

So subgrade modulus is taken as 1.5kg/cm^3

=1.8kg/cm^3

=18000 kN/m^3

$$k = \frac{Safe\ Bearing\ Capacity\ X\ FOS}{Permissible\ Settlement}$$

Safe bearing Capacity=150kN/m^2

Table 46:Selection of suitable foundation type

Column ID	$Fz(kN)$	$Mx(kNm)$	$My(kNm)$	$ex(m)$	$ey(m)$	Length (m)	Area(m)
A3	1228.27	59.24	96.58	0.05	0.08	3.46	11.99
A4	952.74	62.83	80.81	0.07	0.08	3.13	9.82
A5	1199.83	55.42	-93.1	0.05	-0.08	3.00	9.00
B2	1468.98	69.23	97.25	0.05	0.07	3.46	11.99
B3	2321.53	83.88	99.66	0.04	0.04	4.53	20.52
B4	2456.42	76.05	91.05	0.03	0.04	4.62	21.39
B5	2665.64	79.71	-99.71	0.03	-0.04	4.60	21.12
B6	1758.01	57.73	90.99	0.03	0.05	3.98	15.86
C1	1680.21	63.68	88.39	0.04	0.05	3.91	15.31

C2	2005.15	64.52	122.83	0.03	0.06	4.26	18.15
C3	3587.76	65.48	90.73	0.02	0.03	5.48	30.07
C4	3257.4	98.65	92.99	0.03	0.03	5.27	27.80
C5	3765.05	57.27	90.18	0.02	0.02	5.60	31.38
C6	2978.86	61.14	-111.03	0.02	-0.04	4.83	23.34
C7	1317.44	55.65	-88.73	0.04	-0.07	3.17	10.04
D1	2223.5	65.47	91.68	0.03	0.04	4.42	19.50
D2	1874.64	59.7	13.18	0.03	0.01	3.98	15.88
D3	4059.6	-65.65	107.24	-0.02	0.03	5.73	32.83
D5	3514.17	54.78	-121.75	0.02	-0.03	5.24	27.50
D6	2808.25	53.12	-108.44	0.02	-0.04	4.68	21.90
D7	874.33	52.05	-93.13	0.06	-0.11	2.49	6.20
E1	1492.41	-62.4	86.68	-0.04	0.06	3.50	12.27
E2	2112.08	-60.59	112.61	-0.03	0.05	4.18	17.49
E3	3092.08	99.47	90.68	0.03	0.03	5.15	26.51
E4	3317.97	-91.78	92.13	-0.03	0.03	5.15	26.55
E5	3429.29	-54.2	88.36	-0.02	0.03	5.27	27.75
E6	2467.22	-64.24	-109.44	-0.03	-0.04	4.21	17.76
E7	1244.11	-55.34	80.96	-0.04	0.07	3.21	10.34
F2	1481.76	-65.97	91.52	-0.04	0.06	3.49	12.21
F3	2374.34	-84.25	94.76	-0.04	0.04	4.37	19.11
F4	2685	-74.44	87.29	-0.03	0.03	4.65	21.61
F5	2744.07	-77.25	-108.3	-0.03	-0.04	4.47	19.96
F6	2730.38	-17.37	199.4	-0.01	0.07	4.86	23.64
G3	1176.54	-55.75	86.52	-0.05	0.07	3.14	9.88
G4	1694.63	-57.74	84.06	-0.03	0.05	3.73	13.90
G5	1206.42	-52.52	-79.21	-0.04	-0.07	2.70	7.31
Total Area 667.8							
Total Plinth Area 795.6							
% Foundation Area 83.94%							

Since area covered by isolated foundation is more than 50%, mat foundation will be more economical than isolated footing.

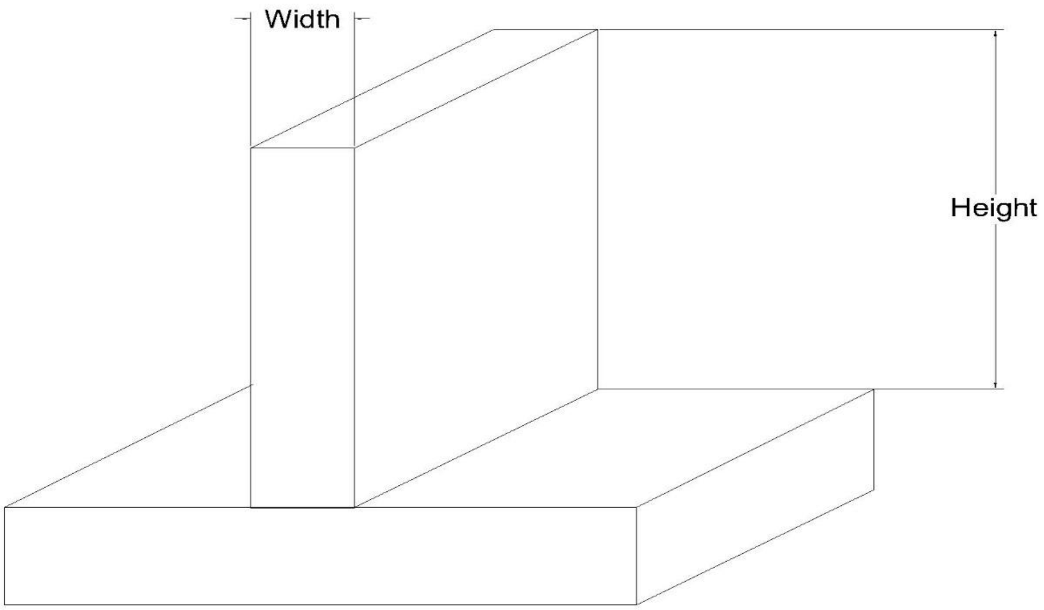


Figure 15: Foundation idealization

Unit weight =25kN/m^3

Height=3.645*2=7.29m

Width=0.2m

Dead load=25*7.29*0.2

=36.45kN/m

Parking Loading

12 car parks and 36 bike parks

Avg weight of car=1302kg

Avg weight of bike=30kg

Total weight=16704kg

=0.21kN/m^2

Considering factor of safety, 0.5kN/m^2 parking load is applied.

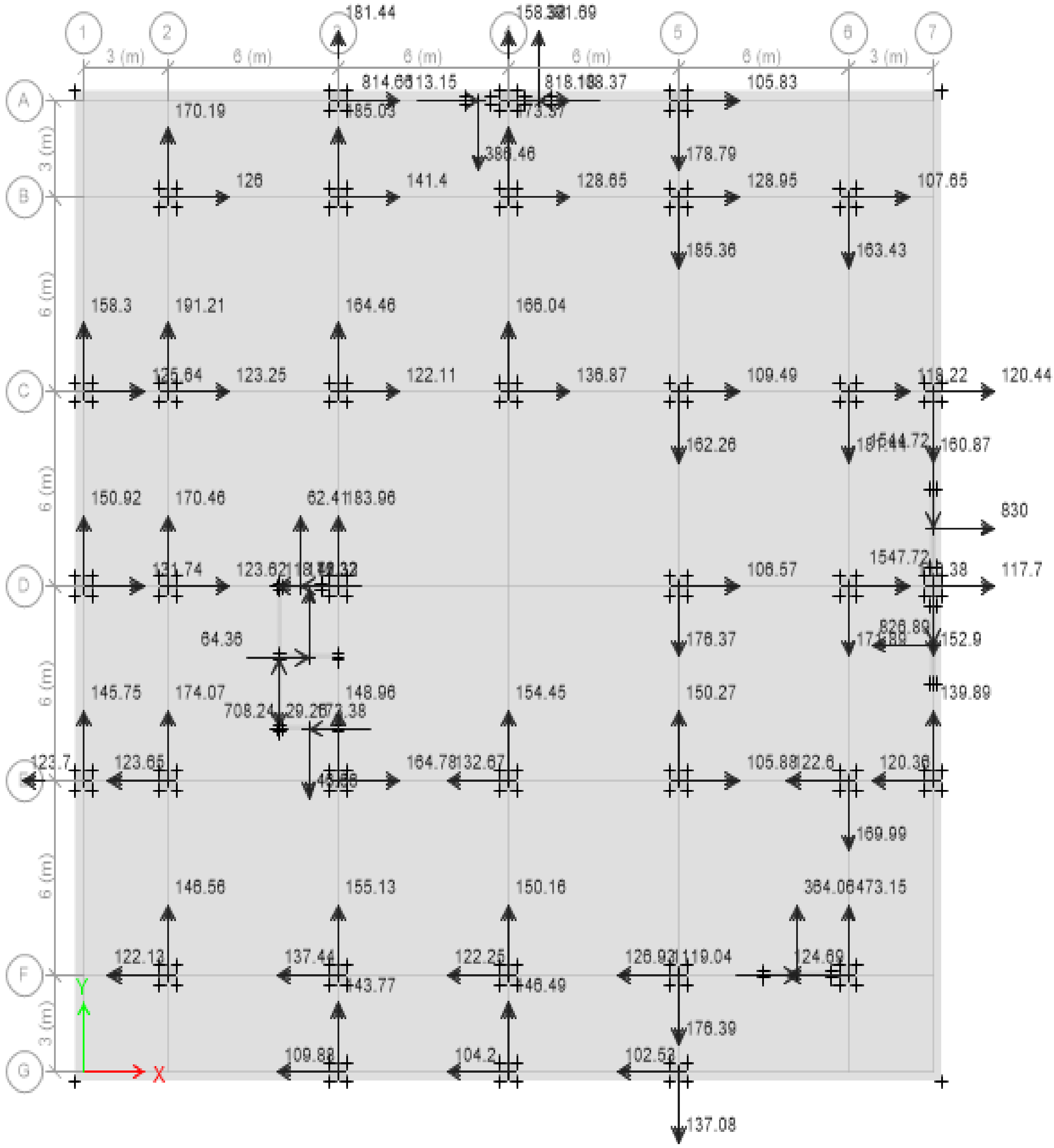


Figure 16: Loads on Foundation for Ultimate state

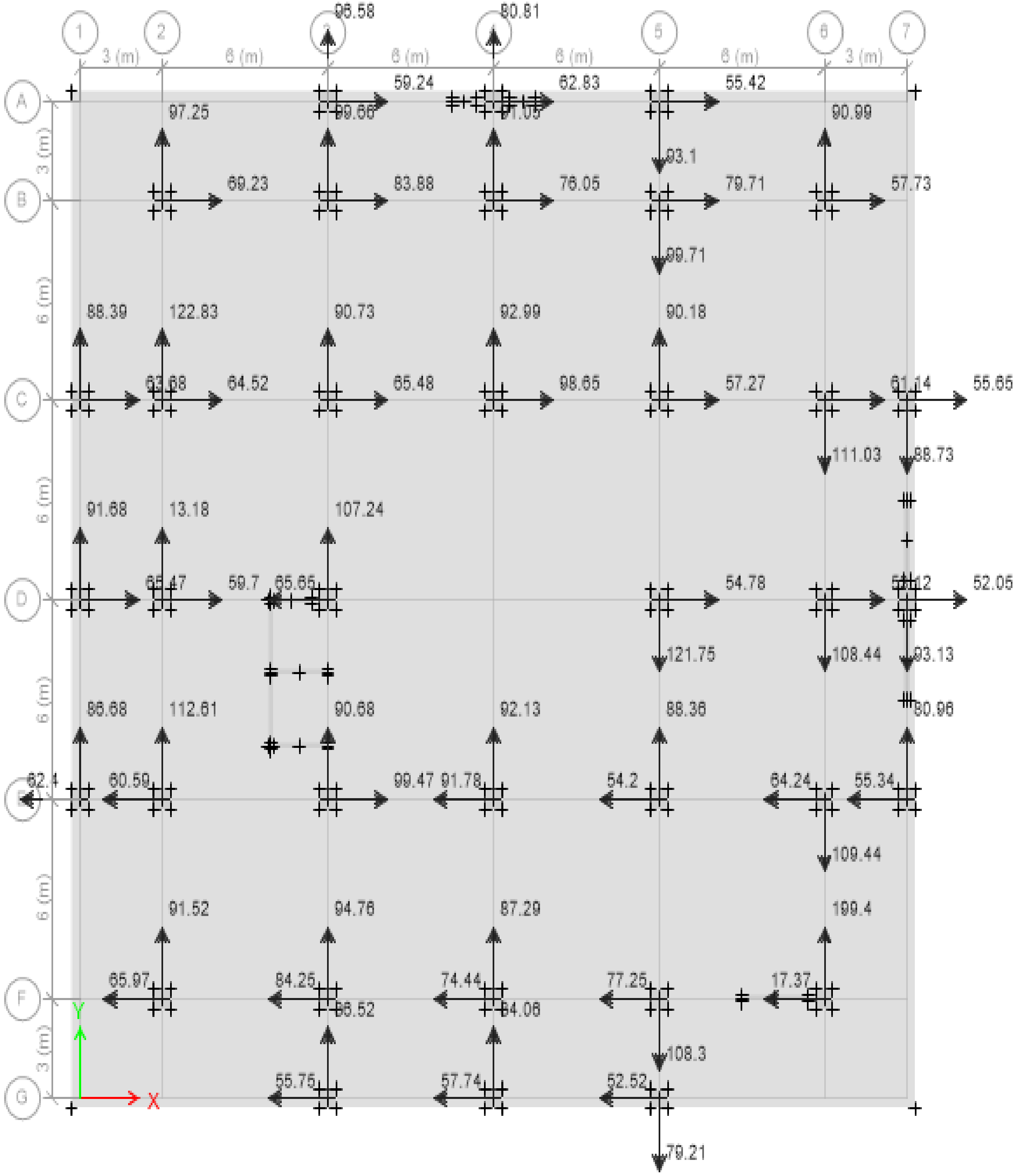


Figure 17 : Loads at serviceability Condition

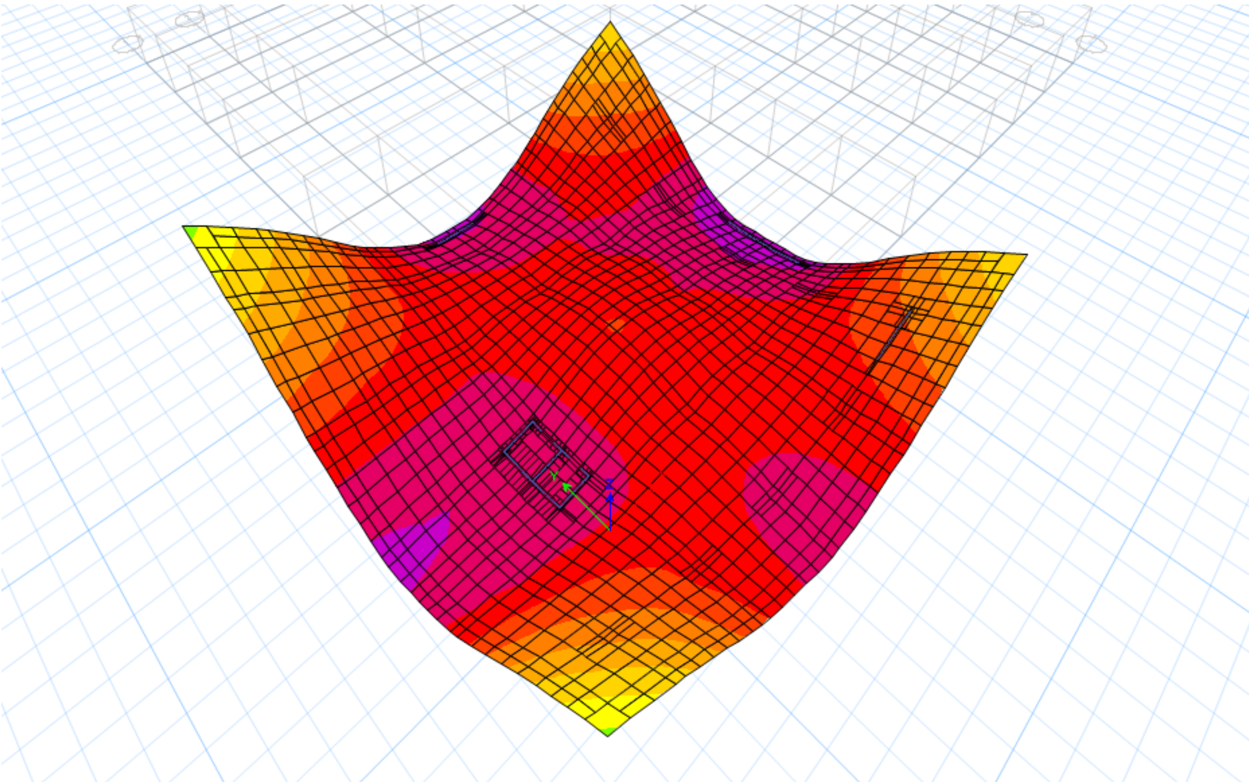


Figure 18: Deflected shape of foundation scale 2000:1

Settlement Criteria <25mm

Table 47: Settlement of foundation under different load combination

Load Combination	Settlement(mm)	Remarks
(DL+LL)	8.745	Okay
(DL+0.8LL+0.8EQx)	9.31	Okay
(DL+0.8LL-0.8EQx)	9.521	Okay
(DL+0.8LL+0.8EQy)	9.321	Okay
(DL+0.8LL-0.8EQy)	9.019	Okay
0.9DL+EQx	7.758	Okay
0.9DL-EQx	8.165	Okay
0.9DL+EQy	8.112	Okay
0.9DL-EQy	7.775	Okay
DL+EQx	8.657	Okay
DL-EQx	8.852	Okay
DL+EQy	8.89	Okay
DL-EQy	8.47	Okay

Safe bearing capacity

Permissible safe bearing capacity=150kN/m^2

As per cl 6.3.5 IS 1893:2016 an increment of 25% (Table 1,IS 1893) is allowed in the safe bearing capacity of the soil for seismic load combinations

Safe Bearing Capacity considering seismic case=187.5 kN/m^2

$$Alpha = 1 + 6 * ex / Lx + 6 * ey / Ly$$

$$qa = \frac{Fz}{A} * alpha$$

Table 48: Safe bearing capacity

Output Case	Fz kN	e_x m	e_y m	α	Bearing Pressure	Bearing Pressure from SAFE	q_a	Type	Remark
(DL+LL)	119133.20	0.24	0.36	1.12	140.10	143.47	150	Long term	Okay
(DL+0.8LL+0.8 EQx)	115147.42	-0.34	0.34	1.13	137.56	155.53	187.5	Seismic	Okay
(DL+0.8LL-0.8EQx)	115147.42	0.81	0.34	1.22	148.61	159.26	187.5	Seismic	Okay
(DL+0.8LL+0.8 EQy)	115147.42	0.23	-0.24	1.09	132.56	156.77	187.5	Seismic	Okay
(DL+0.8LL-0.8EQy)	115147.42	0.23	0.92	1.22	148.61	151.52	187.5	Seismic	Okay
0.9DL+EQx	89283.87	-0.71	0.26	1.19	111.87	131.49	187.5	Seismic	Okay
0.9DL-EQx	89283.86	1.15	0.26	1.27	119.94	137.63	187.5	Seismic	Okay
0.9DL+EQy	89283.87	0.22	-0.68	1.17	110.57	134.35	187.5	Seismic	Okay
0.9DL-EQy	89283.86	0.22	1.19	1.27	119.94	128.31	187.5	Seismic	Okay
DL+EQx	99204.30	-0.62	0.26	1.17	122.40	143.37	187.5	Seismic	Okay
DL-EQx	99204.29	1.06	0.26	1.26	131.37	149.18	187.5	Seismic	Okay
DL+EQy	99204.30	0.22	-0.58	1.16	120.95	146.32	187.5	Seismic	Okay
DL-EQy	99204.29	0.22	1.09	1.26	131.37	139.79	187.5	Seismic	Okay

Check for Overturning and Sliding

In order to ensure the structural stability of a building or any other structure, it is important to consider both its resistance to overturning and sliding. To achieve this, it is recommended that the Factor of Safety (FOS) for both scenarios should be greater than 1.5. The FOS is a ratio that represents the capacity of the structure to withstand the forces acting upon it, divided by the magnitude of those forces. In the case of overturning, the FOS should be greater than 1.5 to ensure that the structure can resist the rotational forces that may cause it to tip over. Similarly, for sliding, the FOS should also be greater than 1.5 to ensure that the structure can withstand the horizontal forces that may cause it to slide off its foundation. By ensuring that the FOS is greater than 1.5 for both overturning and sliding, it can be reasonably assured that the structure is adequately designed to resist the forces that it is likely to experience during its lifespan.

Table 49 Check for Overturning

Output Case	F_Z(KN)	$M_{x,CG}$	$M_{y,CG}$	M_{rx}	M_{ry}	FOS_X	FOS_Y	Remarks
(DL+LL)	119133.2	-42510.6	-28180.3	1834651	1834651	43.15753	65.1041	Okay
(DL+0.8LL+0.8EQx)	115147.419	-39079.7	39699.74	1773270	1773270	45.37569	44.66705	Okay
(DL+0.8LL-0.8EQx)	115147.418	-39079.7	-93525.2	1773270	1773270	45.3757	18.96034	Okay
(DL+0.8LL+0.8EQy)	115147.419	27532.54	-26912.3	1773270	1773270	64.40634	65.89081	Okay
(DL+0.8LL-0.8EQy)	115147.418	-105692	-26912.2	1773270	1773270	16.77771	65.89086	Okay
0.9DL+EQx	89283.8661	-22820.8	63607.46	1374972	1374972	60.25084	21.61651	Okay
0.9DL-EQx	89283.864	-22820.8	-102924	1374972	1374972	60.25086	13.35916	Okay
0.9DL+EQy	89283.8657	60444.57	-19658.5	1374972	1374972	22.74764	69.94274	Okay
0.9DL-EQy	89283.8642	-106086	-19658.5	1374972	1374972	12.9609	69.94282	Okay
DL+EQx	99204.2955	-25356.4	61422.85	1527746	1527746	60.25084	24.8726	Okay
DL-EQx	99204.2934	-25356.4	-105108	1527746	1527746	60.25086	14.535	Okay
DL+EQy	99204.295	57908.93	-21842.1	1527746	1527746	26.38188	69.94488	Okay
DL-EQy	99204.2936	-108622	-21842.1	1527746	1527746	14.06482	69.94495	Okay

Table 50 Check for Sliding

Output Case	F_Z	F_X	F_Y	F_H	FOS	Remark
	kN	kN	kN	kN	$\mu F_Z/F_H$	
(DL+LL)	119133.2	0.00	0.0001	0.000141	4.2E+08	Okay
(DL+0.8LL+0.8EQx)	115147.4	-2853.58	0.0005	2853.583	20.1759	Okay
(DL+0.8LL-0.8EQx)	115147.4	2853.58	0.0004	2853.584	20.1759	Okay
(DL+0.8LL+0.8EQy)	115147.4	0.00	-2853.64	2853.644	20.1755	Okay
(DL+0.8LL-0.8EQy)	115147.4	0.00	2853.644	2853.644	20.1755	Okay
0.9DL+EQx	89283.87	-3566.98	0.0006	3566.984	12.5153	Okay
0.9DL-EQx	89283.86	3566.98	0.0004	3566.984	12.5153	Okay
0.9DL+EQy	89283.87	0.00	-3567.05	3567.054	12.5151	Okay
0.9DL-EQy	89283.86	0.00	3567.055	3567.055	12.5151	Okay
DL+EQx	99204.30	-3566.98	-0.0003	3566.978	13.9059	Okay
DL-EQx	99204.29	3566.98	0.0004	3566.983	13.9059	Okay
DL+EQy	99204.30	0.00	-3567.05	3567.055	13.9056	Okay
DL-EQy	99204.29	0.00	3567.055	3567.055	13.9056	Okay

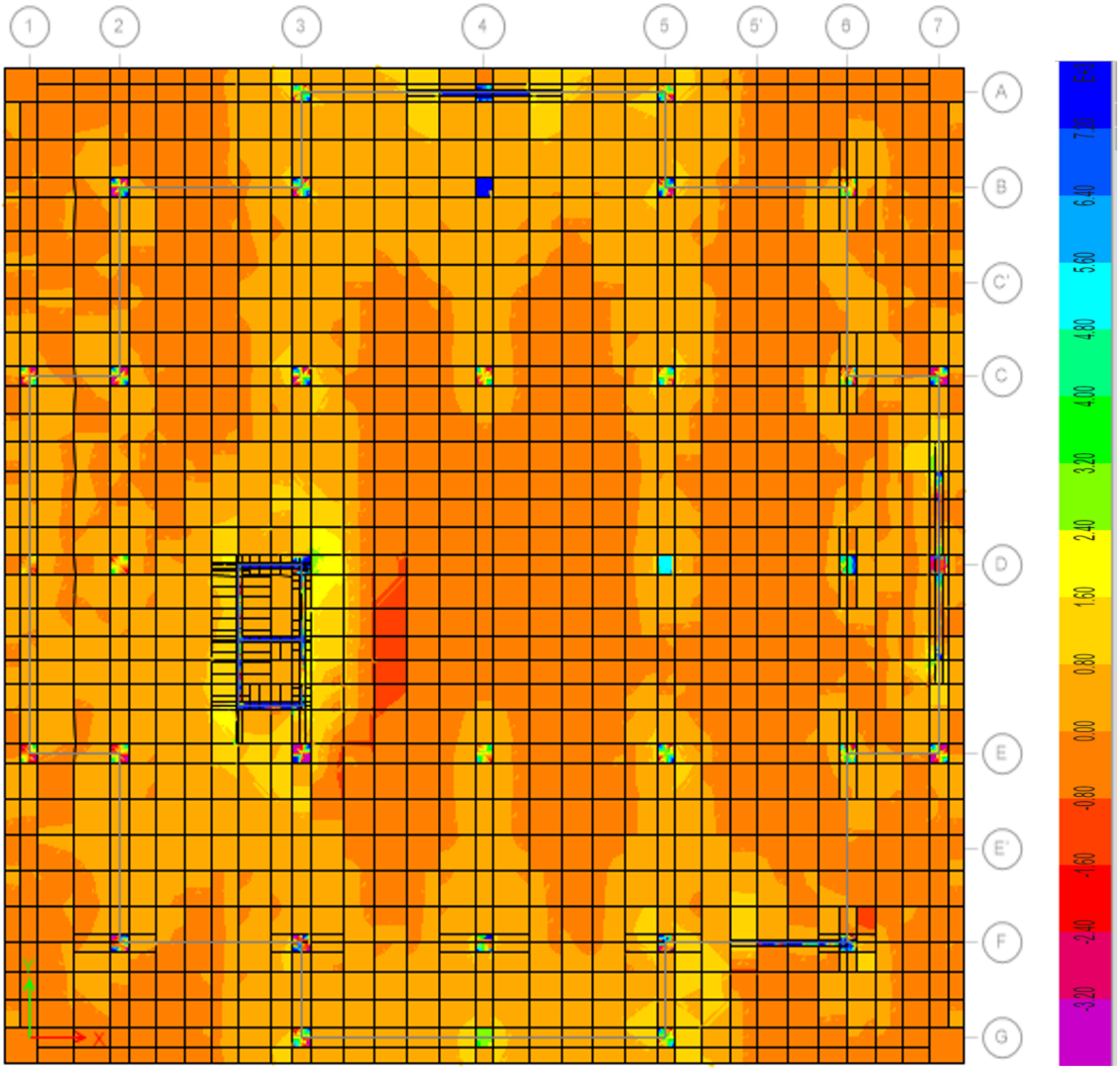


Figure 19: Shell stress M11 at collapse envelope

Upon analyzing, it can be observed that the majority of the maximum moment values occur near the shear wall end and column end of the structure. In contrast, significantly lower moments are observed in other areas of the structure. Taking this into account, the structural design has been optimized for economic efficiency by providing additional reinforcement only in the areas where the maximum moment values are observed, i.e., at the column and shear wall ends. Specifically, the main reinforcement has been designed to withstand a moment value of 1000kNm, which is the maximum moment that can be sustained by the structure as a whole. However, to ensure that the observed maximum moment values near the column and shear wall ends can be safely sustained without requiring unnecessary reinforcement throughout the rest of the structure, additional reinforcement has been strategically placed in those areas. By doing so, the design becomes more economical as it minimizes the amount of unnecessary reinforcement needed while still ensuring the safety and stability of the structure in areas where maximum moment values are observed.

Table 51:Regular Reinforcement in foundation

Location	**Moment**	**Bending Moment from Analysis(kNm)**	**Reinforcement Provided**	**Moment Capacity(kNm)**
X-T	$-M_{11}$	<=1600	25ϕ @ 100 mm	2407.697
X-B	$+M_{11}$	<=1800	25ϕ @ 100 mm	2407.697
Y-T	$-M_{22}$	<=1500	25ϕ @ 100 mm	2407.697
Y-B	$+M_{22}$	<=1800	25ϕ @ 100 mm	2407.697

Table 52: Additional reinforcement at column and shear wall area

Location	**Moment**	**Bending Moment from Analysis (kNm)**	**Reinforcement Provided**	**Moment Capacity(kNm)**
X-T	$-M_{11}$	3374.47	32ϕ @ 100 mm	3793.72
X-B	$+M_{11}$	1582.742	25ϕ @ 150 mm	1637.22
Y-T	$-M_{22}$	893.78	25ϕ @ 250 mm	997.74
Y-B	$+M_{22}$	2016.79	25ϕ @ 100 mm	2407.69677

Punching Shear

In structural design, the analysis of shear is a crucial factor to ensure the safety and stability of slabs and foundations. In this regard, the two-way shear is generally considered to be more critical than the one-way shear for these elements. The reason for this is that two-way shear tends to occur in areas where the applied loads act on a relatively small area of the slab or foundation, resulting in high shear stresses that can cause significant structural damage.

To address this, the critical section for shear has been defined in Clause 31.6.1 of the Indian Standard Code of Practice for Plain and Reinforced Concrete, IS 456:2000. According to this clause, the critical section for shear should be located at a distance d/2 from the periphery of the column, perpendicular to the plane of the slab. This distance d is typically calculated as the effective depth of the slab, which is defined as the distance from the top of the slab to the centroid of the reinforcement in the tension zone. By locating the critical section at this distance, the design can ensure that the slab or foundation can resist the maximum shear stresses that are likely to occur, thereby ensuring the safety and stability of the structure.

For column C3,

Size of column=600x600

Effective Depth of foundation=1440mm

For punching shear stress, shear stress due to both axial force and moment should be considered.

The unbalanced was considered during shear stress calculation as per **clause 31.3.3.**

$M_{unbalanced}=\Upsilon_v*M$

$$\Upsilon v = 1 - \frac{1}{1+(\frac{2}{3})\sqrt{a1/a2}}$$

Where a1 is the width of the critical section measured in the direction of the span and a2 is the width of the critical section measured in the direction perpendicular to the span.

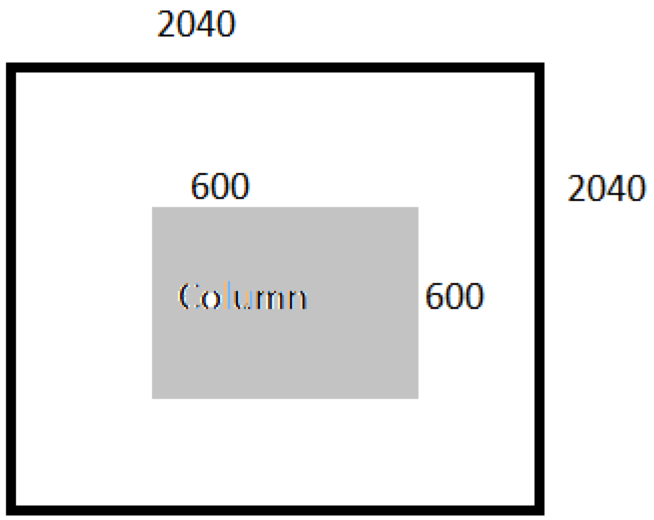


Figure 20: Column C3 with punching shear failure plane

Table 53 Parameters of Failure Plane

d	1440 mm	
Col. Size	B_x	600
	B_y	600
Pun. Area. Dim	PA_X	2040
	Pa_y	2040
b_0		8160
Υ_{v2}		0.4
Υ_{v3}		0.4

Given the punching shear force and the fractions of moments transferred by eccentricity of shear about the two axes, the shear stress is computed assuming linear variation along the perimeter of the critical section.

$$v_U = \frac{V_U}{b_0 d} + \frac{\gamma_{V2}[M_{U2} - V_U(y_3 - y_1)][I_{33}(y_4 - y_3) - I_{23}(x_4 - x_3)]}{I_{22}I_{33} - I_{23}^2} - \frac{\gamma_{V3}[M_{U3} - V_U(x_3 - x_1)][I_{22}(x_4 - x_3) - I_{23}(y_4 - y_3)]}{I_{22}I_{33} - I_{23}^2}$$

The calculation of shear stress is shown in tables below:

Items	Side 1	Side 2	Side 3	Side 4	Sum
X_2	-1020	0	1020	0	N.A
Y_2	0	1020	0	-1020	N.A
L	2040	2040	2040	2040	bo=8160
D	1440	1440	1440	1440	N.A
Ld	2937600	2937600	2937600	2937600	11750400
LdX_2	-2996352000	0	3E+09	0	0
LdY_2	0	2996352000	0	-299635200	0
X_3	$\sum LdX_2/Ld$	0			
Y_3	$\sum LdY_2/Ld$	0			

Items	Side 1	Side 2	Side 3	Side 4	Sum
L	2040	2040	2040	2040	N.A
D	1440	1440	1440	1440	N.A
X_2-X_3	-1020	0	1020	0	N.A
Y_2-Y_3	0	1020	0	-1020	N.A
Parallel to	*X-Axis*	*Y-Axis*	X-Axis	Y-Axis	N.A
I_{xx}	1.52638E+12	3.05628E+12	1.53E+12	4.58266E+12	1.06917E+13
I_{yy}	3.05628E+12	1.52638E+12	3.06E+12	1.52638E+12	9.16531E+12
I_{xy}	0	0	0	0	0

Column Forces		
V_u	3137.28	*kN*
M_{u2}	166.83	kNm
M_{u3}	170.15	kNm
$\Upsilon_{v2}M_{u2}$	66.732	kNm
$\Upsilon_{v3}M_{u3}$	68.06	kNm

Points	X_4	Y_4	v_u-axial	v_uM_{ux}	v_uM_{uy}	*V_u*
A	-1020	1020	0.266993	0.006366	-0.00757	0.265785

B	1020	1020	0.266993	0.006366	0.007574	0.280934
C	1020	-1020	0.266993	-0.00637	0.007574	0.268201
D	-1020	-1020	0.266993	-0.00637	-0.00757	0.253053

Maximum Shear Stress=0.281N/mm^2 at point B

Shear Capacity

Ks = 1

$T_c = 0.25(f_{ck})^{1/2}$=1.25N/mm^2

$k_s*\tau_c$ = 1.25

Shear Ratio=$V_u/(K_s\ \tau_c)$=0.224<1

So this is safe in foundation.

For other column, calculation is shown is in

Table 54

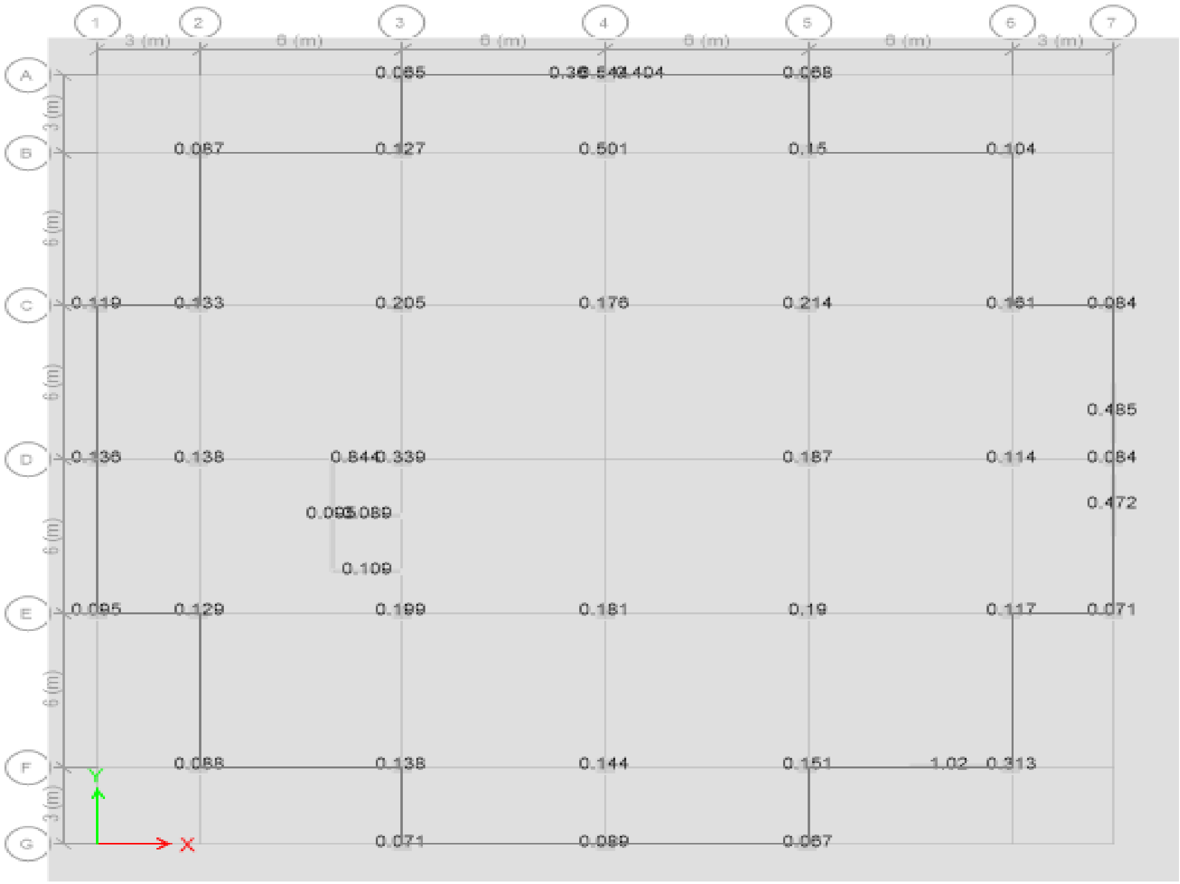


Figure 21: Punching Shear Ratio safe results

Table 54 Punching shear ratio

Column ID	Punching Shear Ratio	Remarks
A3	0.065	okay
A4	0.544	okay
A5	0.068	okay
B2	0.087	okay
B3	0.127	okay
B4	0.501	okay
B5	0.15	okay
B6	0.104	okay
C1	0.11	okay
C2	0.133	okay
C3	0.205	okay
C4	0.176	okay
C5	0.214	okay
C6	161	okay
C7	0.084	okay
D1	0.136	okay
D2	0.138	okay
D3	0.339	okay
D5	0.187	okay
D6	0.114	okay
D7	0.084	okay
E1	0.005	okay
E2	0.12	okay
E3	0.2	okay
E4	0.181	okay
E5	0.19	okay
E6	0.117	okay
E7	0.071	okay
F2	0.088	okay
F3	0.138	okay
F4	0.144	okay
F5	0.151	okay
F6	0.313	okay
G3	0.071	okay
G4	0.089	okay
G5	0.067	okay

Punching shear failure occurs when a concentrated load, such as a column or a point load, is applied to a concrete slab or wall. The critical section for punching shear is the perimeter around the load. In the case of a shear wall, the critical section area is larger due to the wall's larger dimensions. This means that the load is distributed over a larger area, reducing the stress concentration and the likelihood of punching shear failure. Therefore, it was deemed unnecessary to perform a punching shear check for the shear wall in question. However, it's important to note that this decision should be based on a thorough analysis of the wall's design and loadings.

Table 55: Development Length for foundation

Concrete	M	25	N/mm^2
Rebar	Fe	415	N/mm^2
τ_{bd}		1.4	N/mm^2
Ø		32	mm
L_d	$\frac{0.87 f y Ø}{4\tau bd}$	2.063143	m
$\frac{M}{V} + L_0$	$\frac{997.74}{127.81} + 0.38$	8.06>L_d (okay)	m L_0=12Ø

7 Conclusion

In conclusion, this building project has provided us with a valuable learning experience in earthquake resistant design and ductile detailing of concrete structures. As Nepal is located in a seismically active region, it is crucial to consider the safety of structures and human lives during earthquakes. By following the earthquake resistant design code (IS 1983 (Part-I):2016) and ductile detailing of concrete (IS 13920:2016), we have designed a building that is better equipped to withstand lateral earthquake loads and minimize damage. Our team worked together to idealize, analyze, and design the building under the guidance of our respected supervisor, and we hope that our design meets their expectations. We believe that this project has given us a deeper understanding of the transfer mechanism of lateral earthquake loads into vertical members and, finally, into the foundation. Overall, we are grateful for the opportunity to work on this project, and we are confident that the knowledge and skills we have gained will serve us well in our future careers as civil engineers.

8 References

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ANNEX

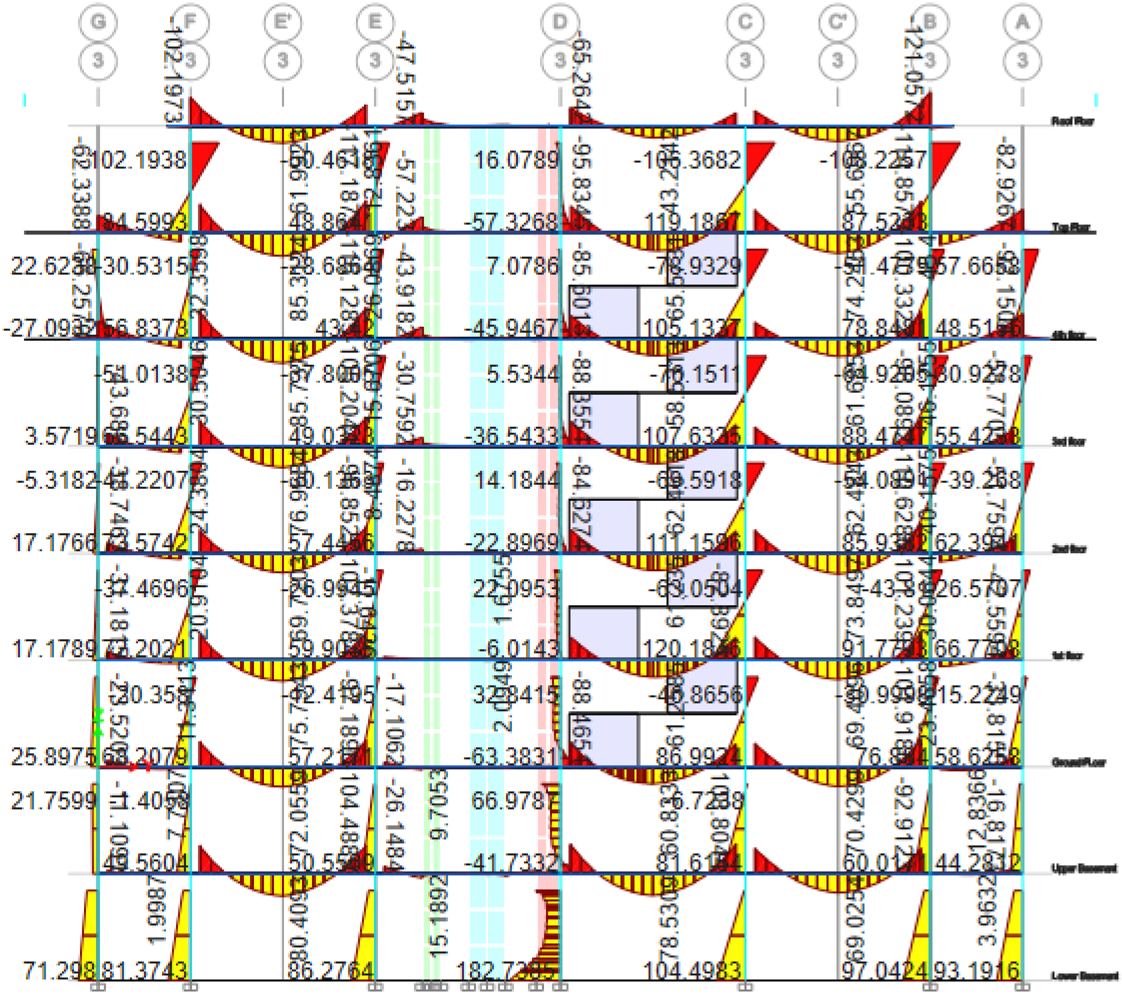


Figure 22 Moment Diagram at 1.2(DL+LL+EQx)

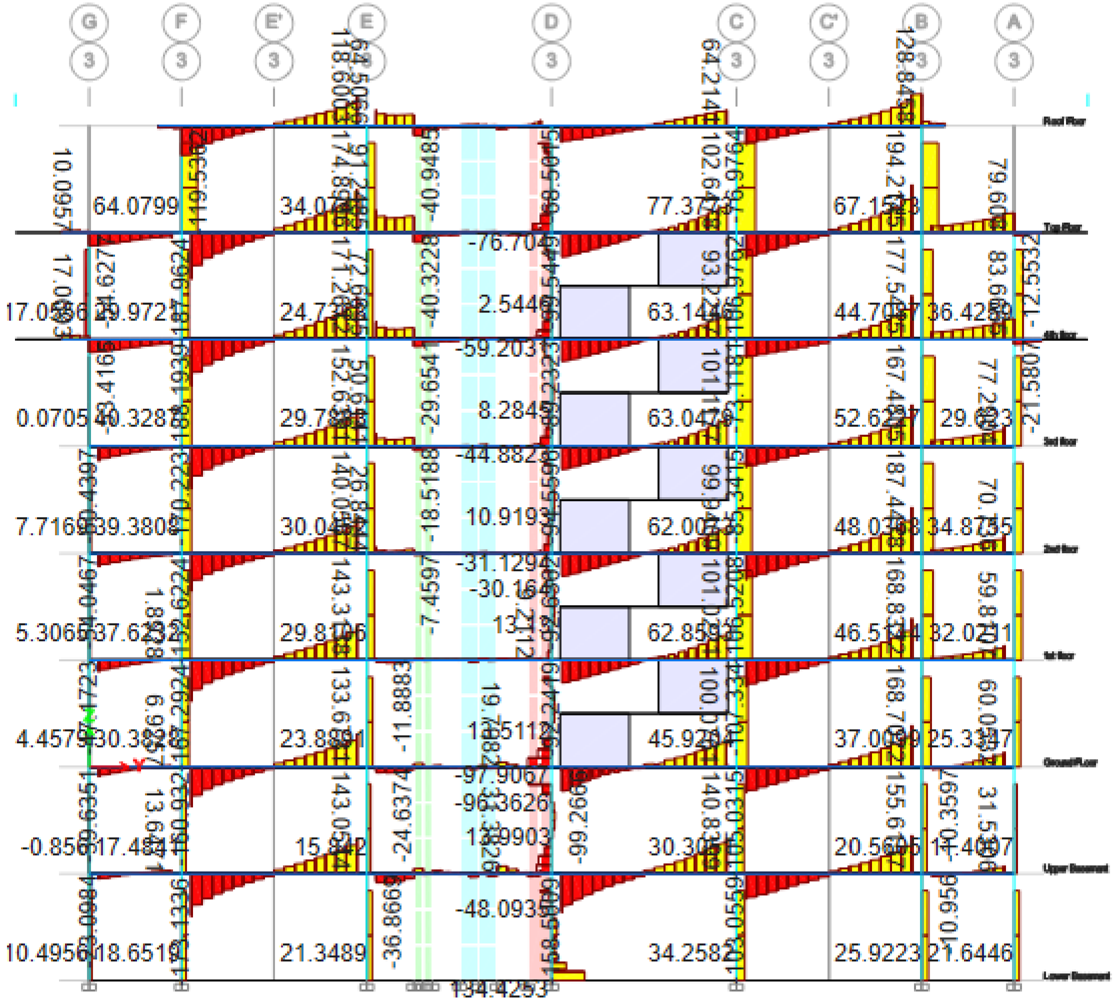


Figure 23 Shear Force Diagram at 1.2(DL+LL+EQx)

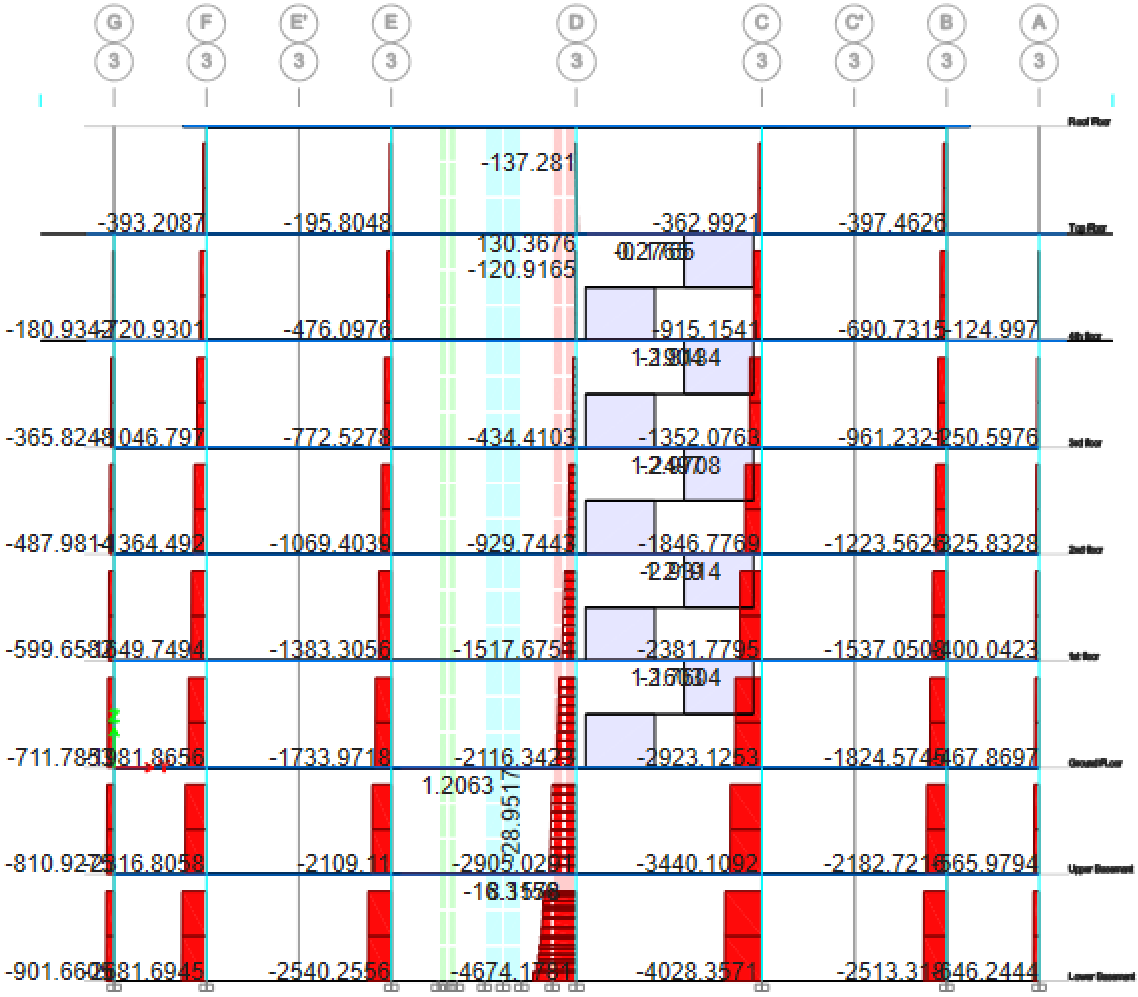


Figure 24 Axial Force Diagram at 1.2(DL+LL+EQx)

Figure 25 Longitudinal Reinforcement

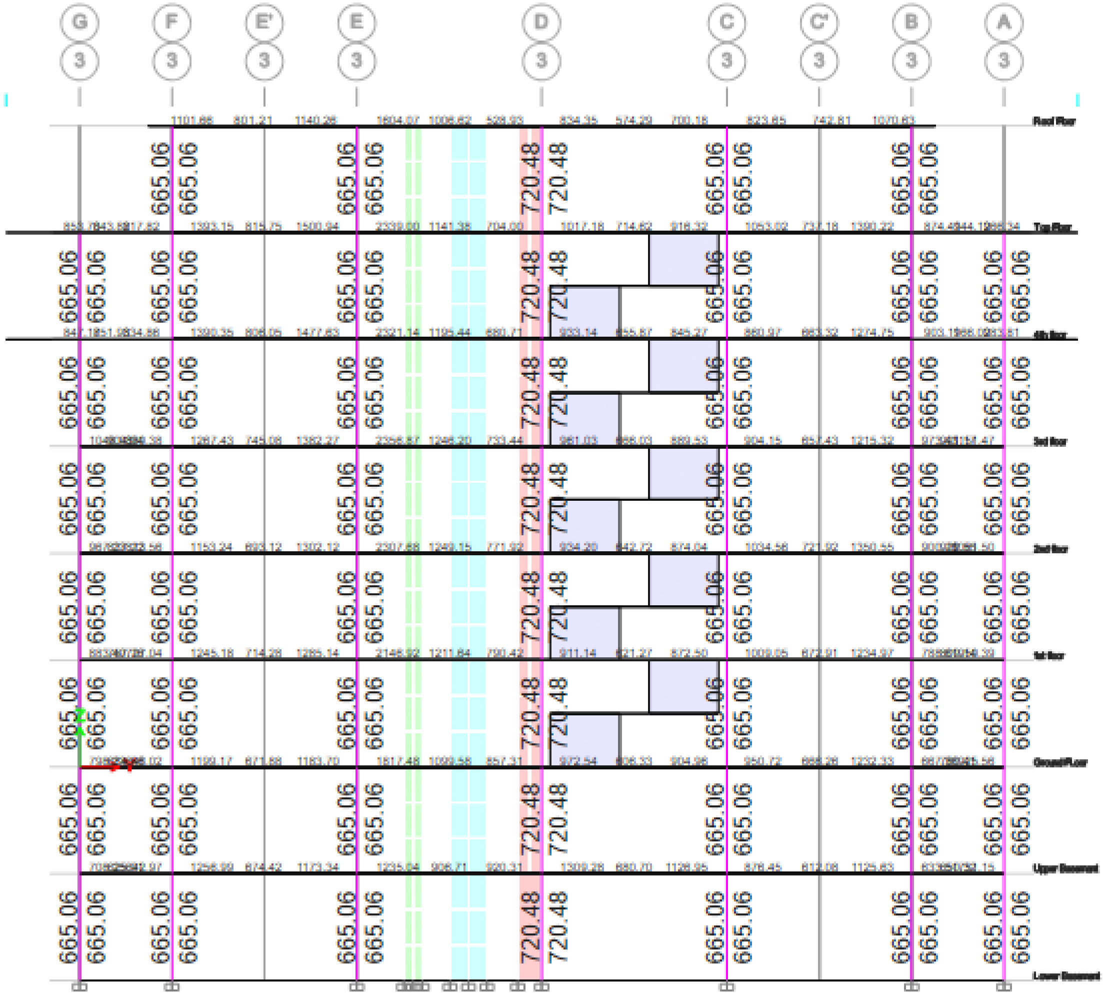
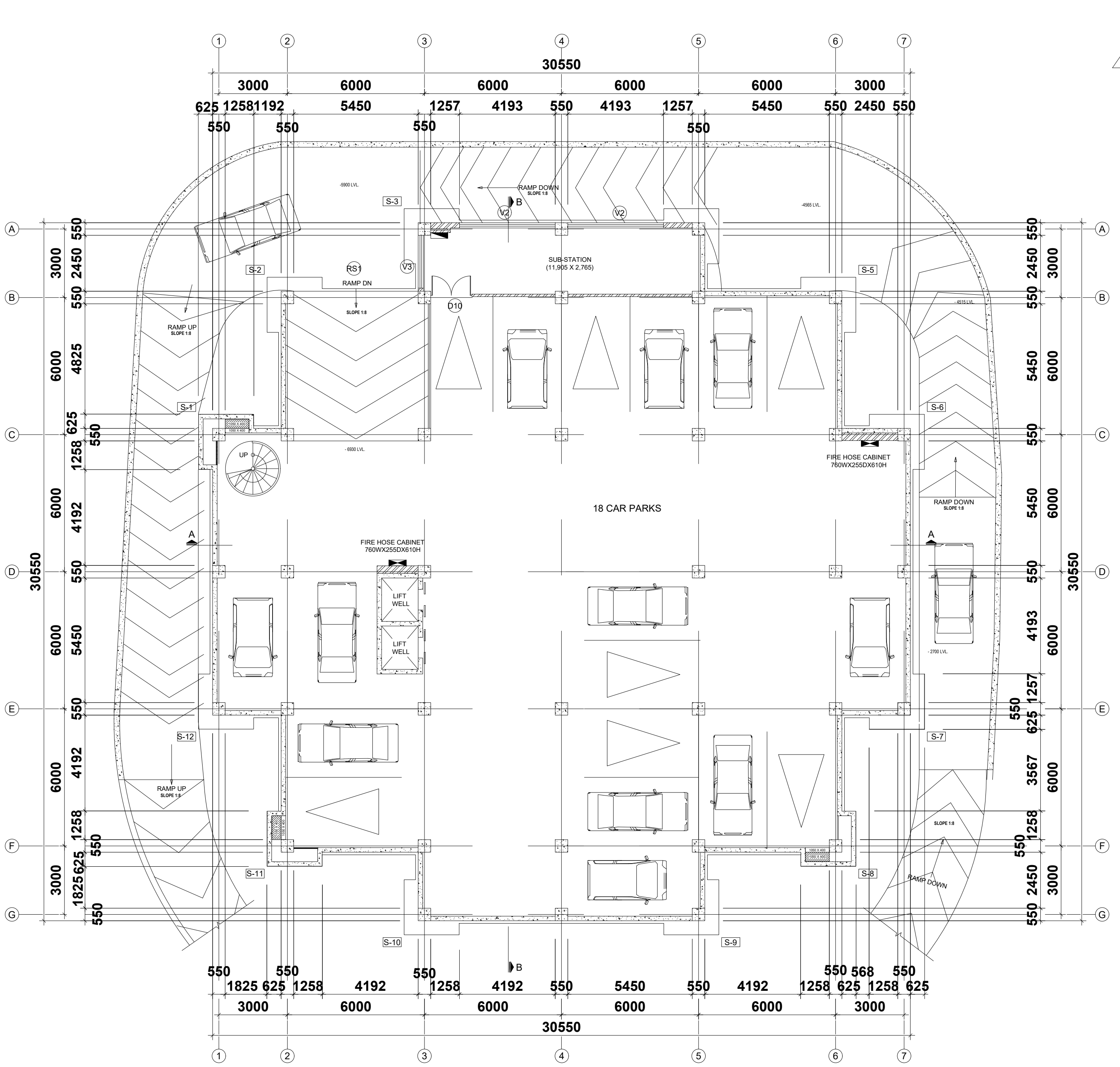
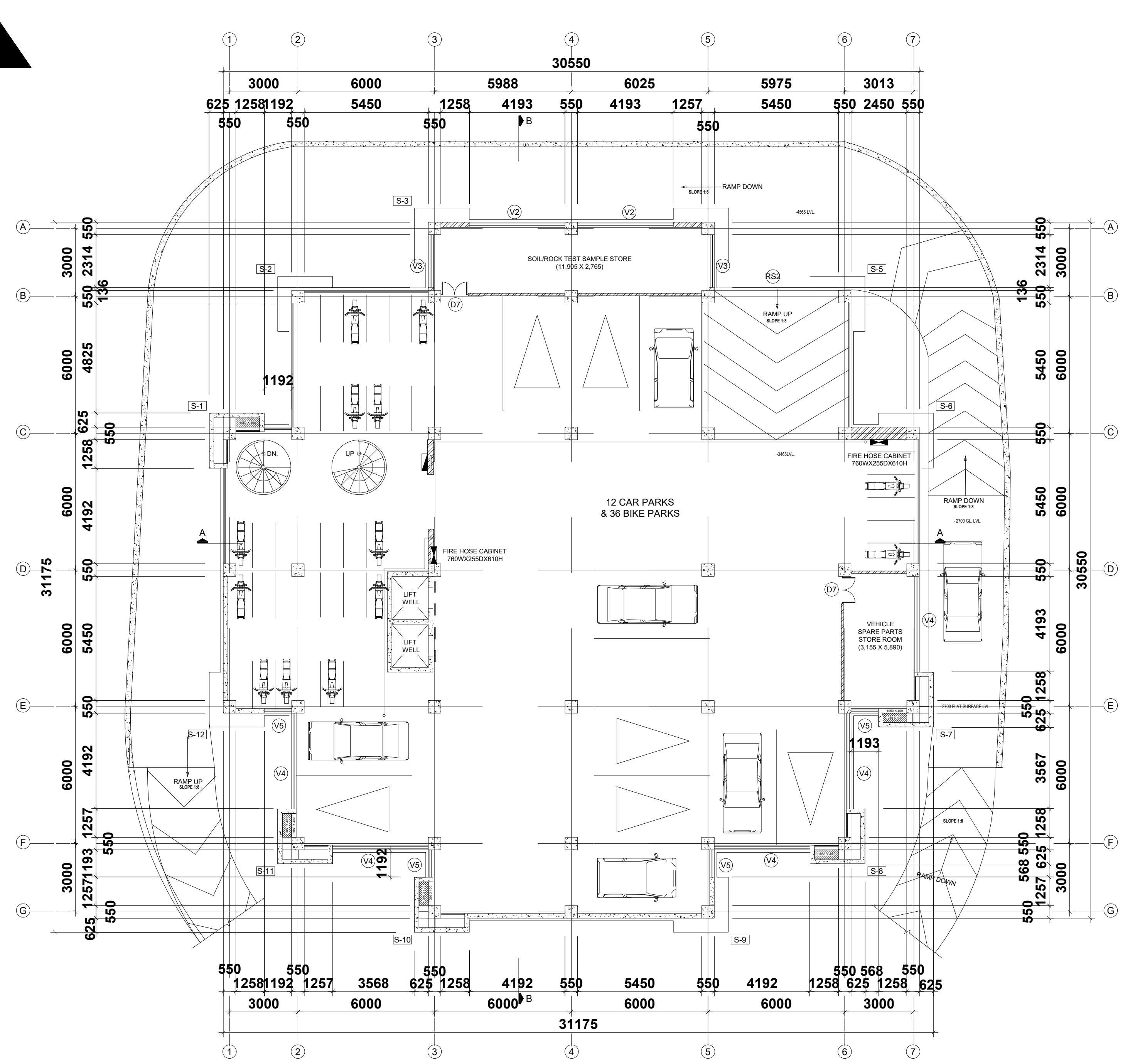


Figure 26 Shear Reinforcement

DRAWINGS



LOWER BASEMENT PLAN (AREA: 761.05 SQ. M.)



UPPER BASEMENT PLAN (AREA: 766.22 SQ. M.)

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Tribhuwan University
Institute of Engineering
Pulchowk Campus
Department of Civil Engineering

Project Title:
Study of Earthquake Resistant Analysis and Detail of Multi-storey Building

Sheet Title:
Floor Plans

Group Members:

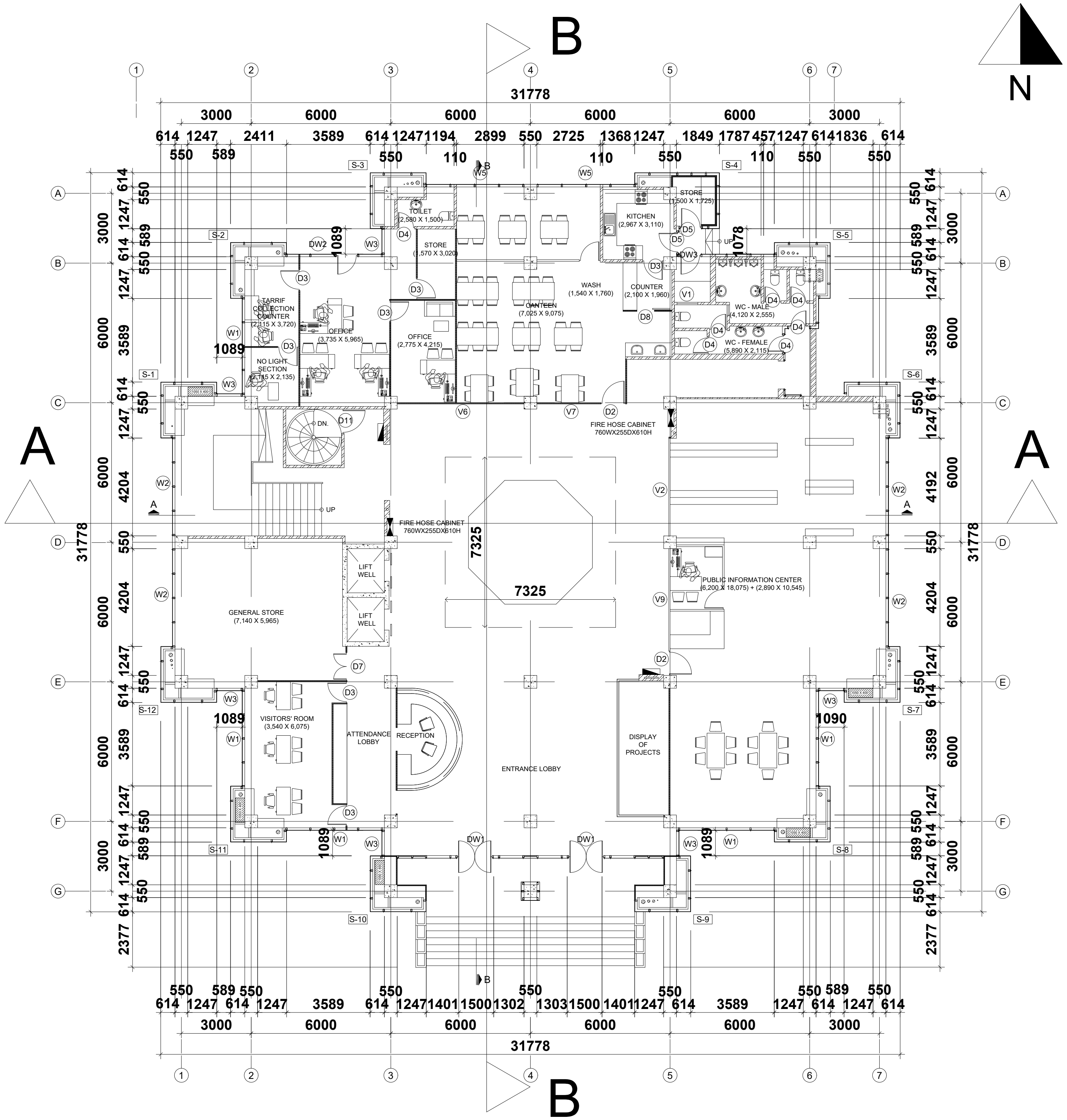
Santosh Pokharel	075BCE146
Saraswati Bhandari	075BCE147
Saroj Basnet	075BCE148
Saugat Dhakal	075BCE149
Shivam Kumar Sah	075BCE156
Uttam Dahal	075BCE189

Project Supervisor
Asst.Prof.Sunita Ghimire

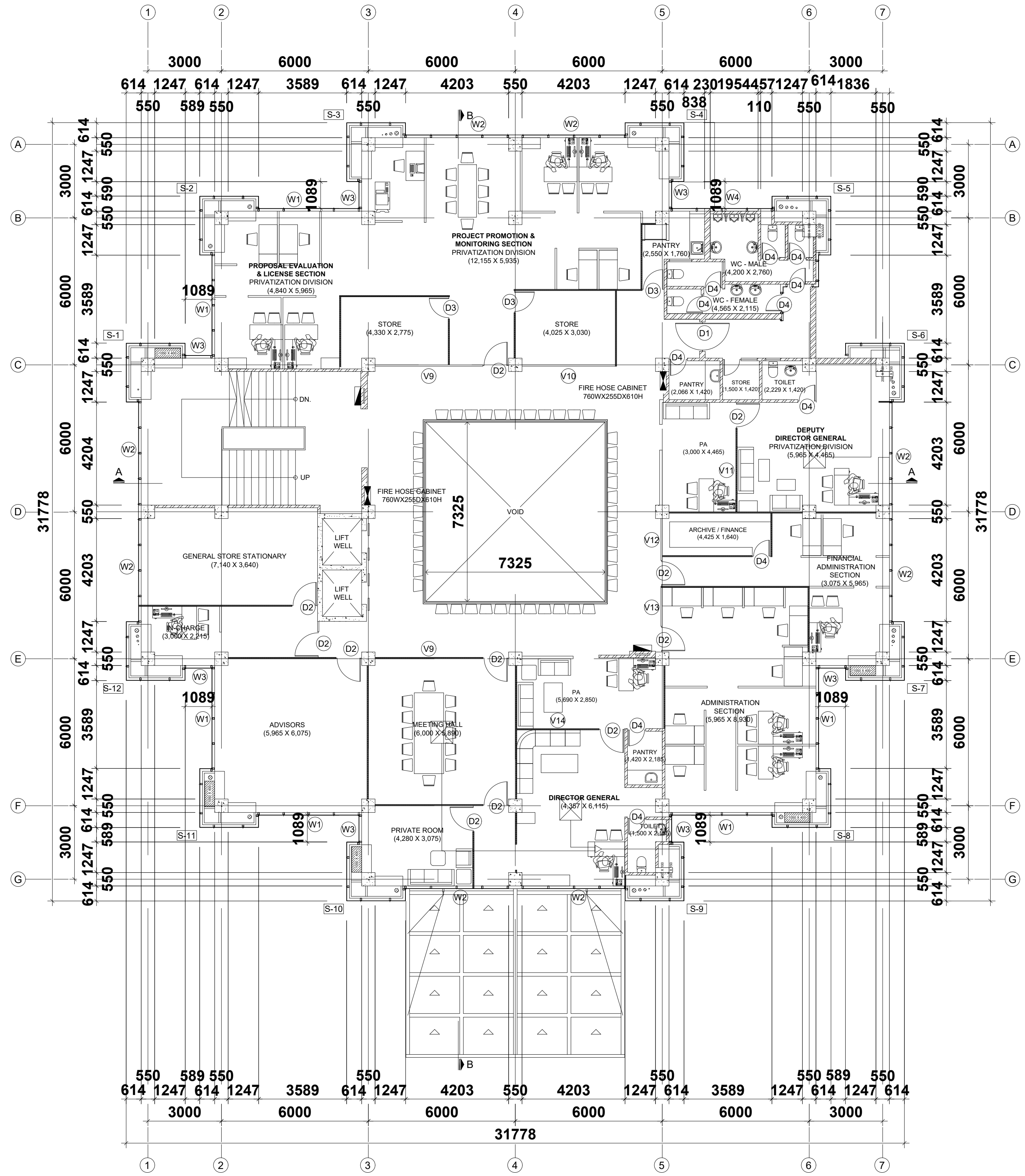
Checked By:

Scale: Fit to scale

Sheet No:
1



GROUND FLOOR PLAN (AREA: 795.61 SQ. M.)



FIRST FLOOR PLAN (AREA: 793.90 SQ. M.)

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Department of Civil Engineering</td>
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<td>Sheet Title:
Floor Plans</td>
<td>Group Members:
Santosh Pokharel 075BCE146
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Uttam Dahal 075BCE189</td>
<td>Project Supervisor
Asst.Prof.Sunita Ghimire

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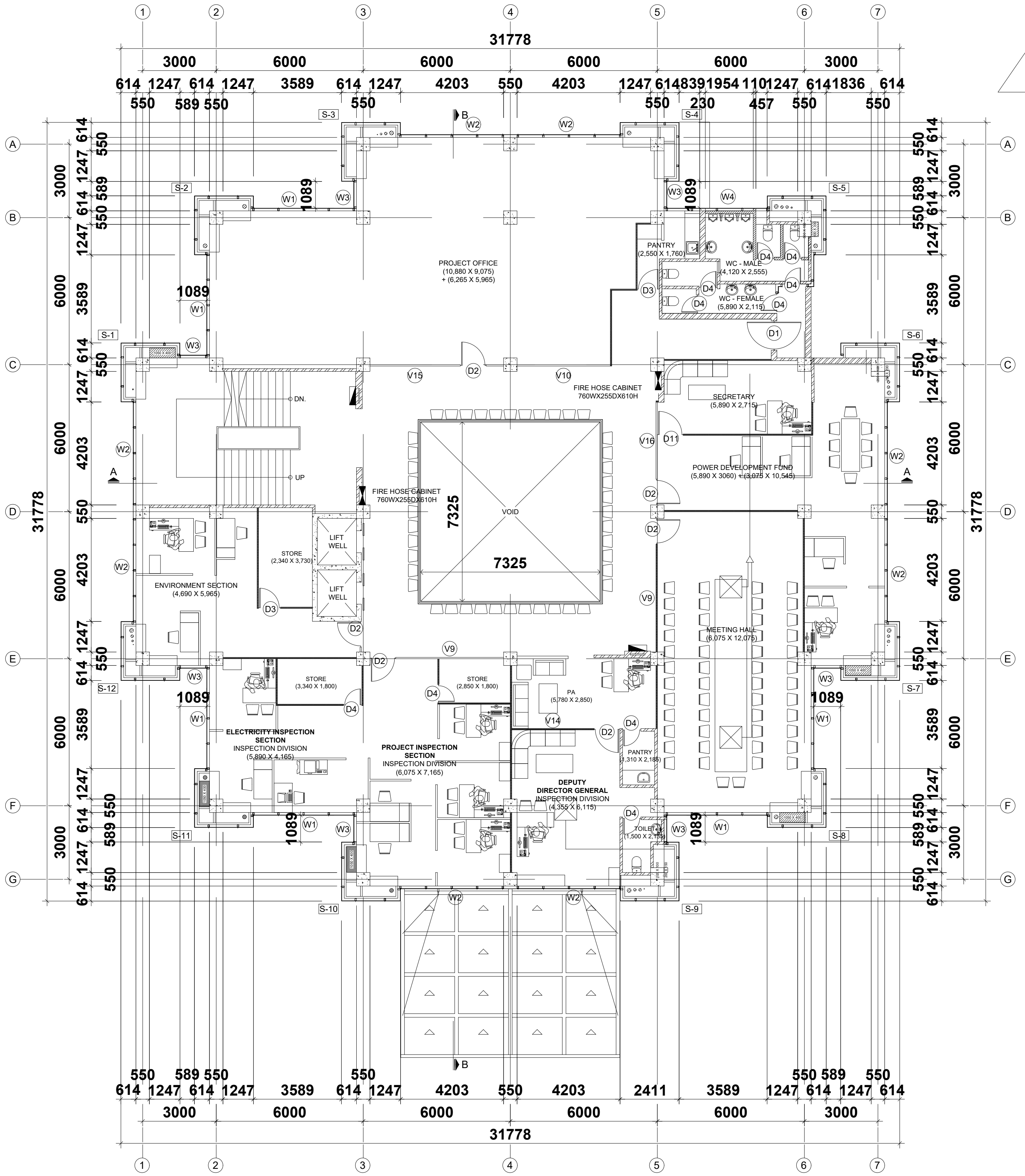
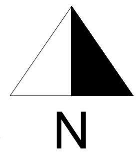
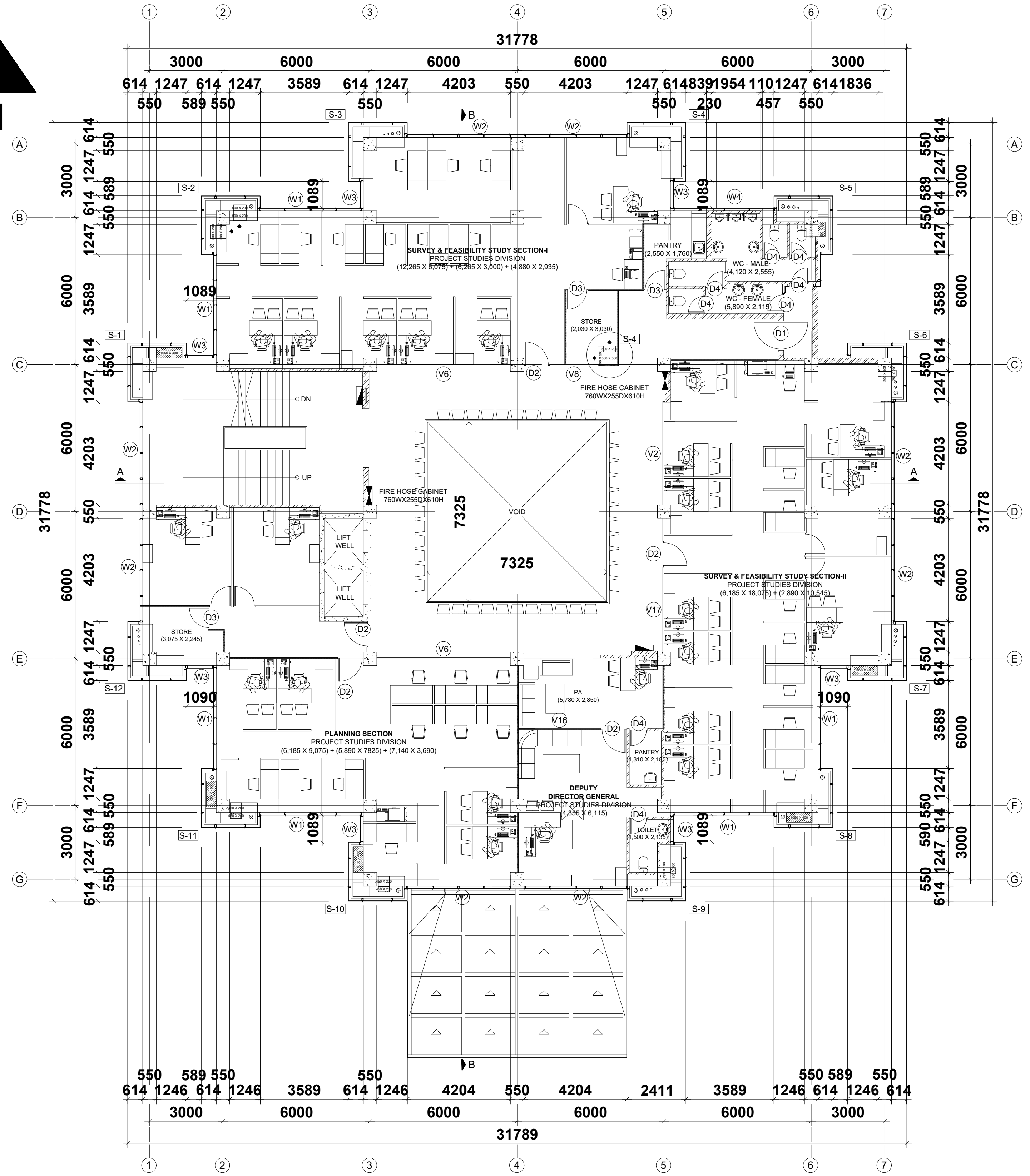


SECOND FLOOR PLAN (AREA: 793.90 SQ. M.)





THIRD FLOOR PLAN (AREA: 793.90 SQ. M.)

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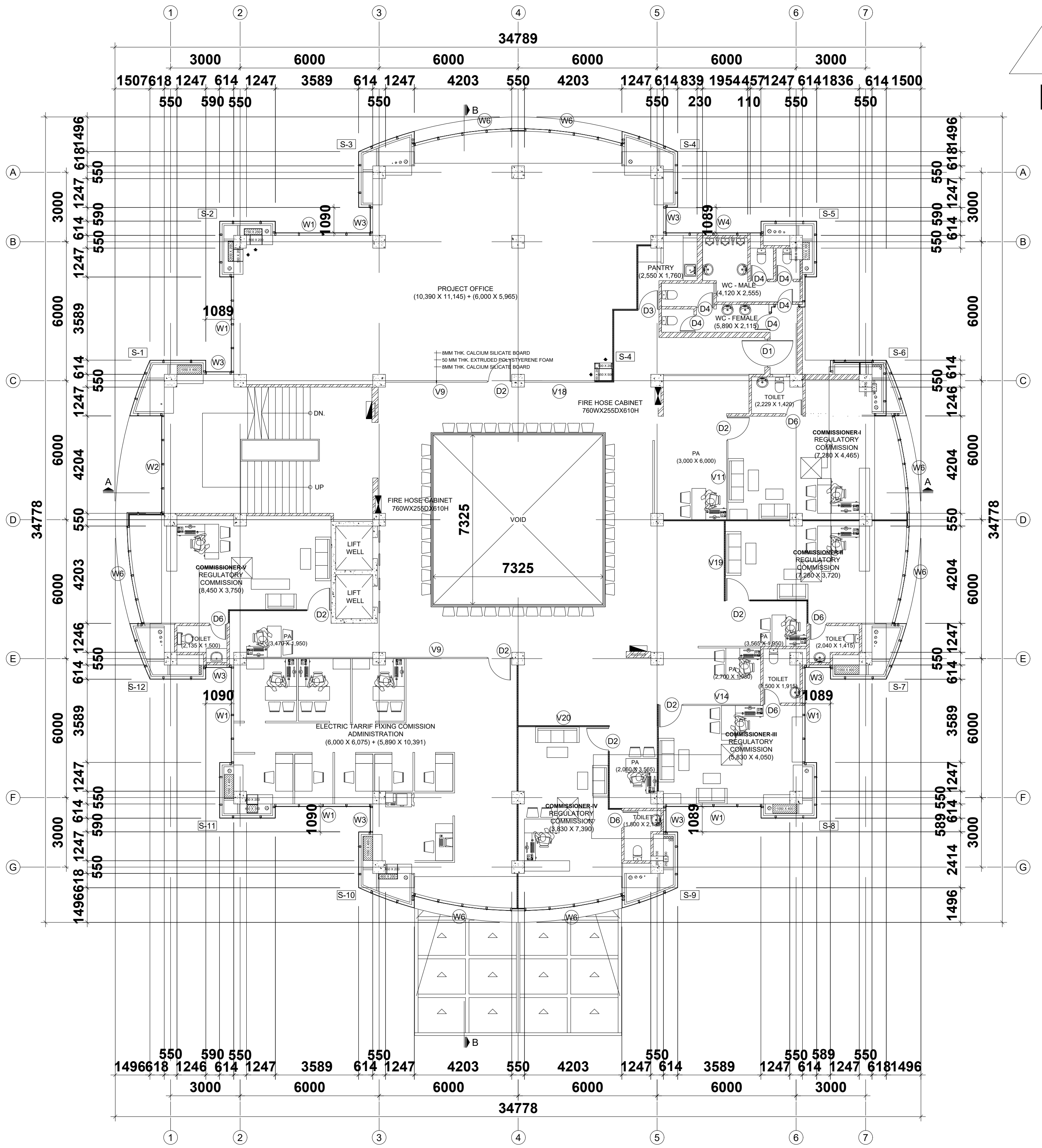
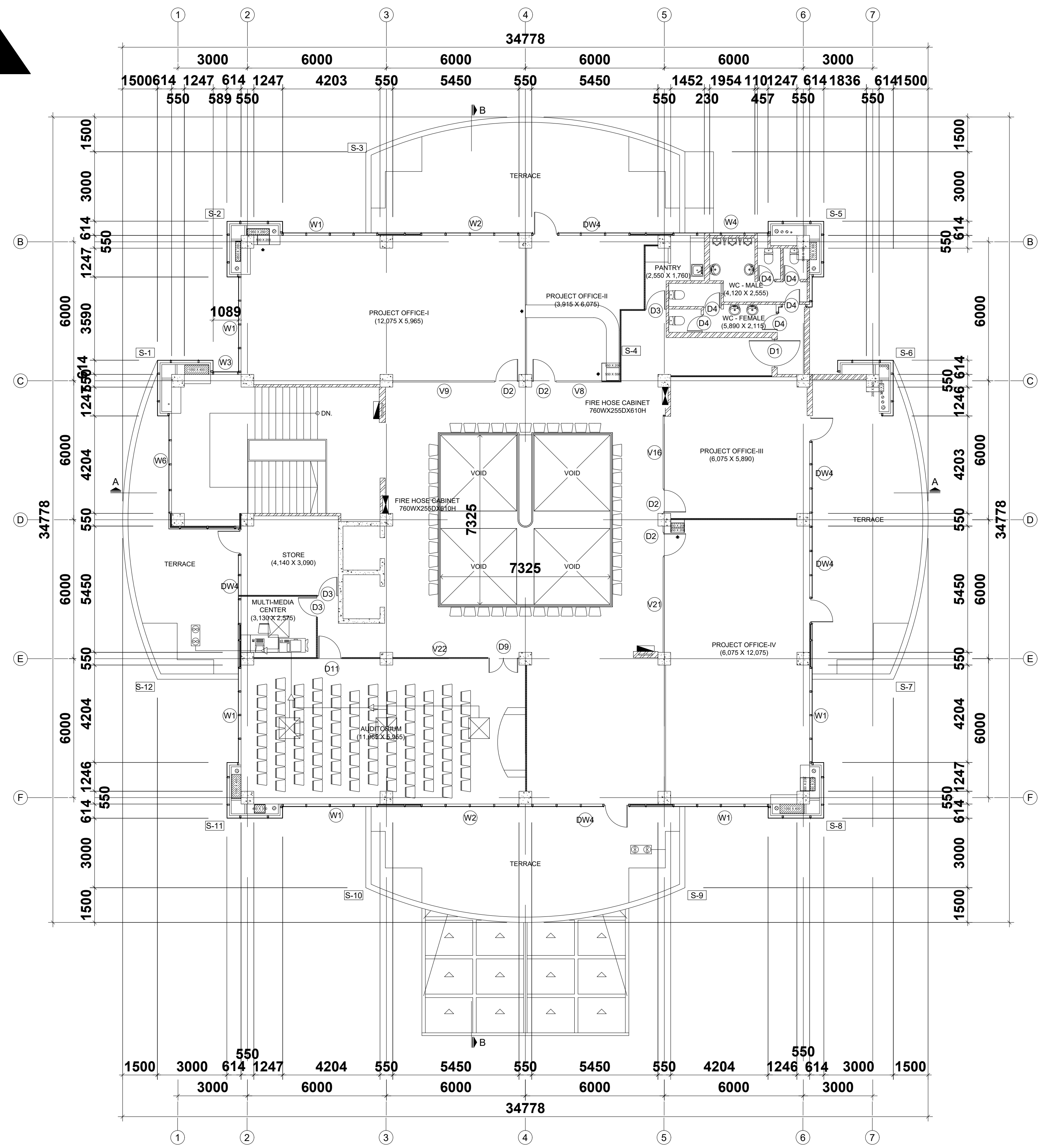


FOURTH FLOOR PLAN (AREA: 849.15 SQ. M.)



TOP FLOOR PLAN (AREA: 653.66 SQ. M.)

ALL DIMENSIONS ARE IN MM

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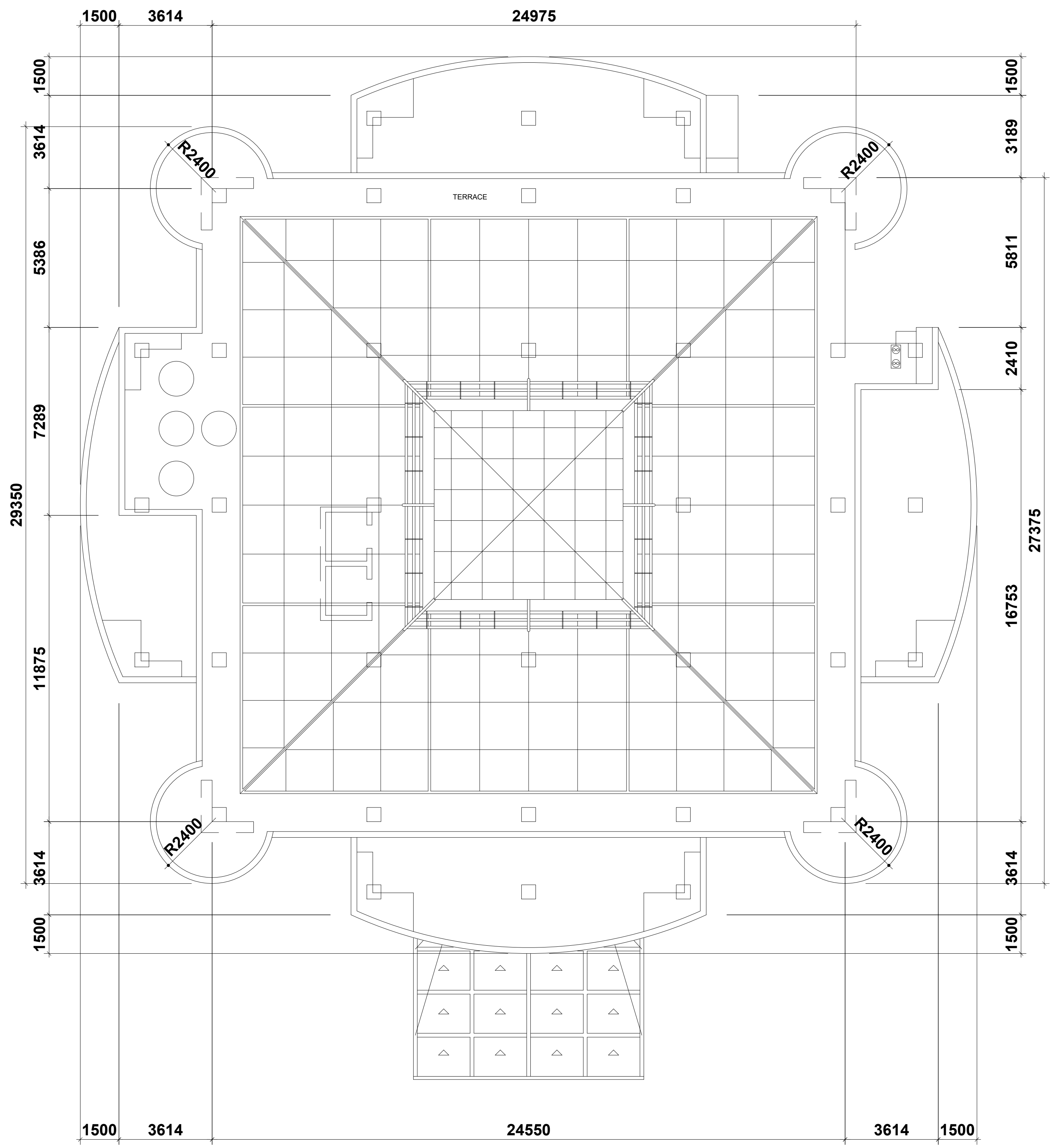

ROOF PLAN

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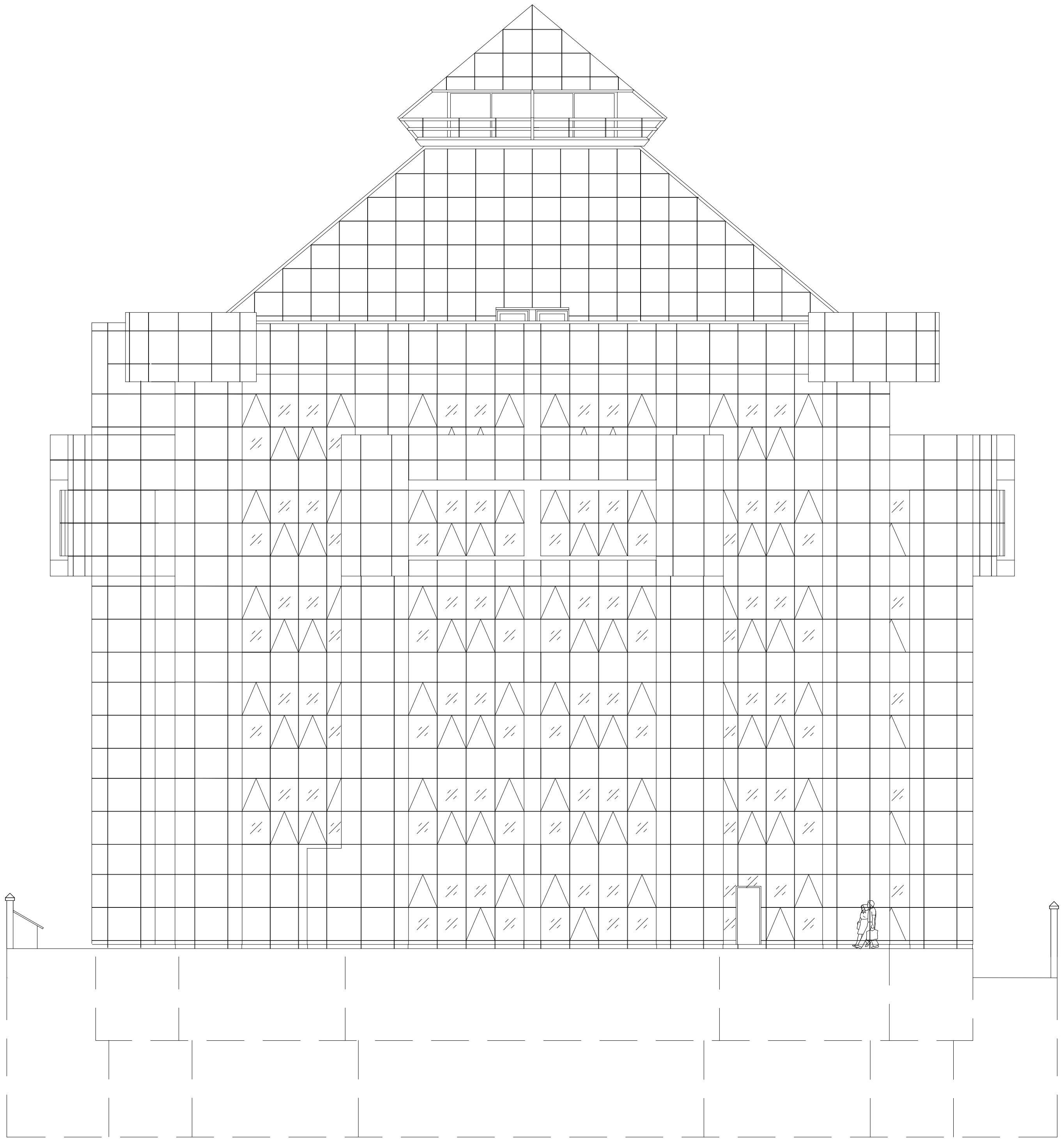
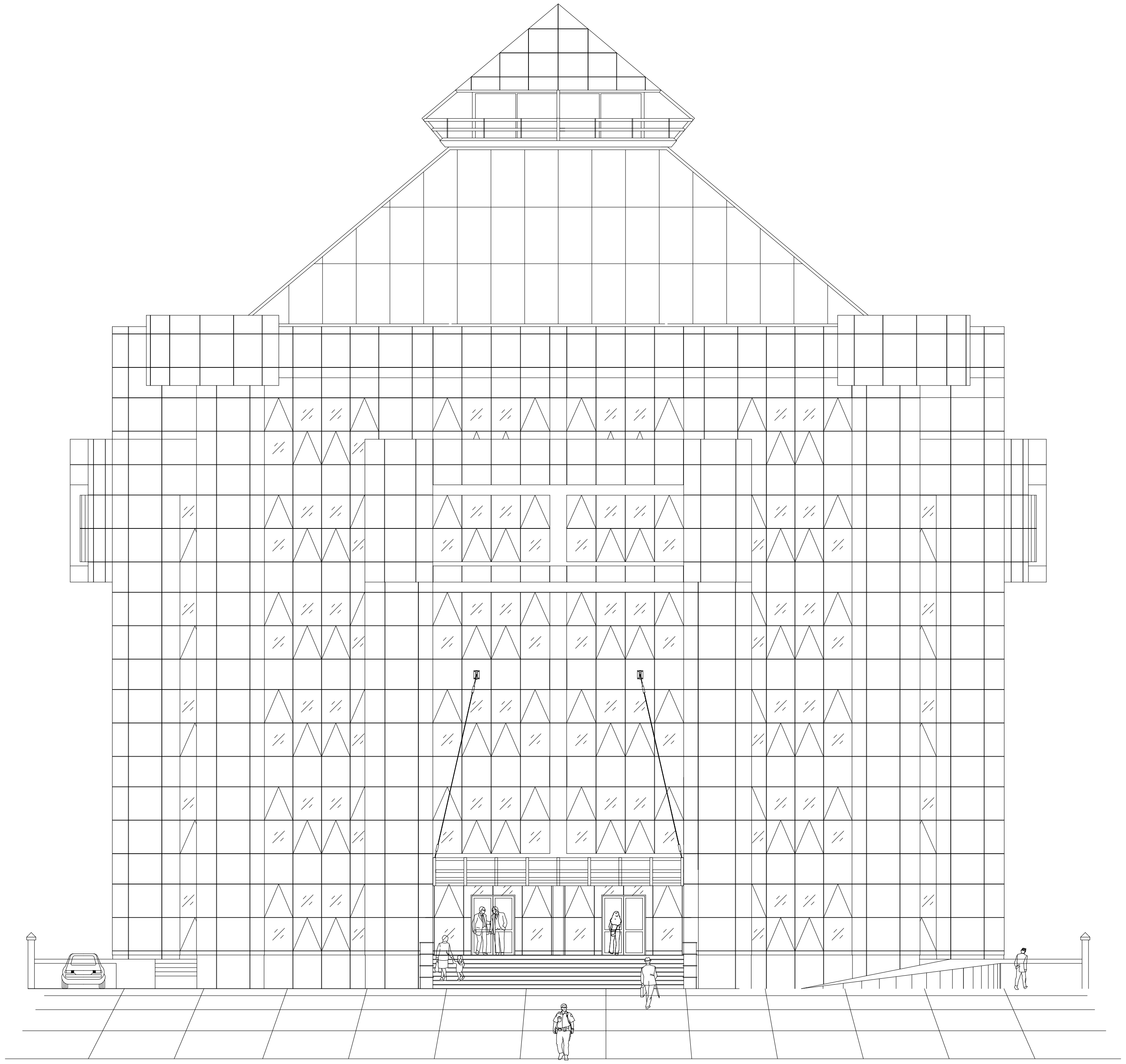
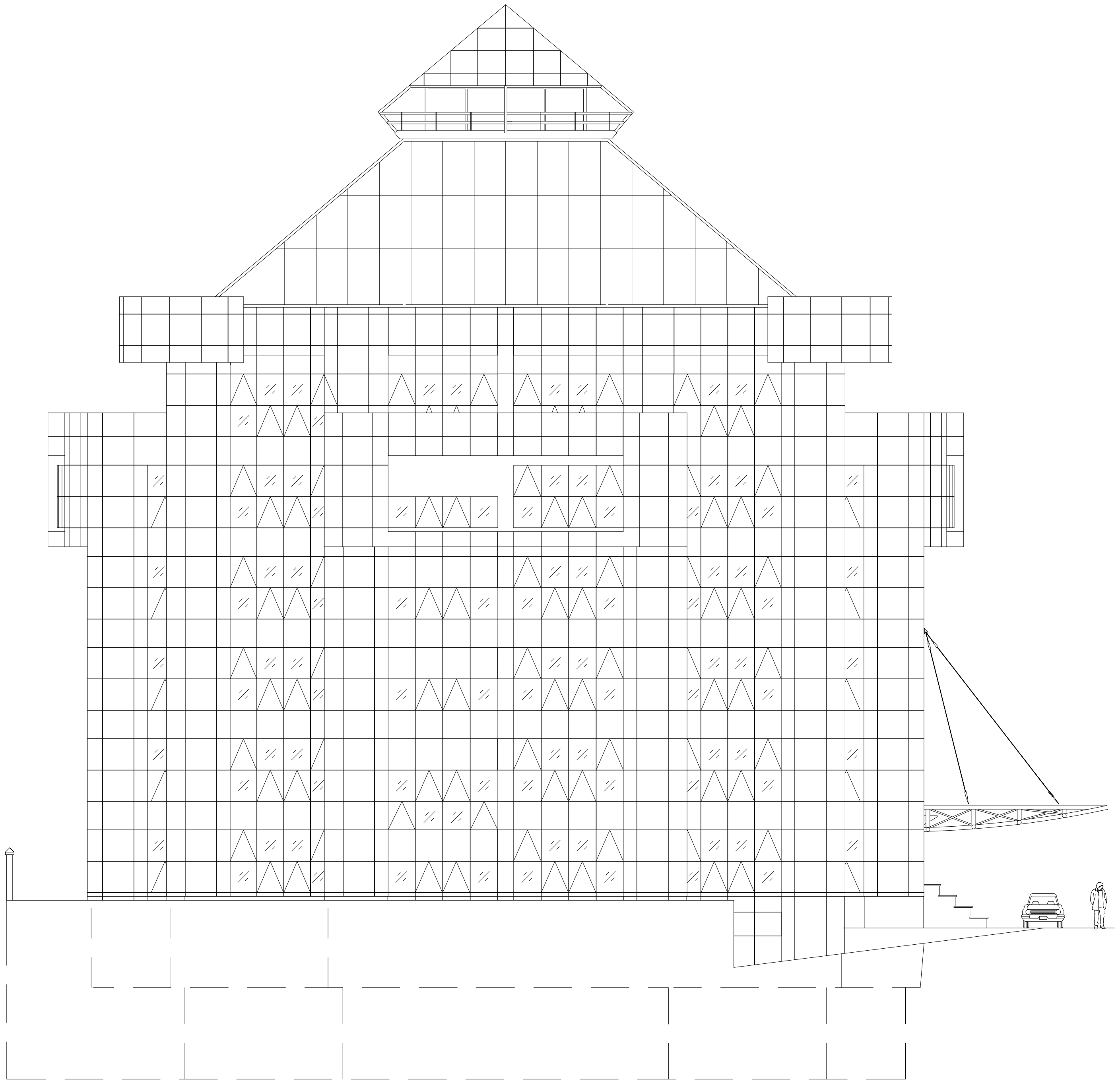


NORTH ELEVAION

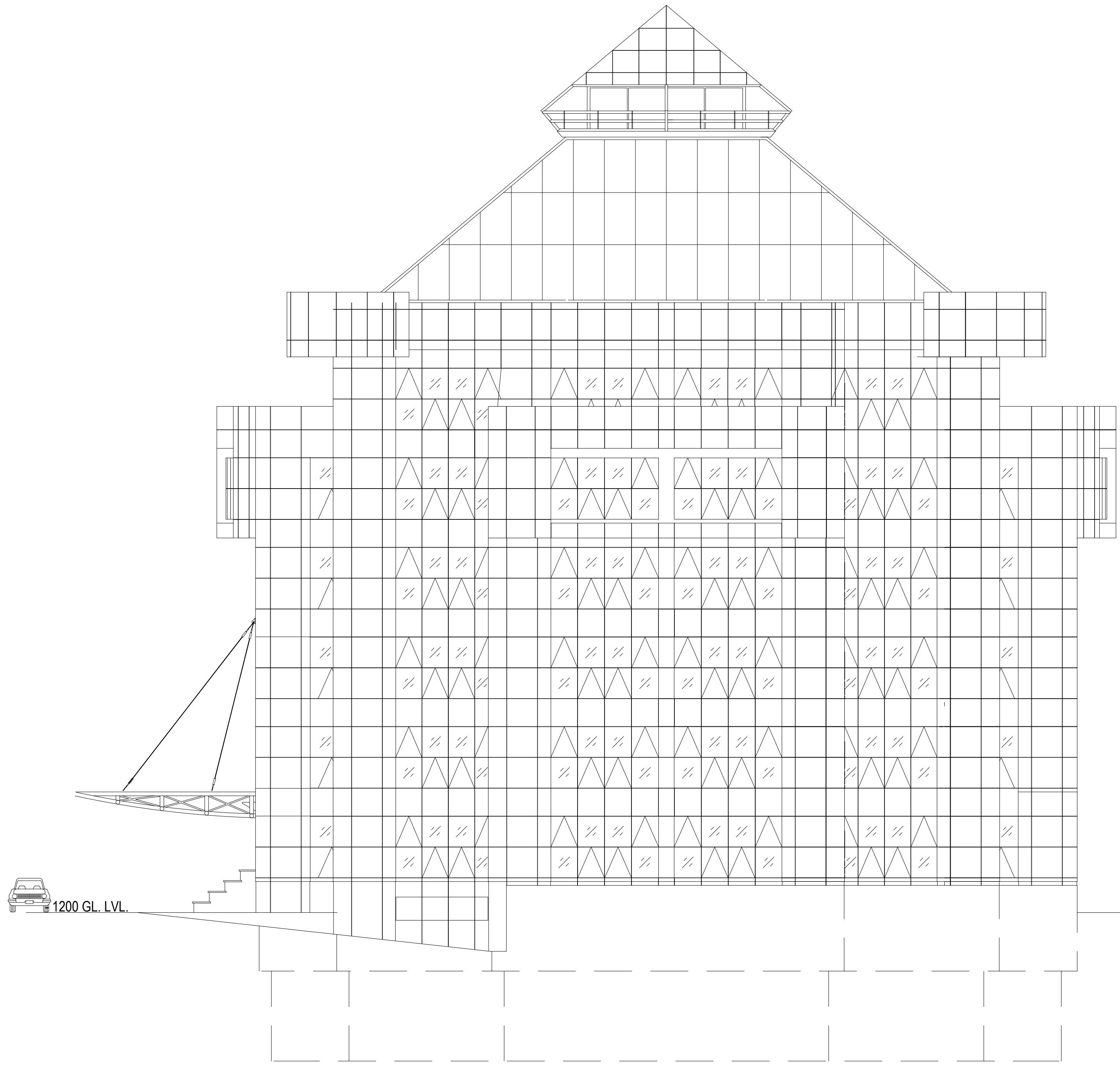


SOUTH ELEVAION

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Detail of Multi-storey Building**	Sheet Title: **Building's Elevation**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 6

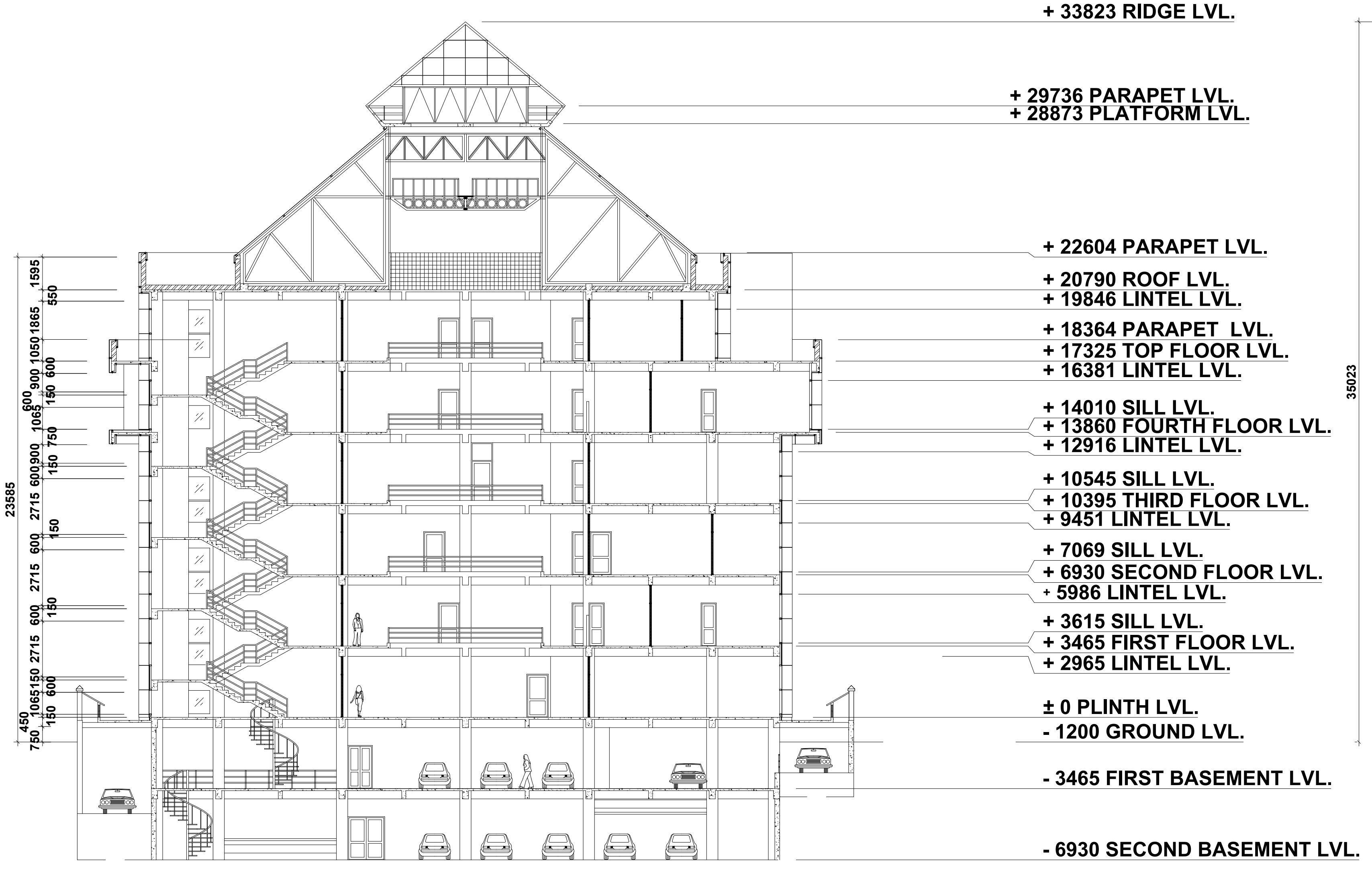


WEST ELEVAION

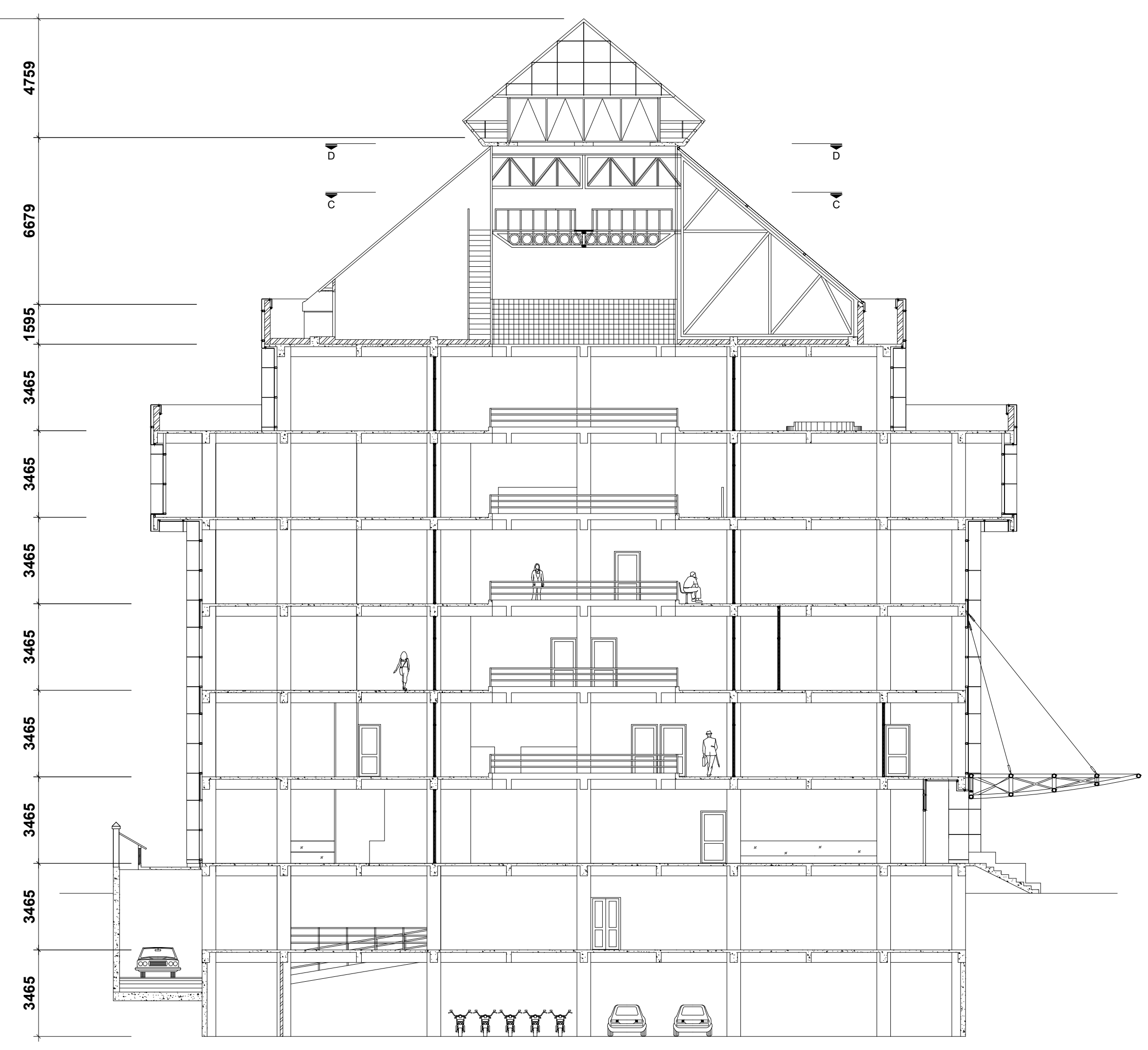


EAST ELEVAION

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Detail of Multi-storey Building**	Sheet Title: **Building's Elevation**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 7



SECTION AT A-A



SECTION AT B-B

ALL DIMENSIONS ARE IN MM

Tribhuwan University
Institute of Engineering
Pulchowk Campus
Department of Civil Engineering

Project Title:
Study of Earthquake Resistant Analysis and Detail of Multi-storey Building

Sheet Title:
Vertical Sections of Building

Group Members:

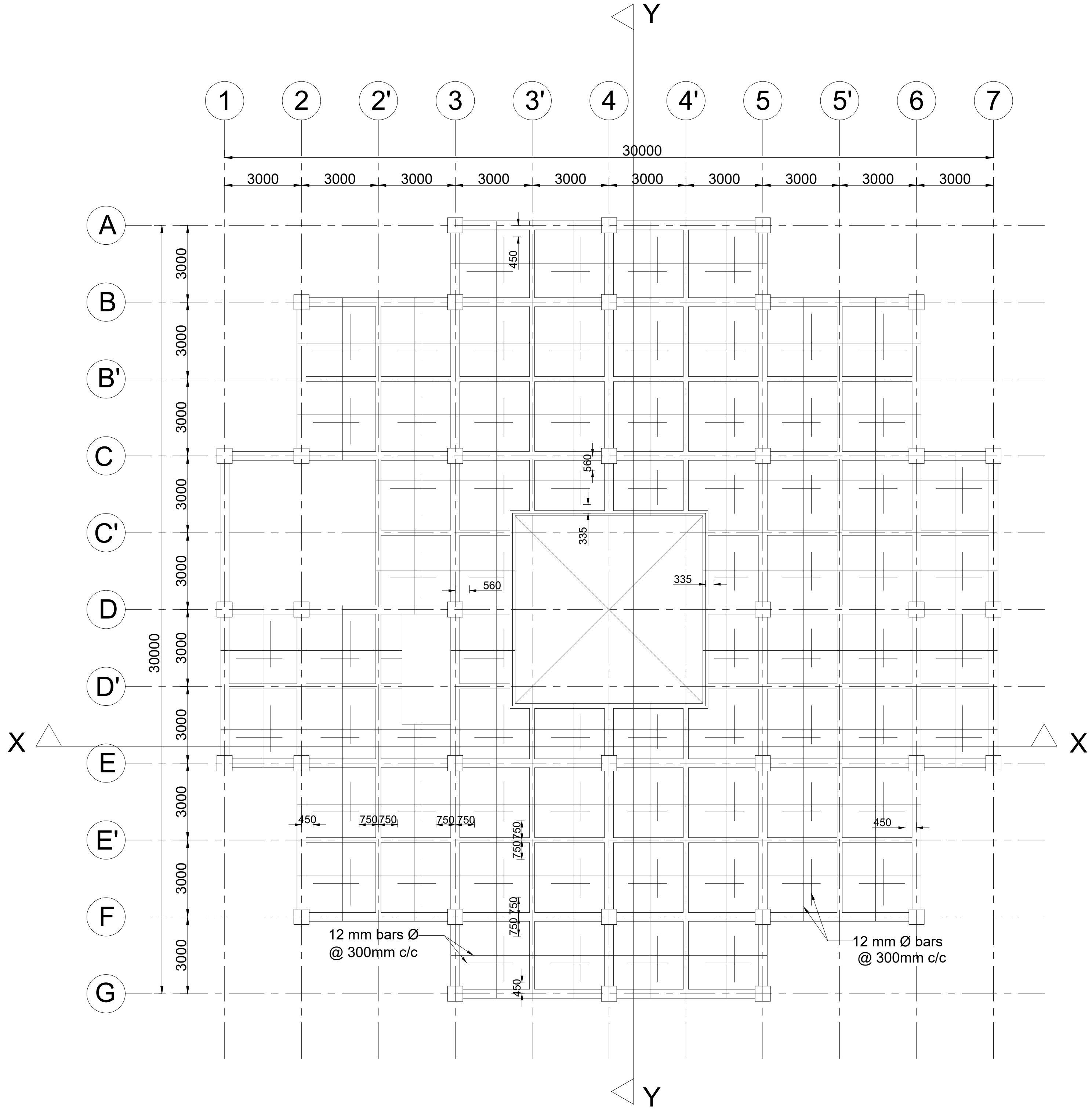
Santosh Pokharel	075BCE146
Saraswati Bhandari	075BCE147
Saroj Basnet	075BCE148
Saugat Dhakal	075BCE149
Shivam Kumar Sah	075BCE156
Uttam Dahal	075BCE189

Project Supervisor
Asst.Prof.Sunita Ghimire

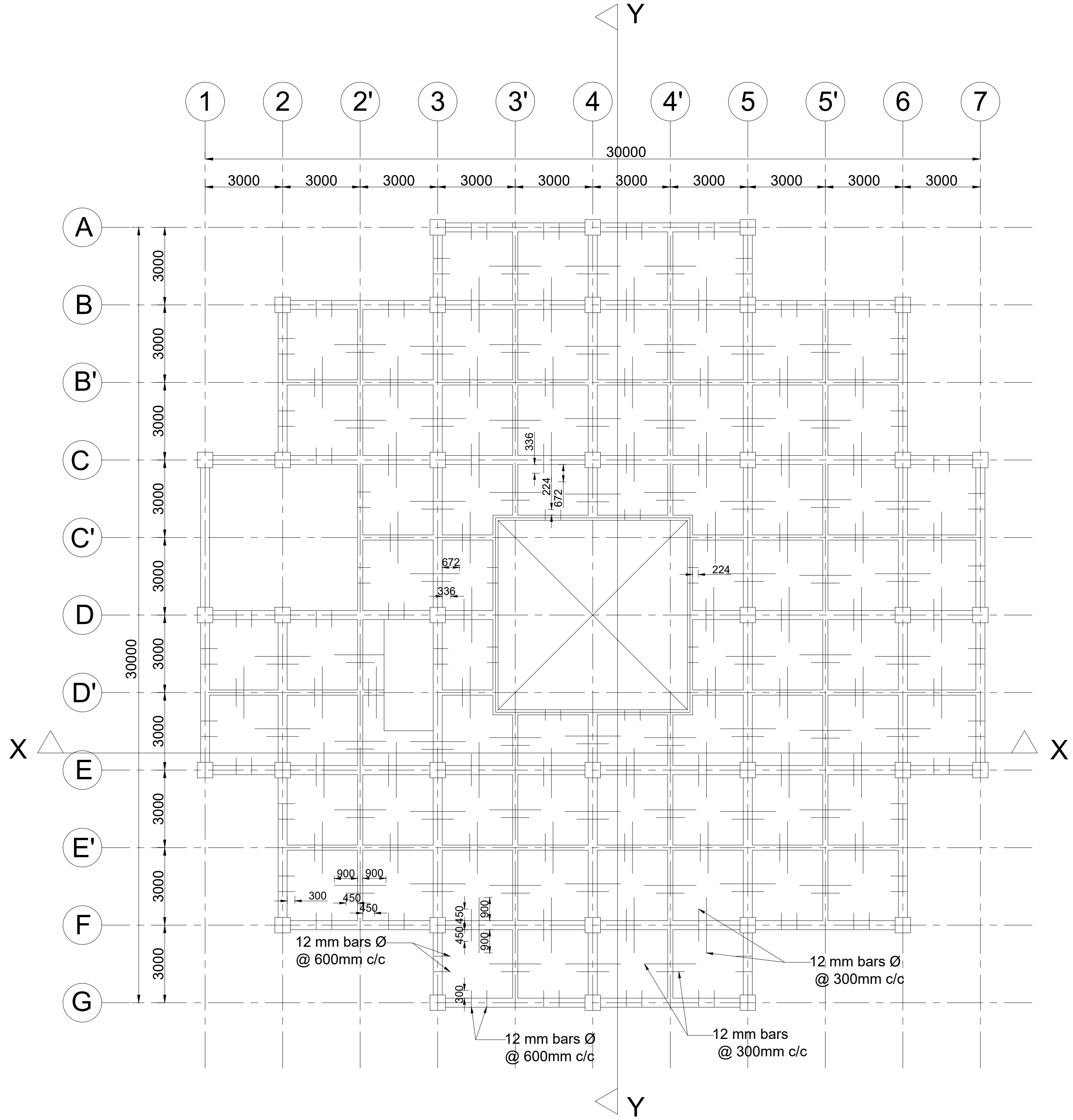
Checked By:

Scale: Fit to scale

Sheet No:
8



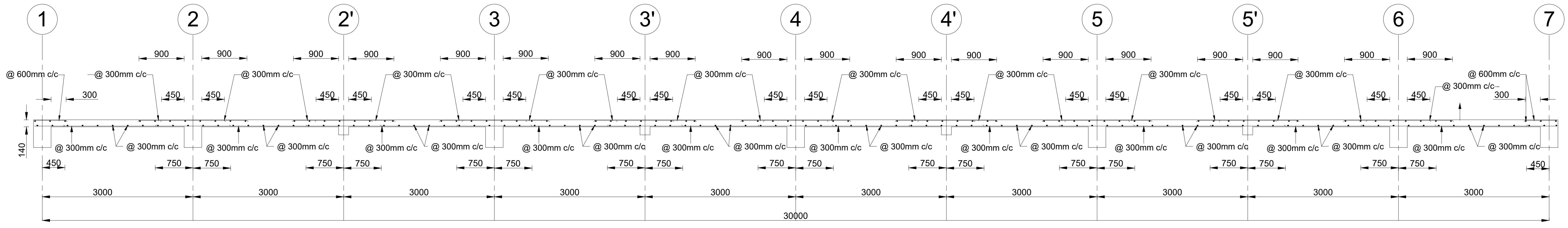
SECOND FLOOR BOTTOM REINFORCEMENT PLAN



SECOND FLOOR TOP REINFORCEMENT PLAN

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **2nd Floor Top and Bottom Reinforcement Plans of Slab**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 9

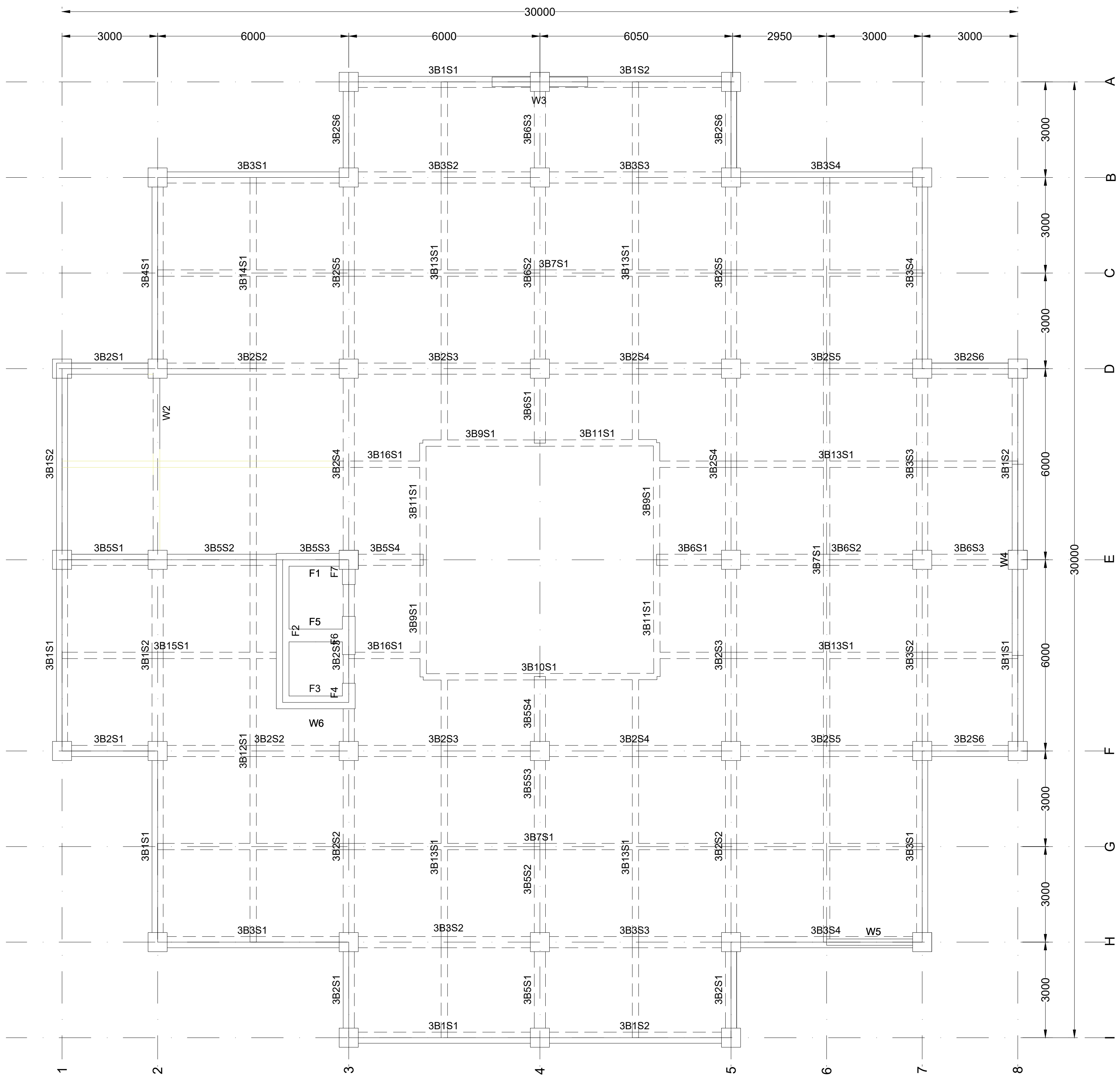


SECTION ALONG X-X

SECTION ALONG Y-Y

ALL DIMENSIONS ARE IN MM

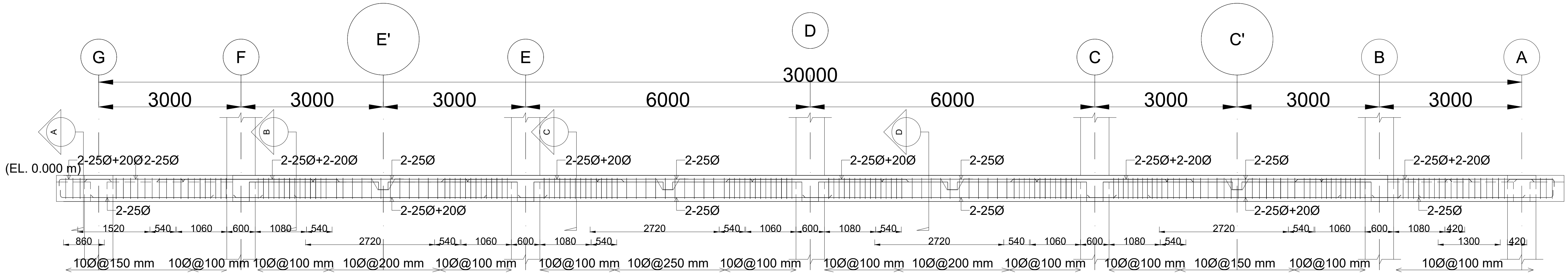
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Slab Sections**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 10



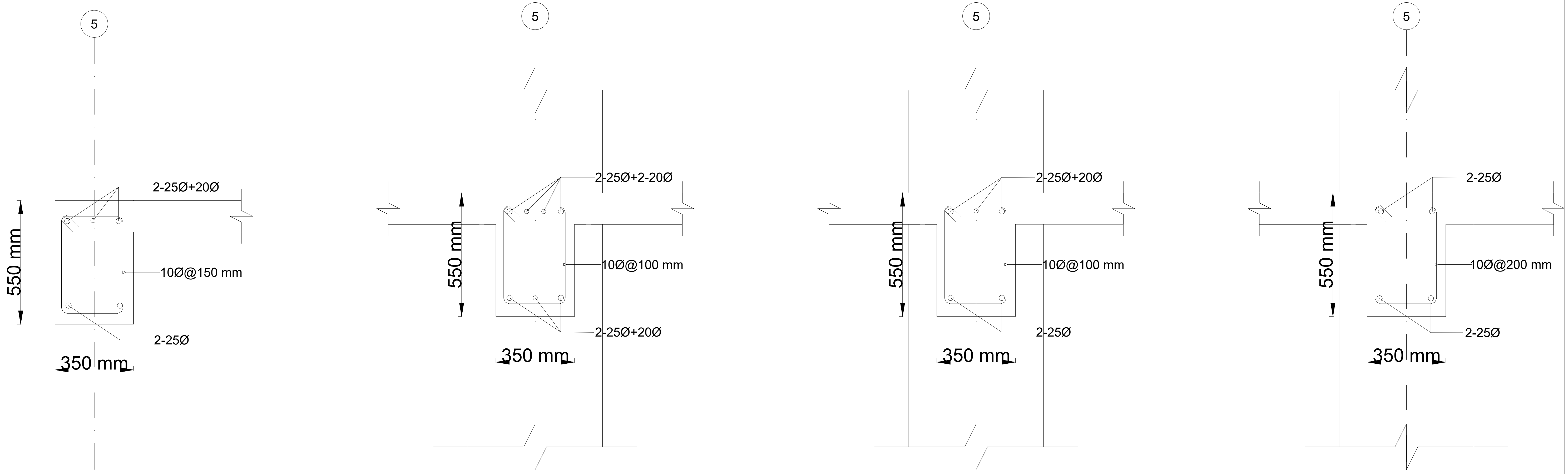
CONCRETE BEAM LAYOUT PLAN

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Concrete Beam Layout**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Shah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 11



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF GROUND FLOOR



7CB10:SECTION A

7CB10:SECTION B

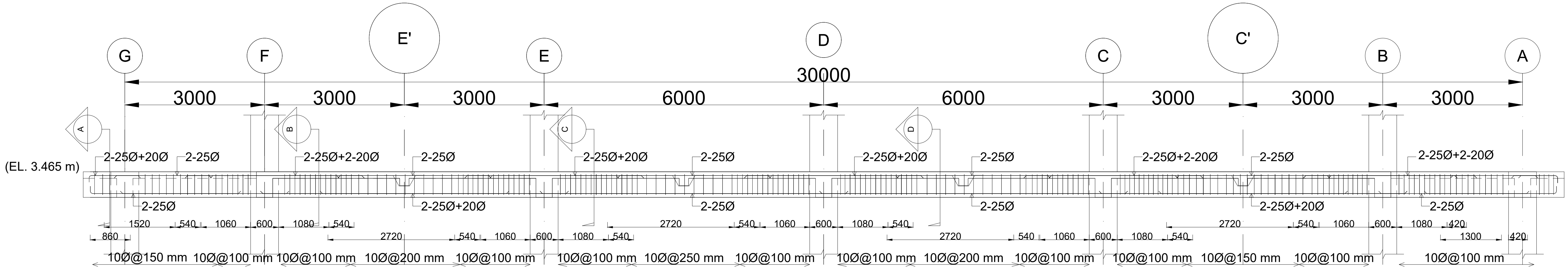
7CB10:SECTION C

7CB10:SECTION D

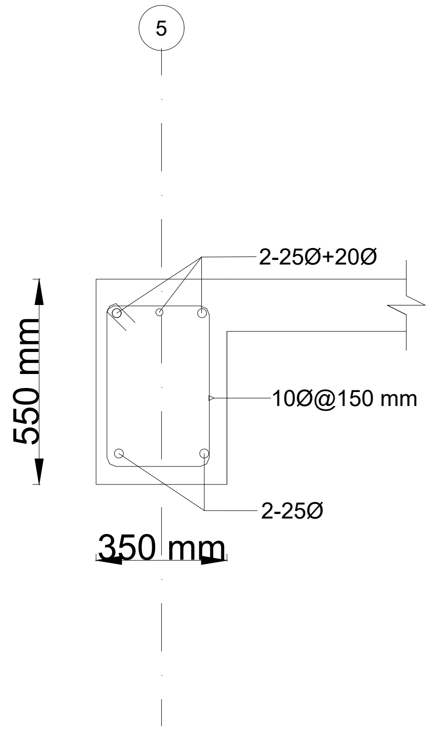
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF GROUND FLOOR

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 5-5 of Ground Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 12



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF FIRST FLOOR



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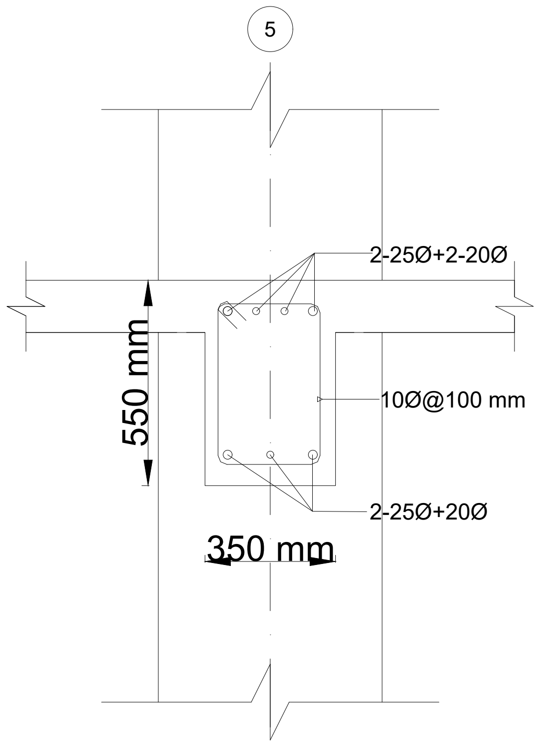

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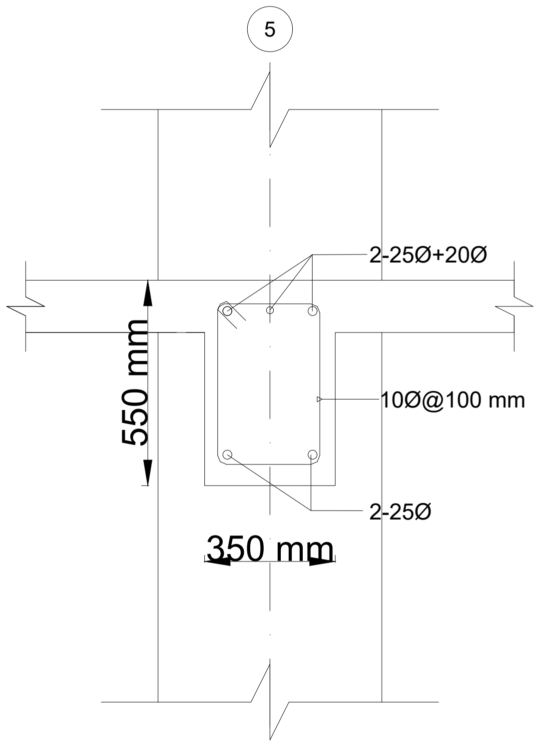

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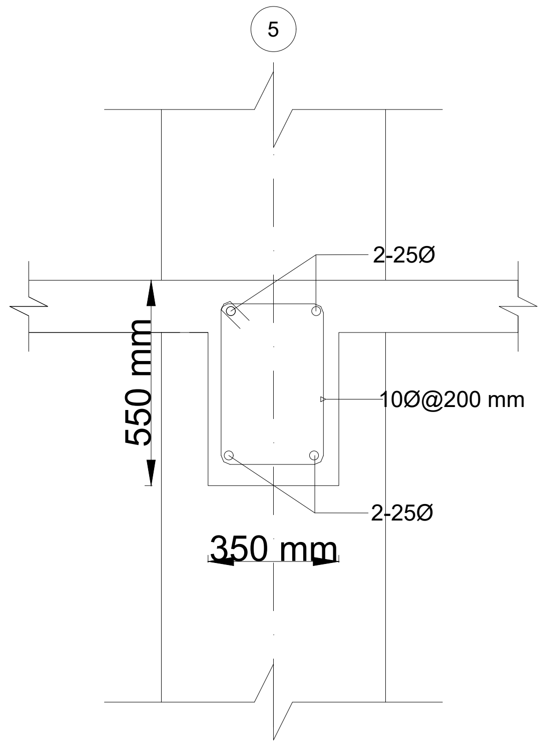


7CB10:SECTION D

REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF FIRST FLOOR

ALL DIMENSIONS ARE IN MM

Tribhuwan University
Institute of Engineering
Pulchowk Campus
Department of Civil Engineering

Project Title:
Study of Earthquake Resistant Analysis and Design of Multi-storey Building

Sheet Title:
Reinforcement Detailing of Beam along 5-5 of First Floor

Group Members:

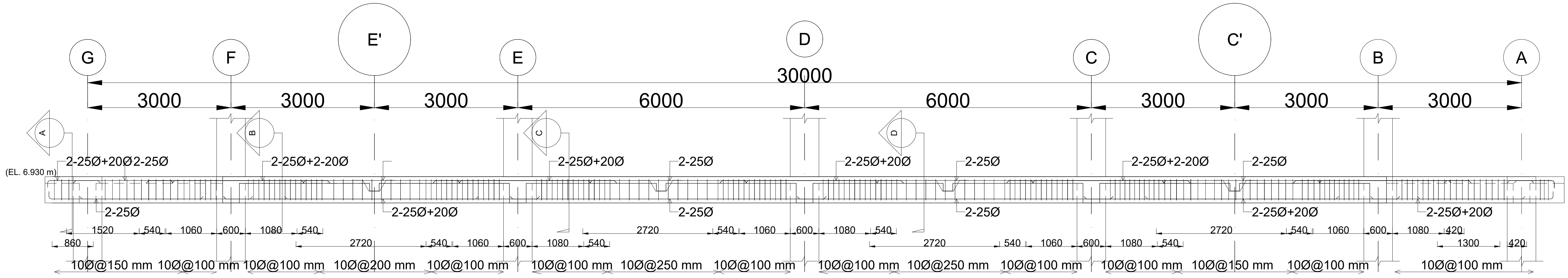
Santosh Pokharel	075BCE146
Saraswati Bhandari	075BCE147
Saroj Basnet	075BCE148
Saugat Dhakal	075BCE149
Shivam Kumar Sah	075BCE156
Uttam Dahal	075BCE189

Project Supervisor
Asst.Prof.Sunita Ghimire

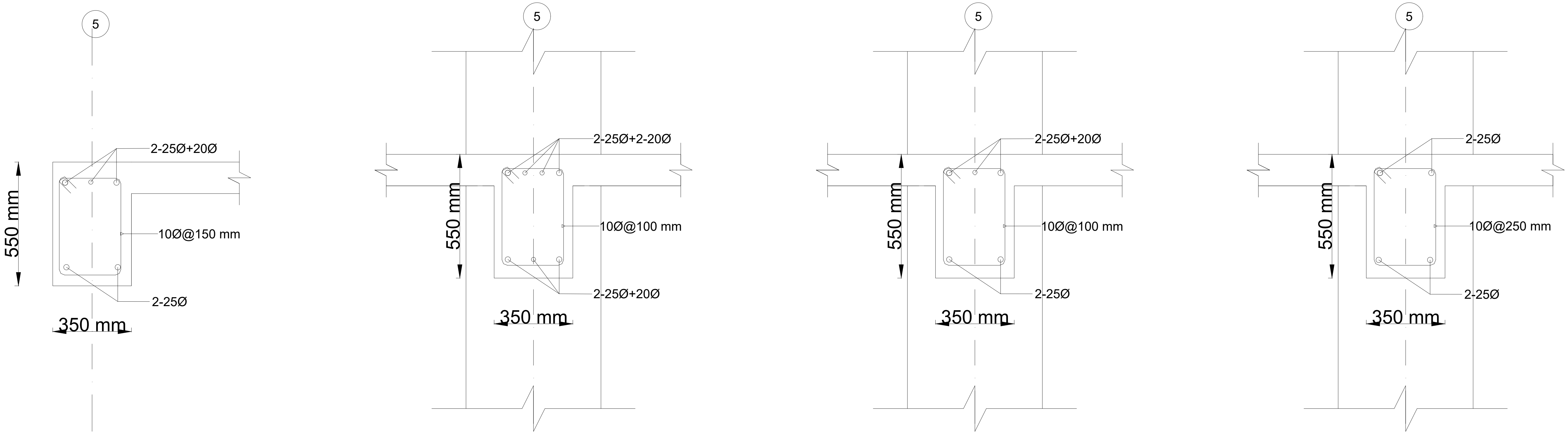
Checked By:

Scale: Fit to scale

Sheet No:
13



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF SECOND FLOOR

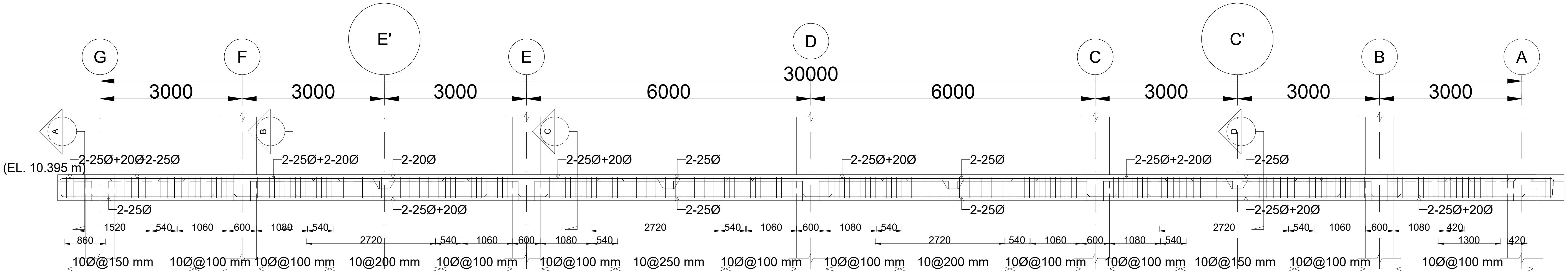


7CB10:SECTION A   7CB10:SECTION B   7CB10:SECTION C   7CB10:SECTION D

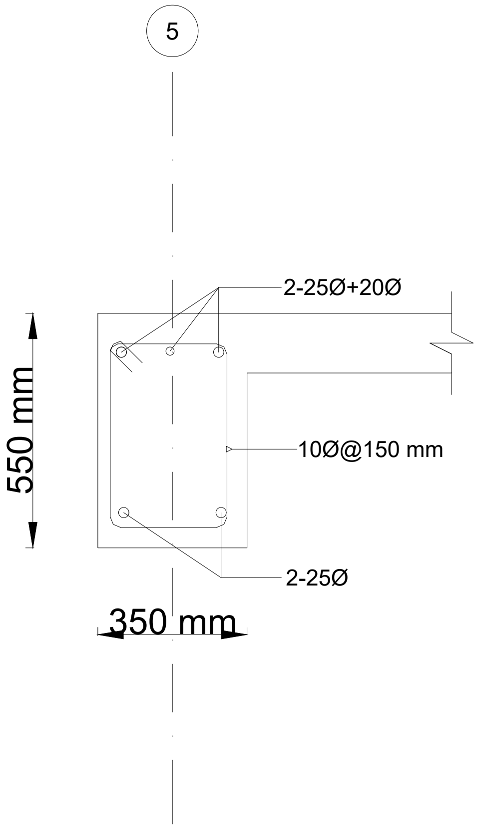
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF SECOND FLOOR

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 5-5 of Second Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 14



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF THIRD FLOOR



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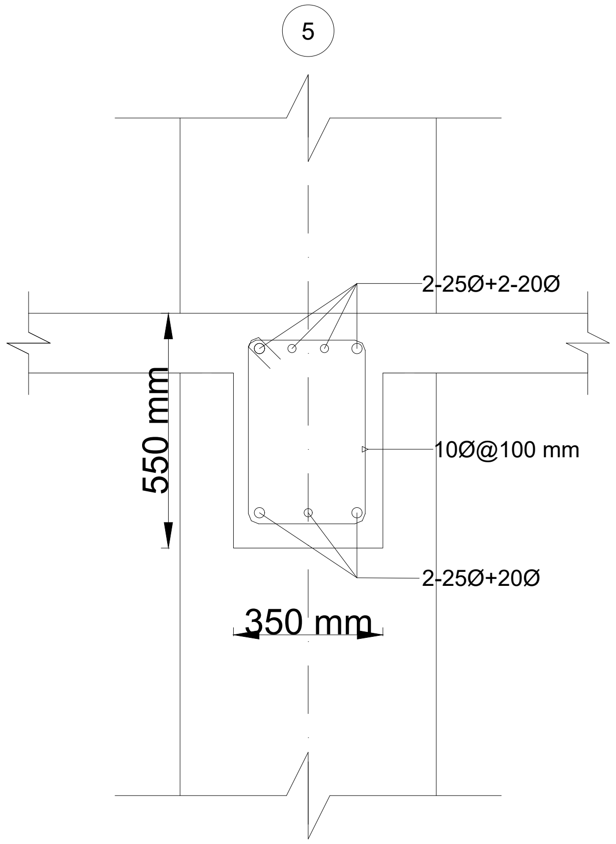

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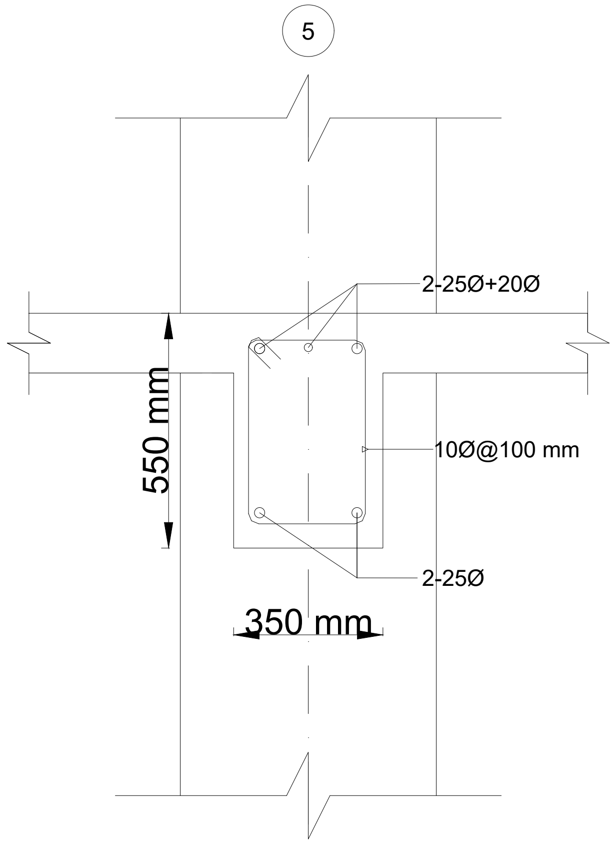

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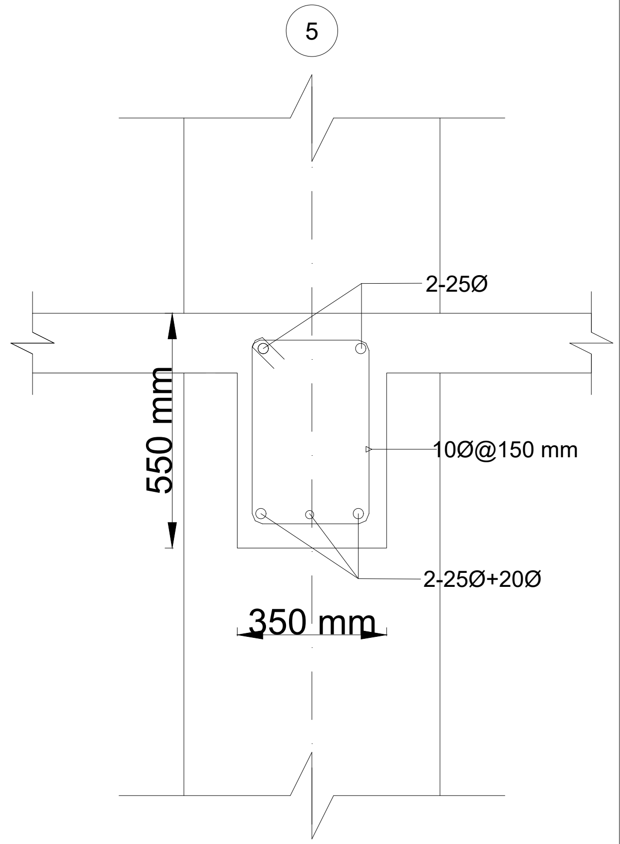
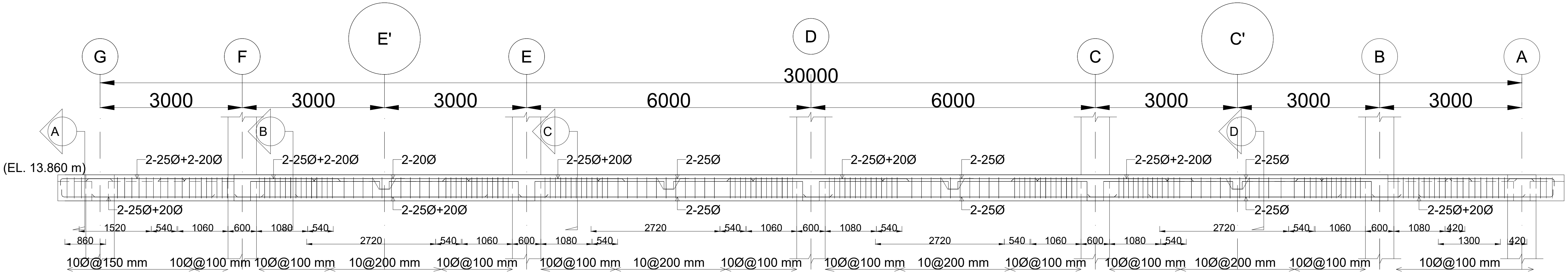


7CB10:SECTION D

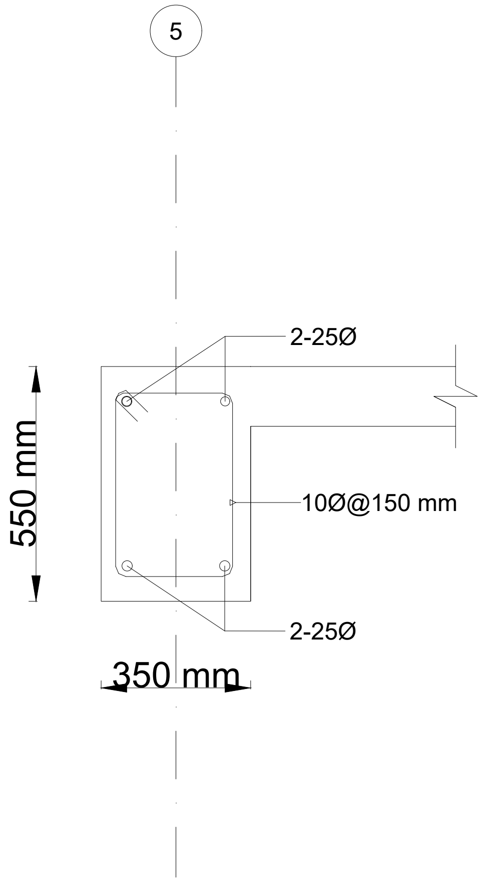
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF THIRD FLOOR

ALL DIMENSIONS ARE IN MM

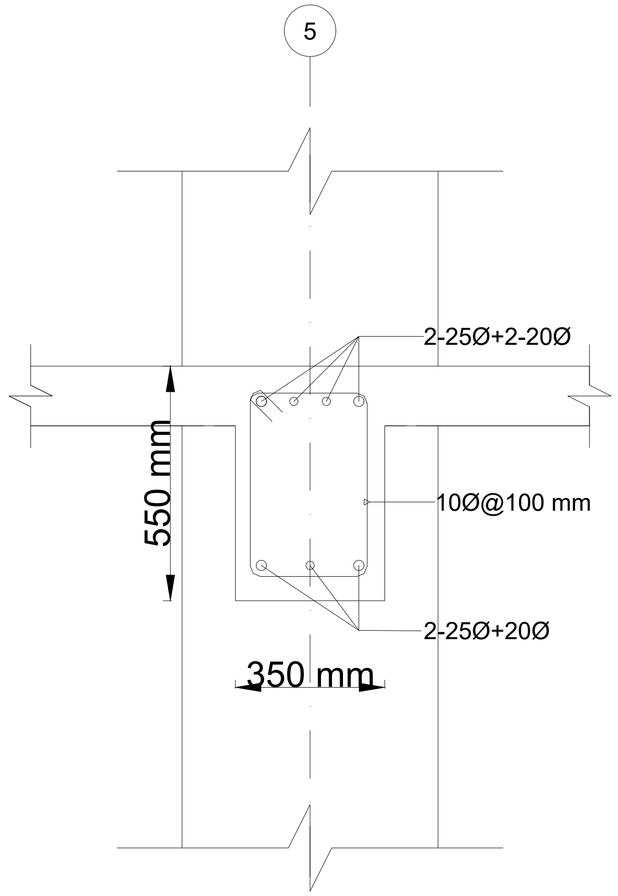
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 5-5 of Third Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 15



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF FOURTH FLOOR



7CB10:SECTION A



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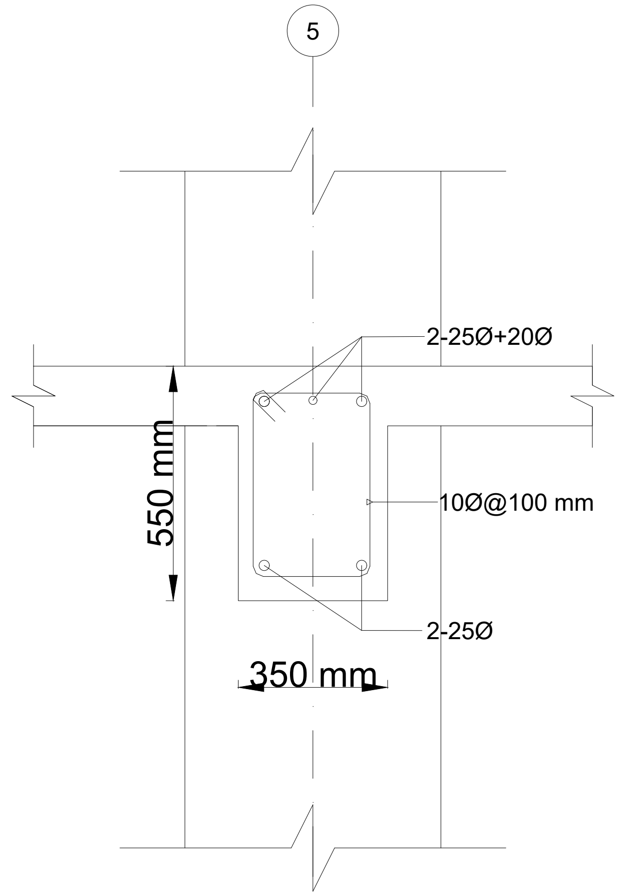

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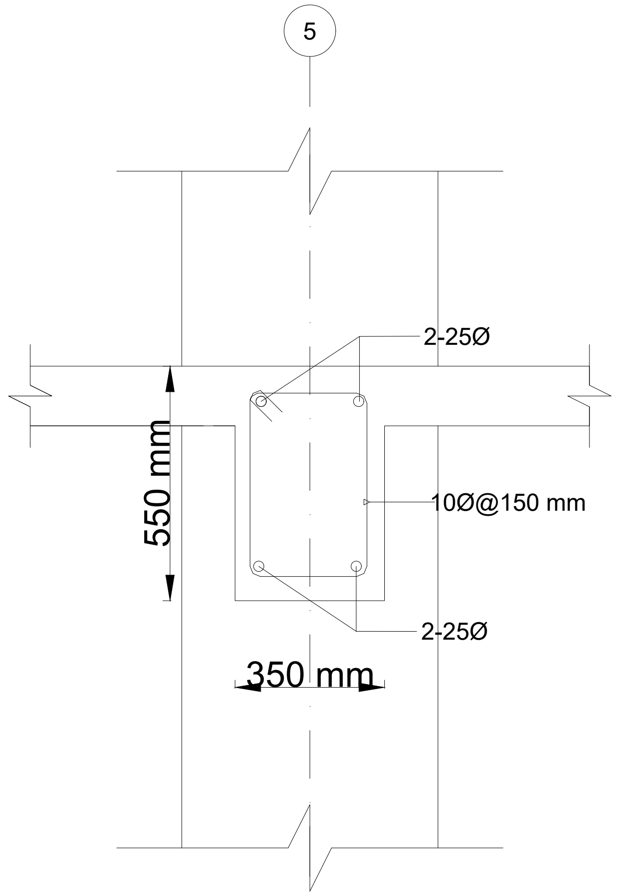
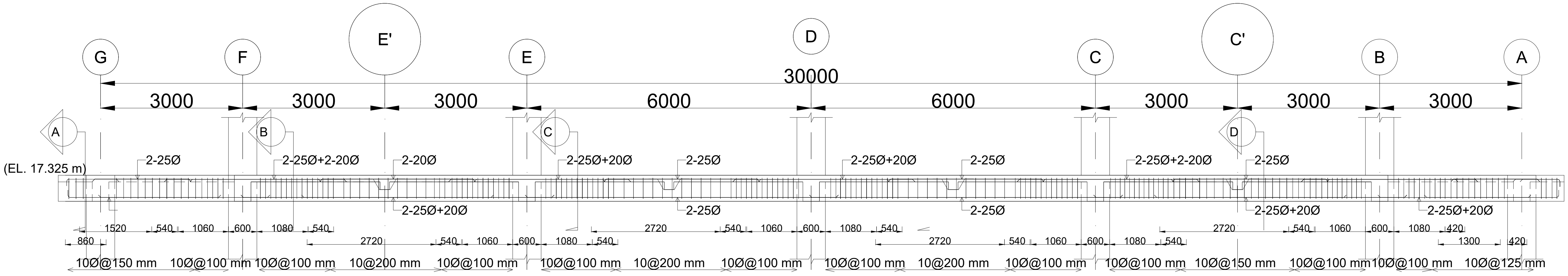


7CB10:SECTION D

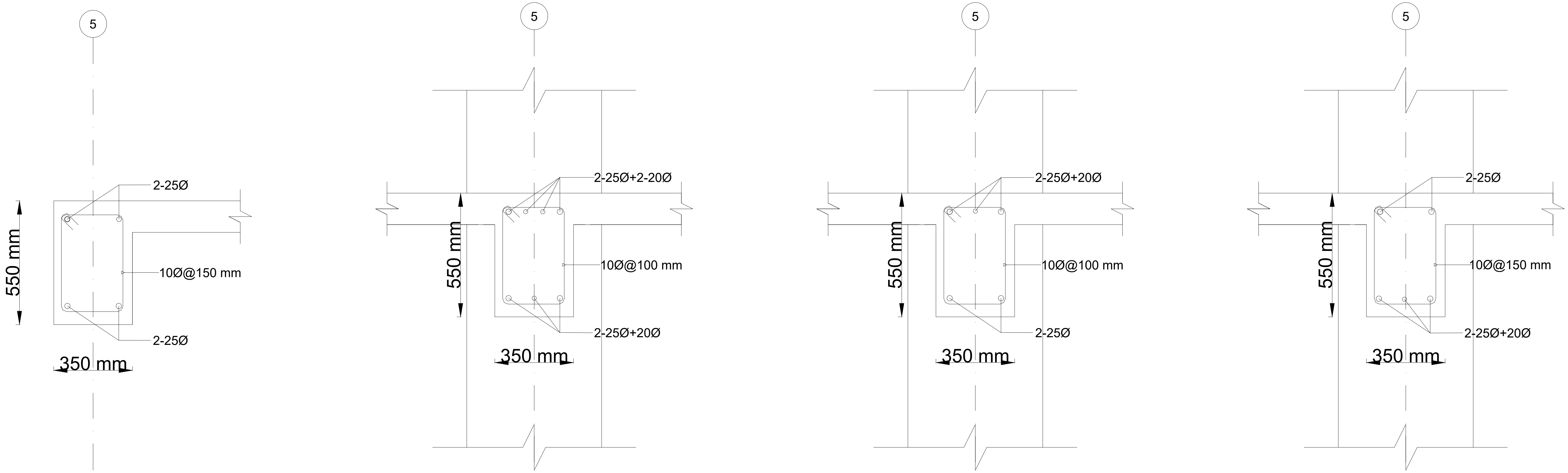
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF FOURTH FLOOR

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 5-5 of Fourth Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 16



REINFORCEMENT DETAILING OF BEAM ALONG 5-5 OF TOP FLOOR



7CB10:SECTION A

7CB10:SECTION B

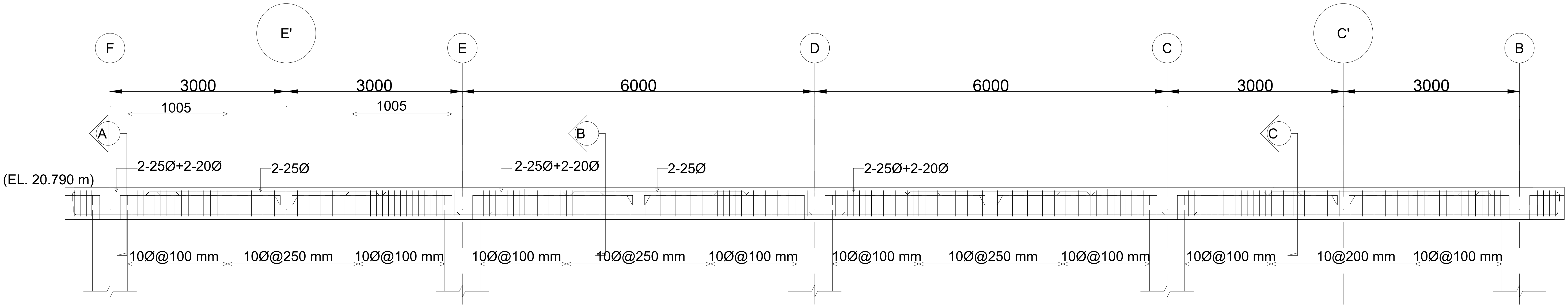
7CB10:SECTION C

7CB10:SECTION D

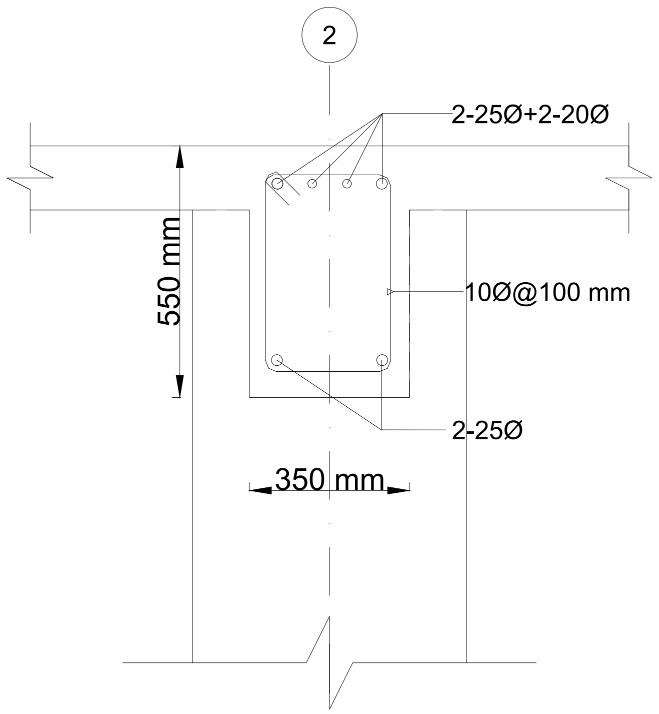
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 5-5 OF TOP FLOOR

ALL DIMENSIONS ARE IN MM

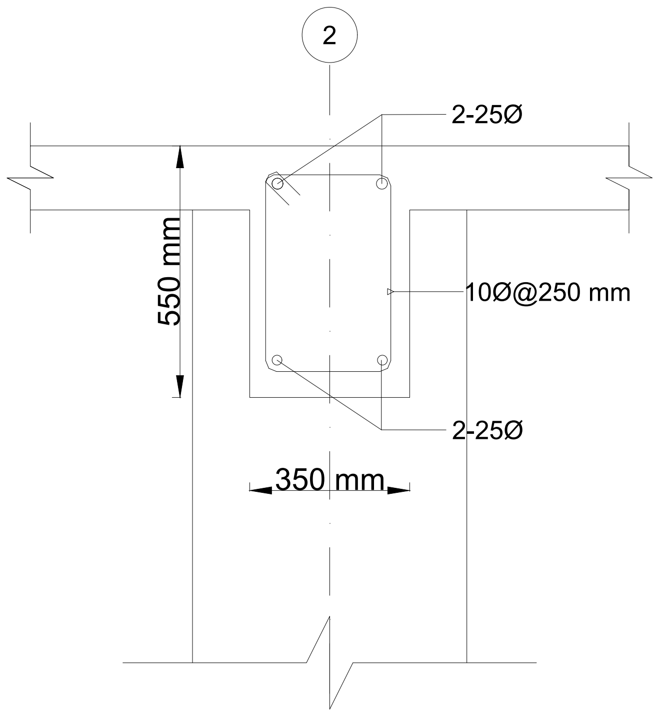
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 5-5 of Top Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 17



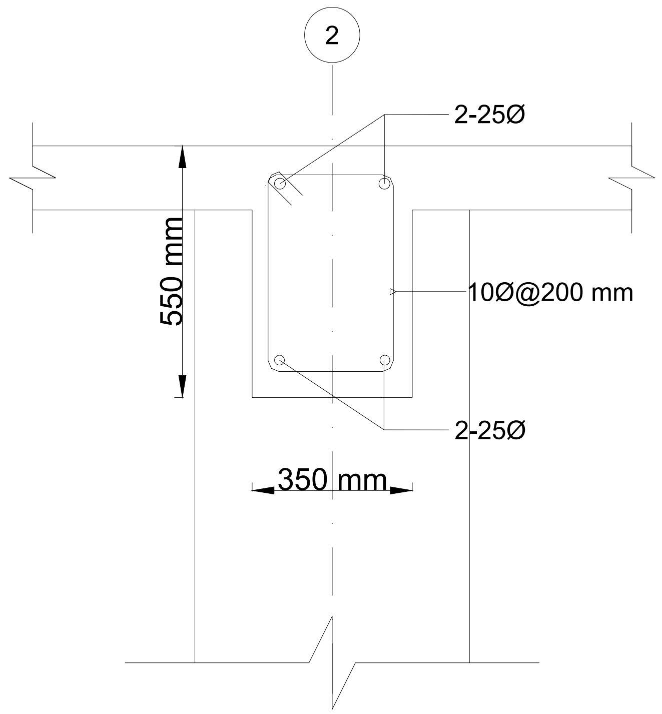
REINFORCEMENT DETAILING OF BEAM ALONG 2-2 OF ROOF FLOOR



8CB1:SECTION A



8CB1:SECTION B

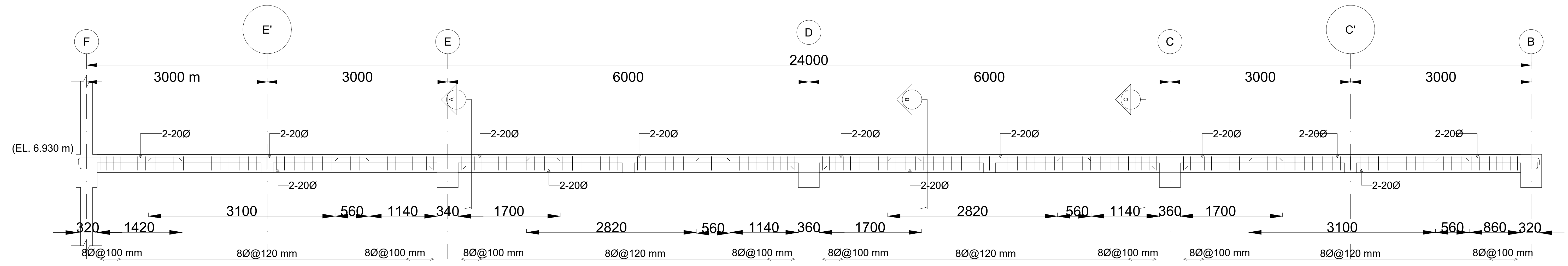


8CB1:SECTION C

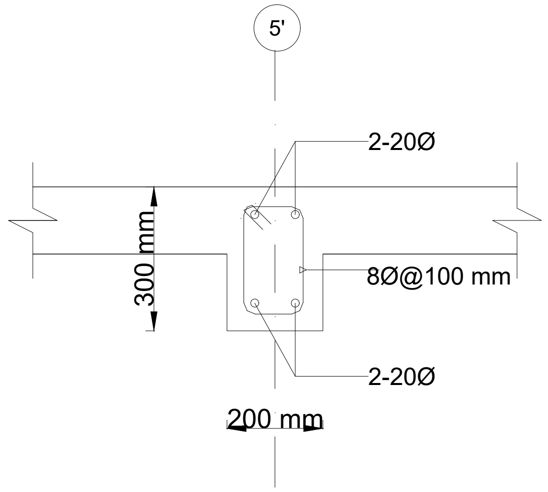
REINFORCEMENT DETAILING OF CROSS SECTION OF BEAM ALONG 2-2 OF ROOF FLOOR

ALL DIMENSIONS ARE IN MM

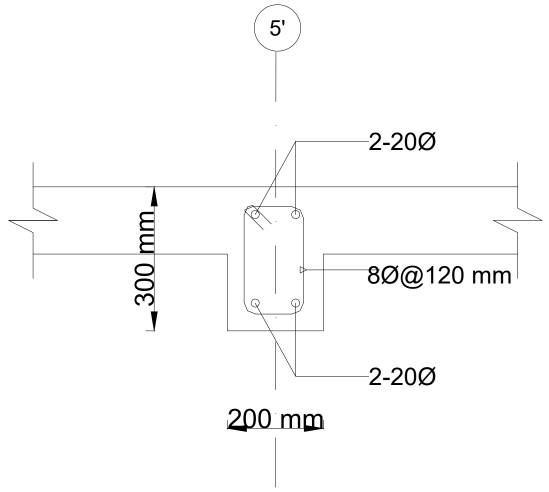
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Beam along 2-2 of Roof Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 18



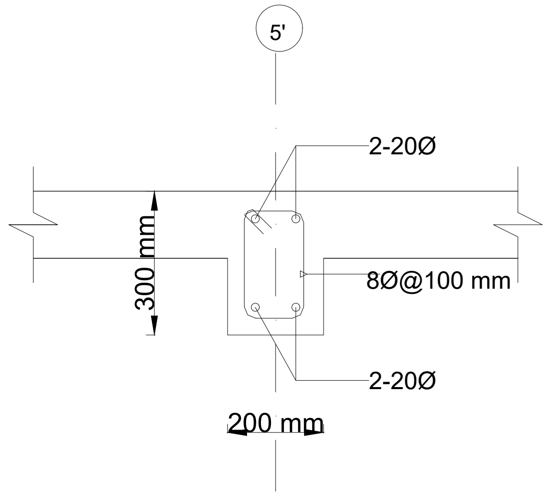
REINFORCEMENT DETAILING OF SECONDARY BEAM ALONG 5'-5' OF SECOND FLOOR



4CB19:SECTION A



4CB19:SECTION B

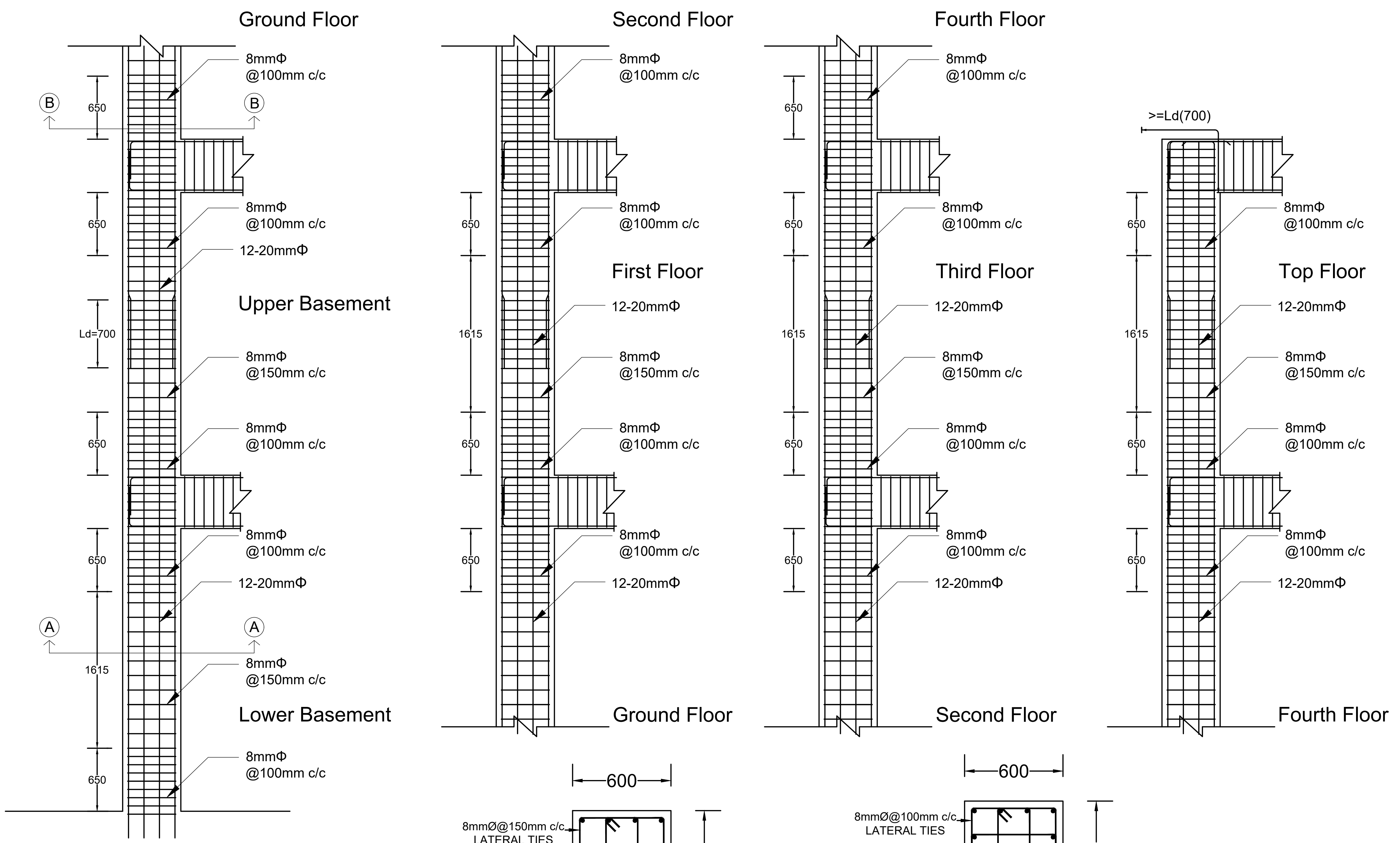


4CB19:SECTION C

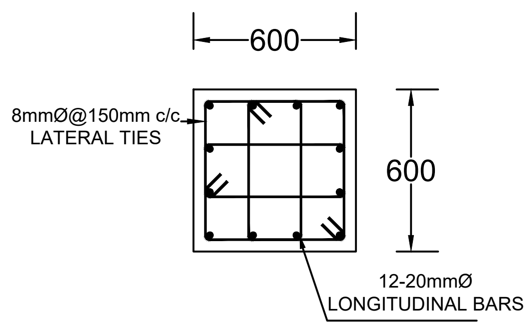
REINFORCEMENT DETAILING OF CROSS SECTION OF SECONDARY BEAM ALONG 5'-5' OF SECOND FLOOR

ALL DIMENSIONS ARE IN MM

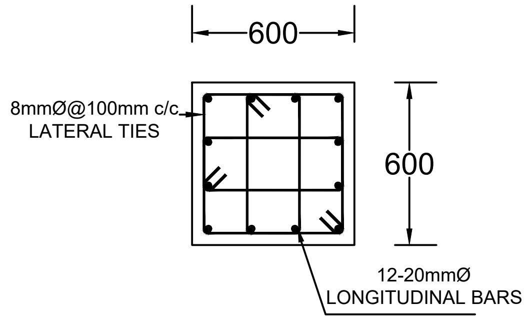
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Secondary Beam along 5'-5' of Second Floor**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 19



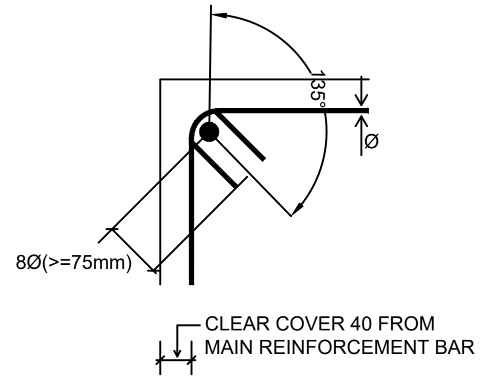
CORNER COLUMN



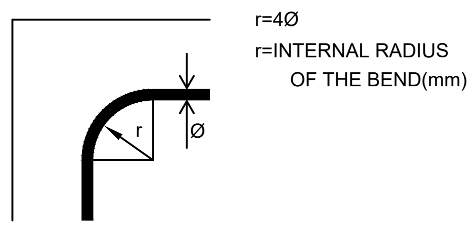
SECTION AT A-A



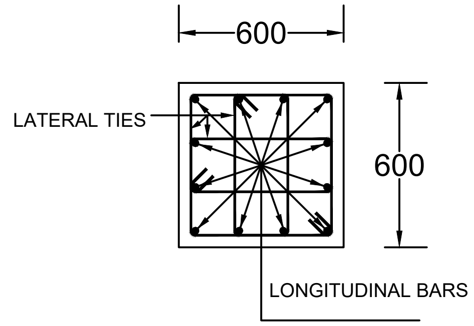
SECTION AT B-B
(FOR ALL ABOVE UPPER BASEMENT)



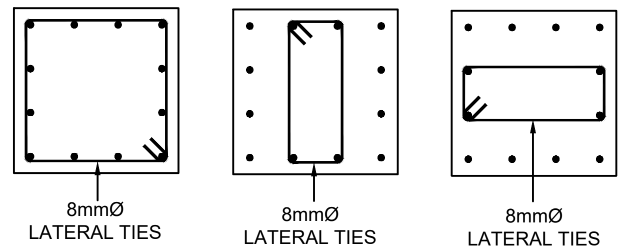
SEISMIC HOOK DETAIL



INTERNAL RADIUS OF BEND



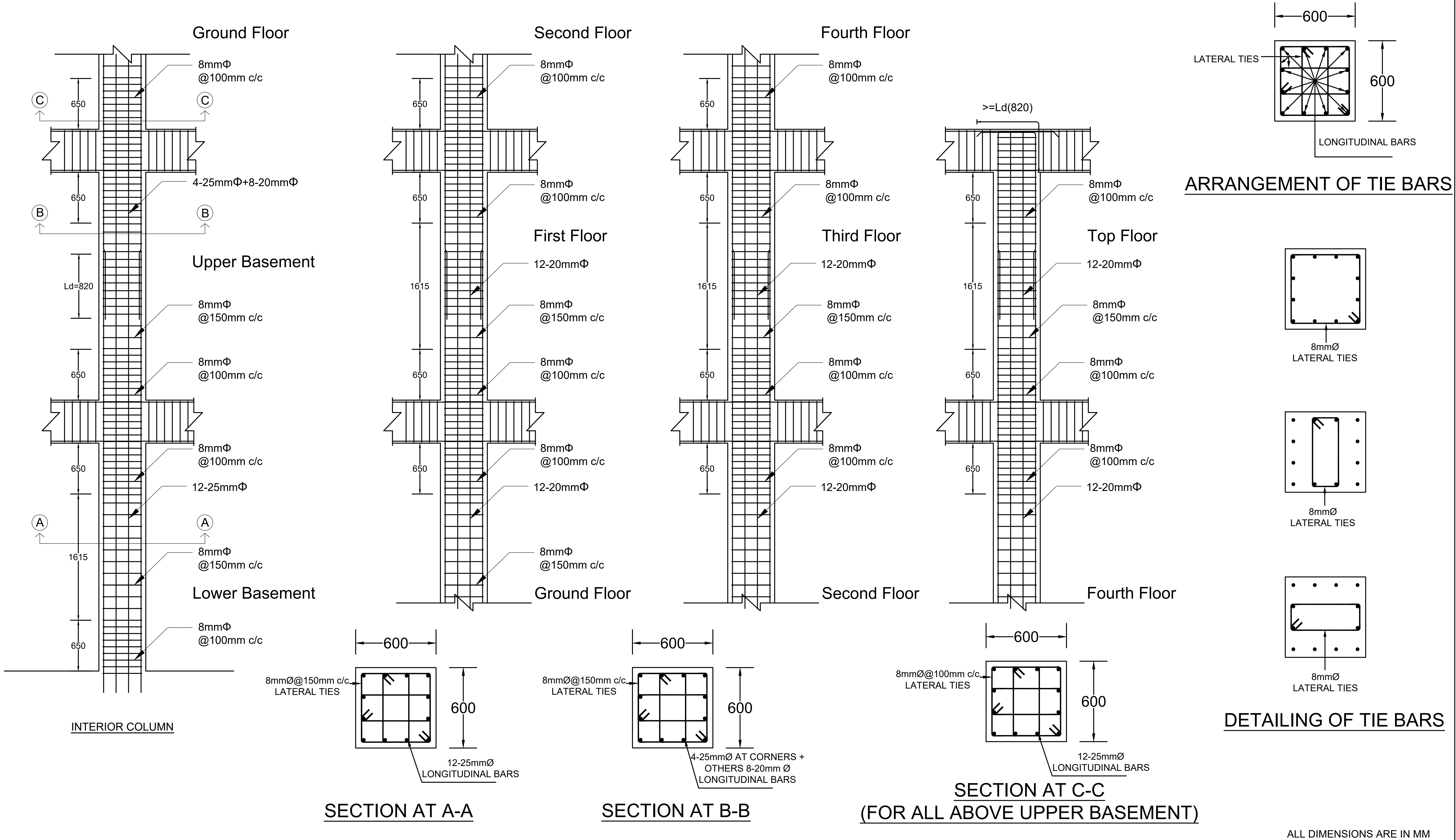
ARRANGEMENT OF TIE BARS



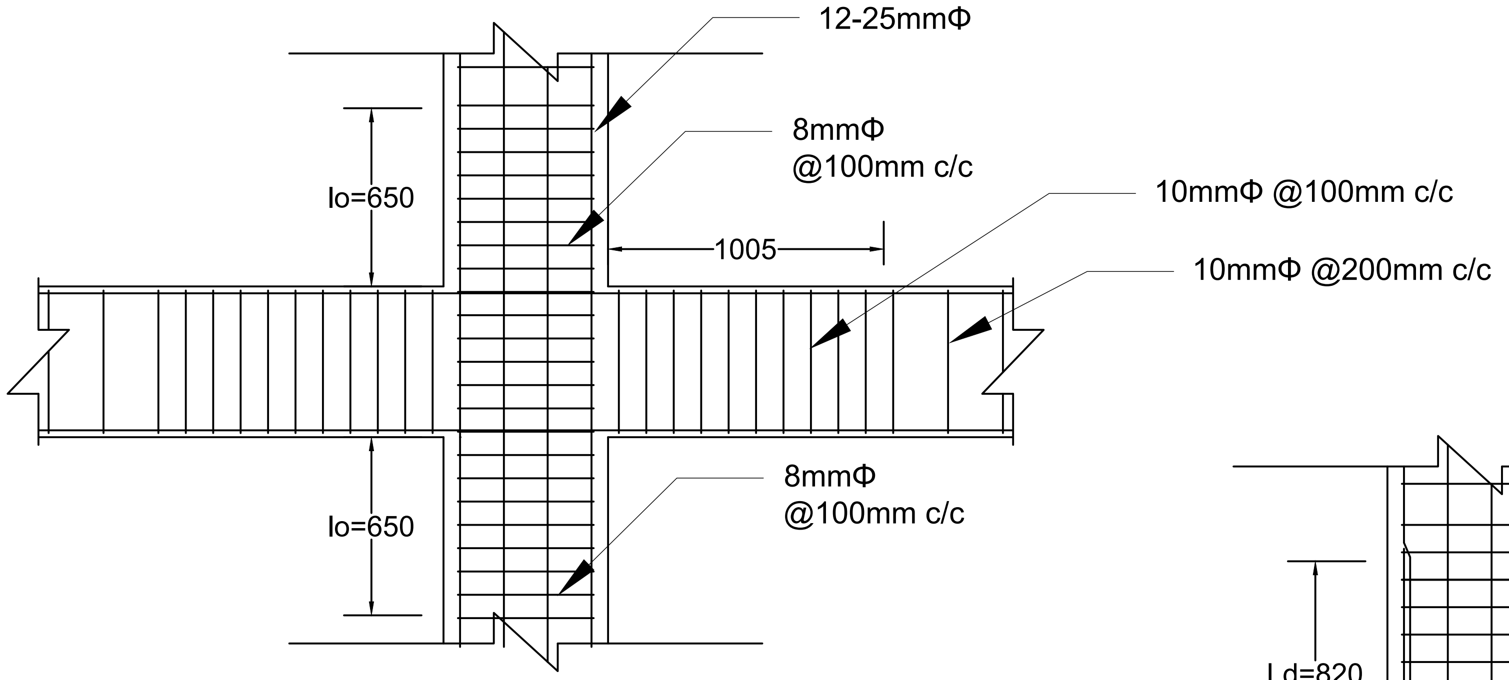
DETAILING OF TIE BARS

ALL DIMENSIONS ARE IN MM

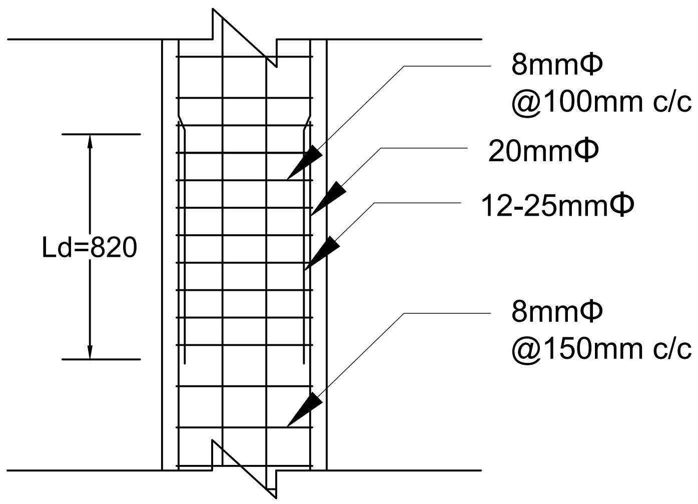
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Column**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 20



Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Column**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 21

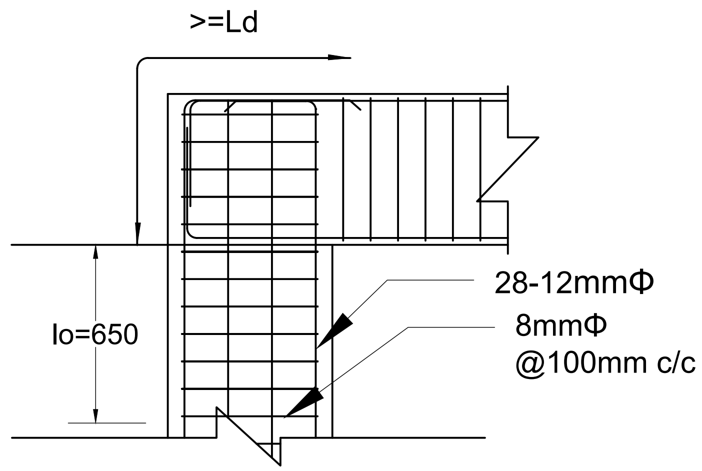


DETAILING OF BEAM-COLUMN JOINT

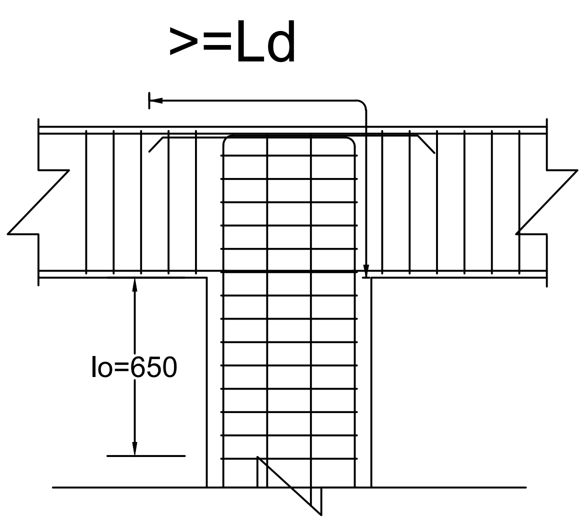


DETAILING OF SPLICING IN COLUMN

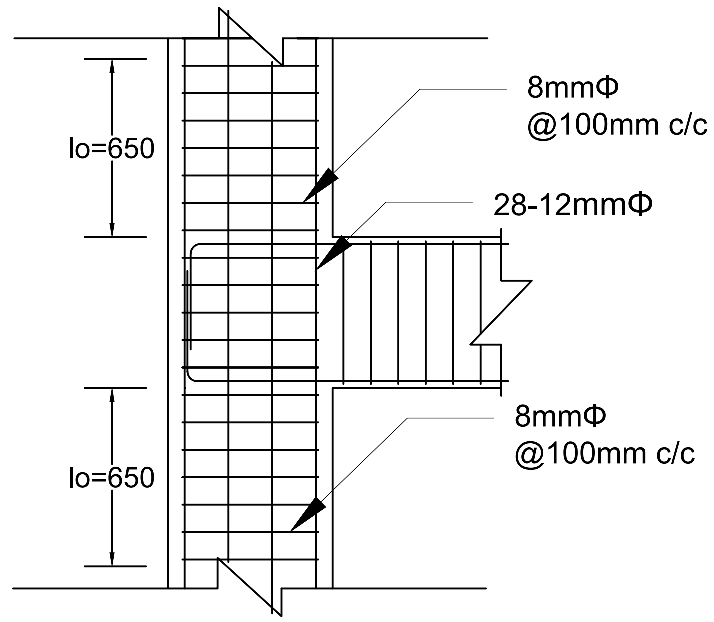
(MAXIMUM OF 50% OF BAR SHOULD BE SPLICED)



DETAILING OF BEAM-COLUMN JOINT AT THE TOP (CORNER)



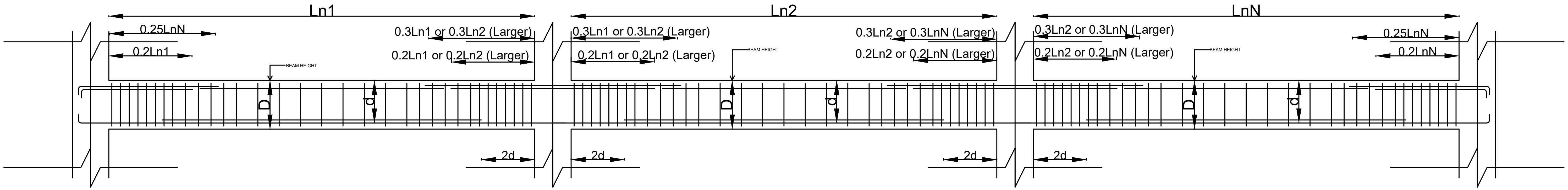
DETAILING OF BEAM-COLUMN JOINT AT THE TOP (INTERIOR)



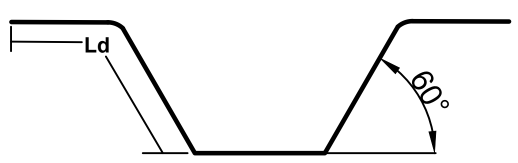
DETAILING OF BEAM-COLUMN JOINT AT THE ENDS (CORNER)

ALL DIMENSIONS ARE IN MM

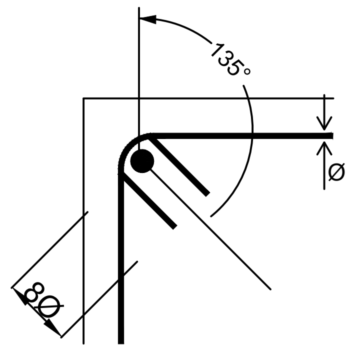
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing in Column**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 22



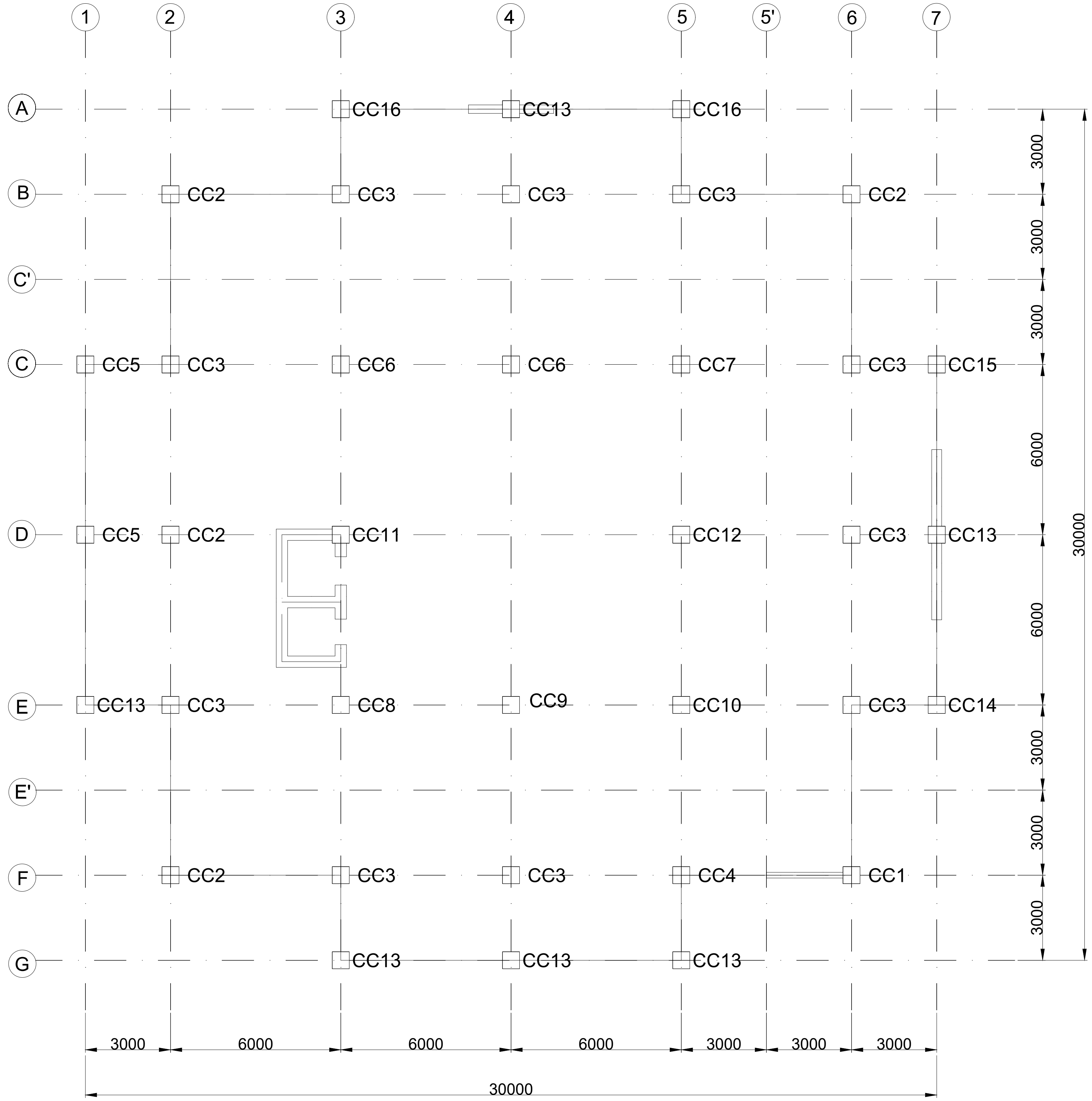
TYPICAL BEAM REINFORCEMENT DETAILING



BENT UP HANGER TYPE BARS



Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Typical Beam Elevation and Reinforcement Detail**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 23



CONCRETE COLUMN LAYOUT-LOWER BASEMENT (EL.-6930MM)

ALL DIMENSIONS ARE IN MM

Tribhuwan University
Institute of Engineering
Pulchowk Campus
Department of Civil Engineering

Project Title:
Study of Earthquake Resistant Analysis and Design of Multi-storey Building

Sheet Title:
Concrete Columns Layout Plan

Group Members:

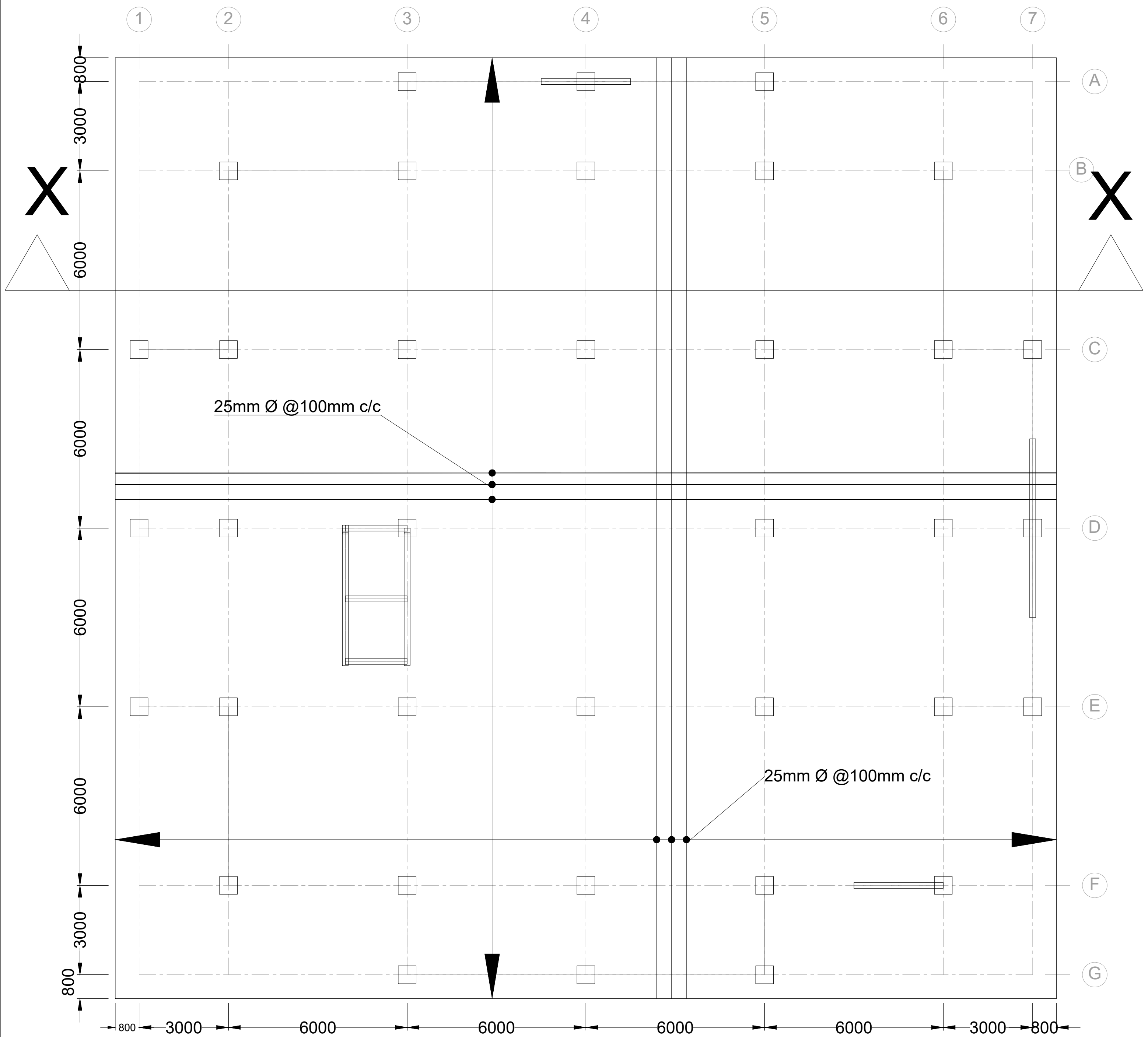
Santosh Pokharel	075BCE146
Saraswati Bhandari	075BCE147
Saroj Basnet	075BCE148
Saugat Dhakal	075BCE149
Shivam Kumar Sah	075BCE156
Uttam Dahal	075BCE189

Project Supervisor
Asst.Prof.Sunita Ghimire

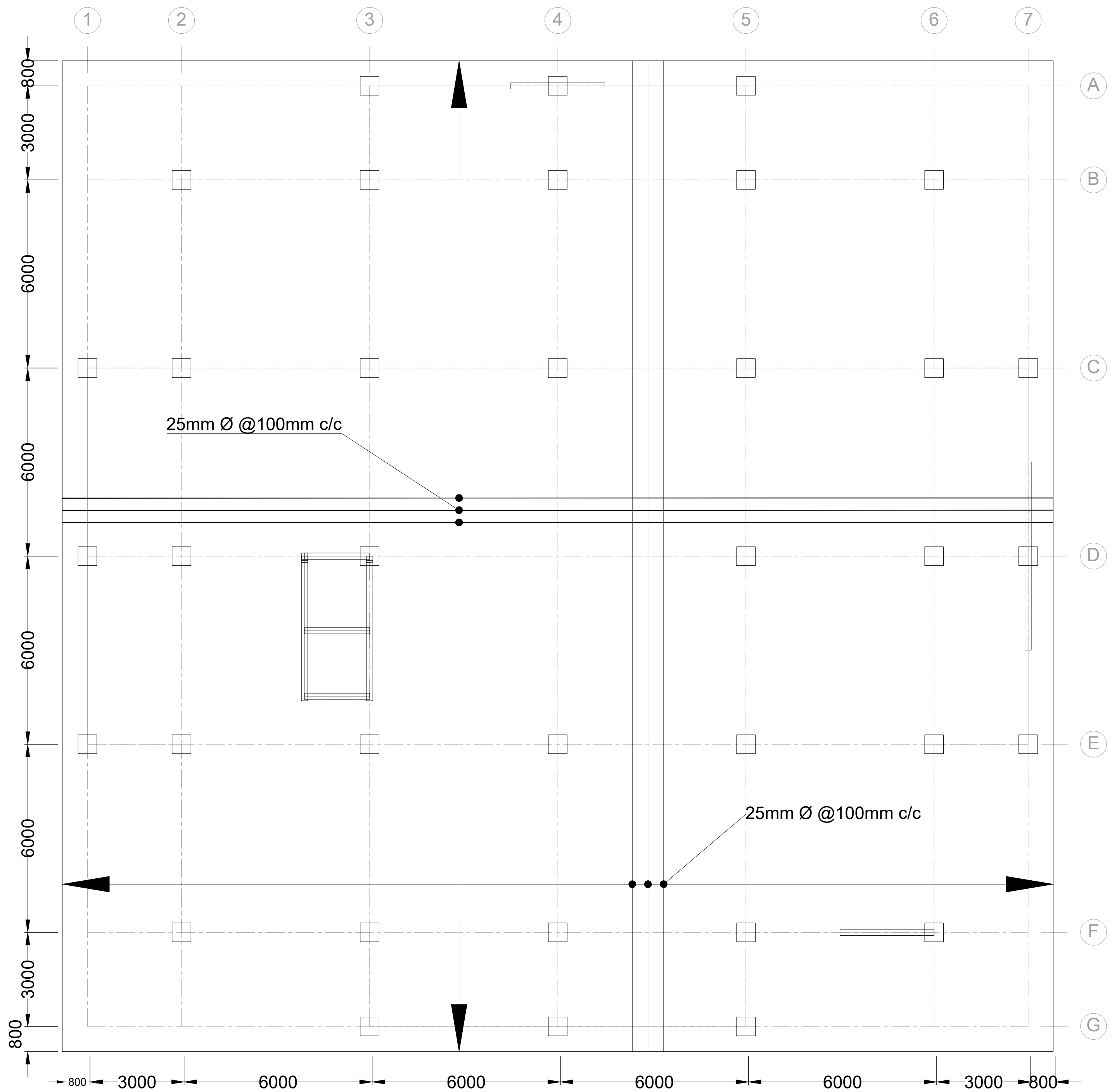
Checked By:

Scale: Fit to scale

Sheet No:
24



BOTTOM REINFORCEMENT (REGULAR)

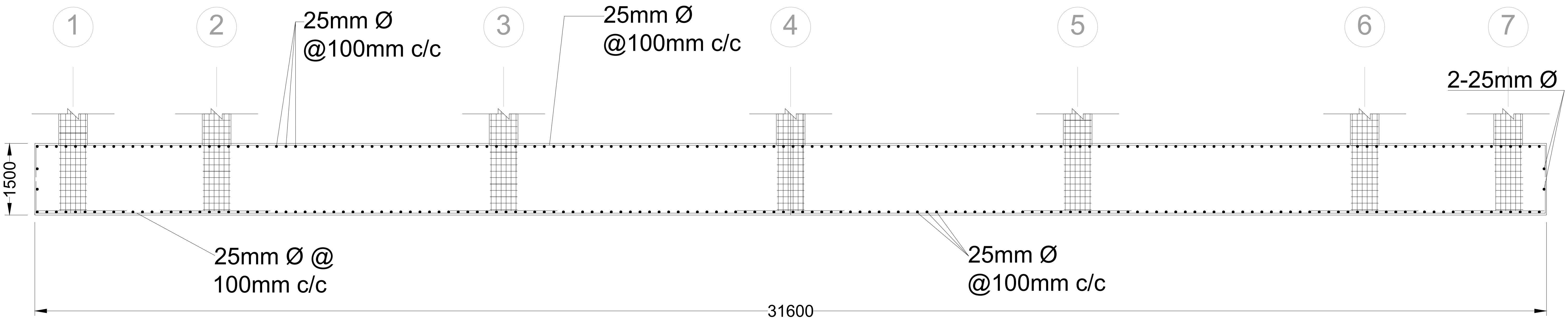


TOP REINFORCEMENT (REGULAR)

MAT FOUNDATION

ALL DIMENSIONS ARE IN MM

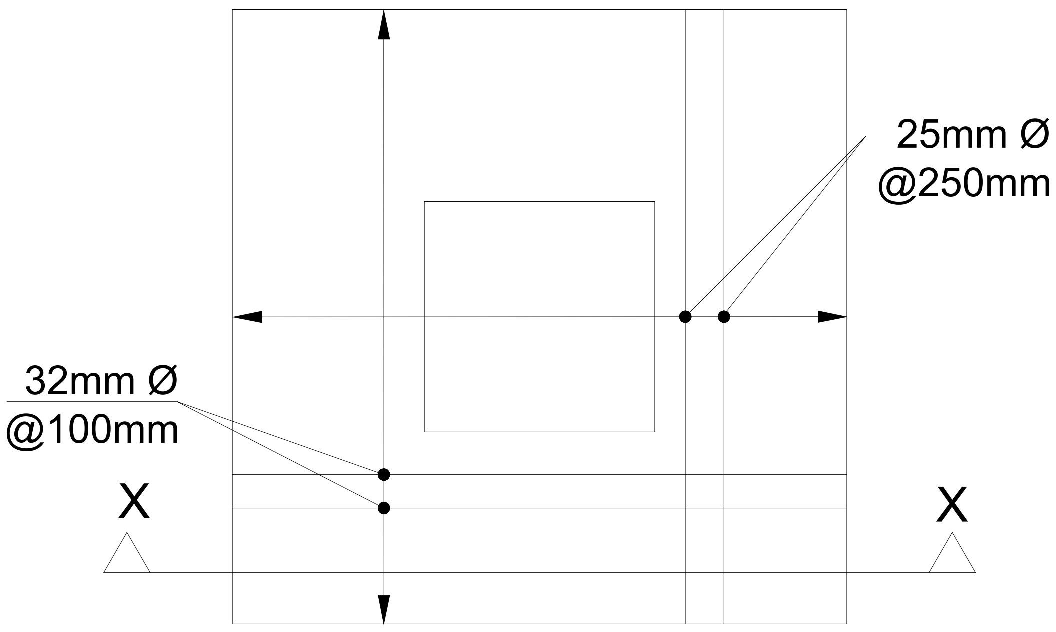
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Foundation**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 25



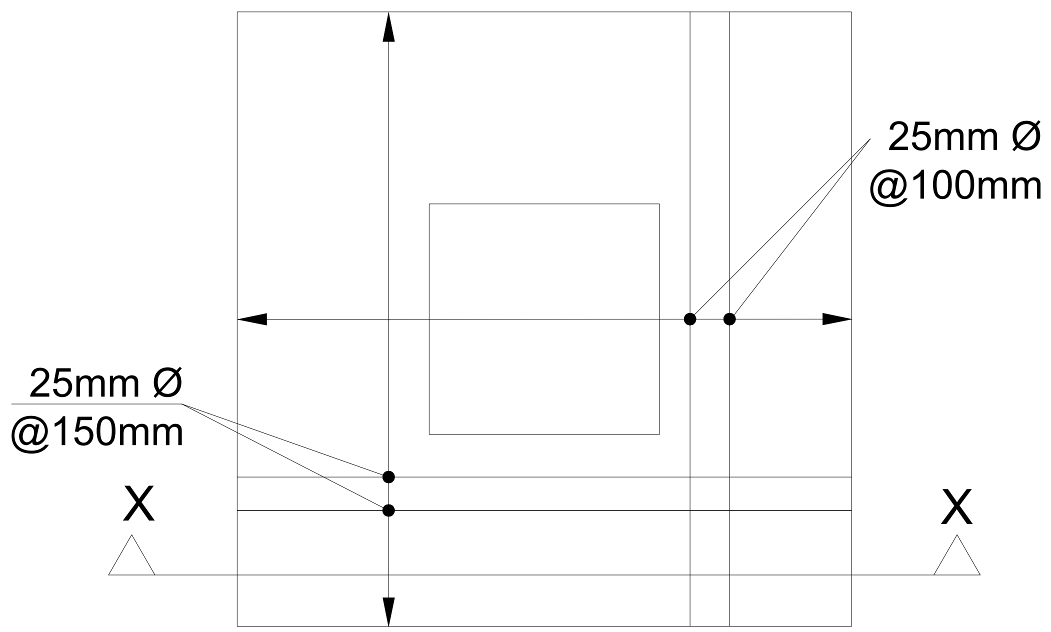
SECTION AT X-X

ALL DIMENSIONS ARE IN MM

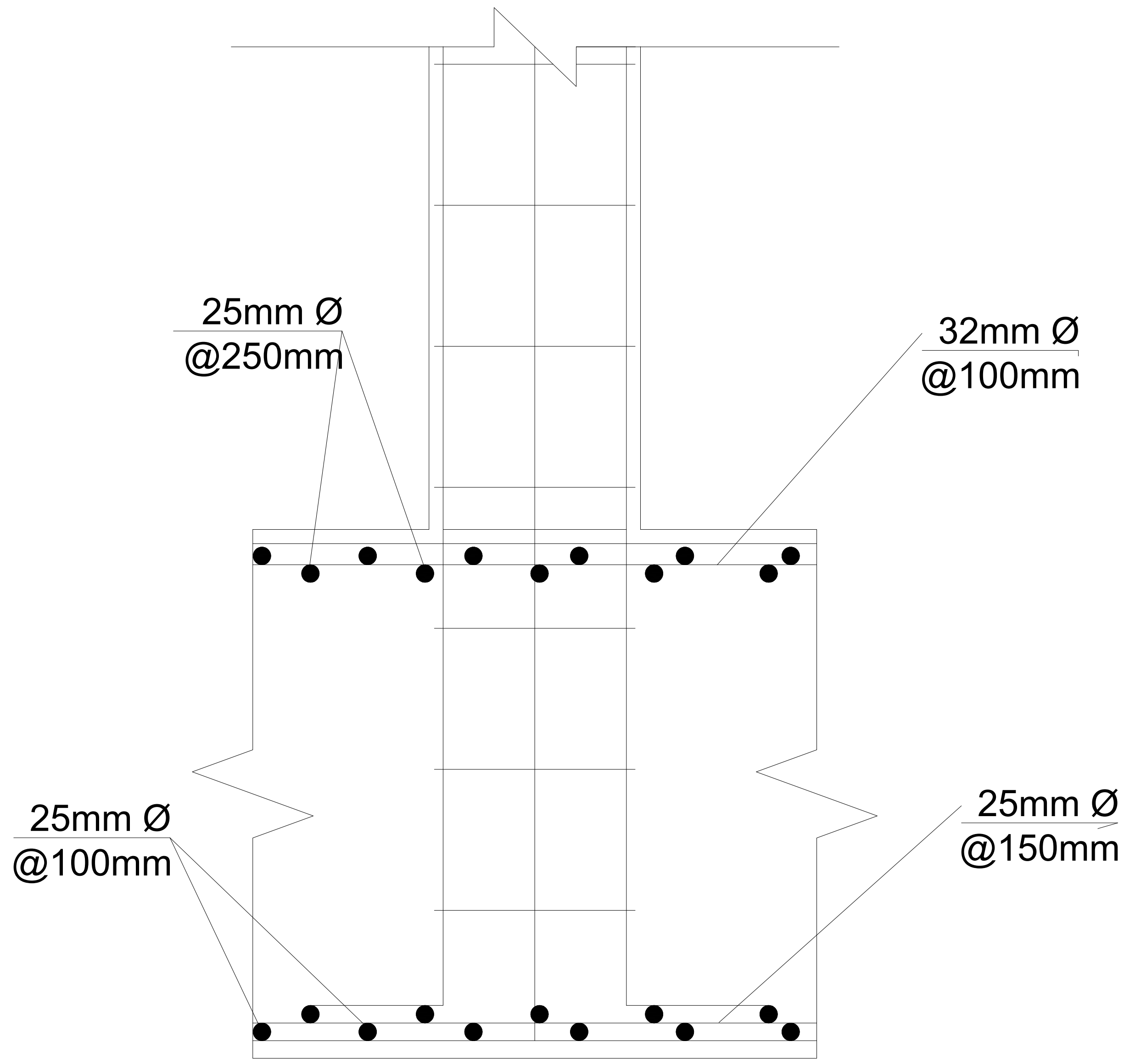
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Foundation**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 26



ADDITIONAL TOP REBAR (BELOW COLUMN/SHEAR WALL)



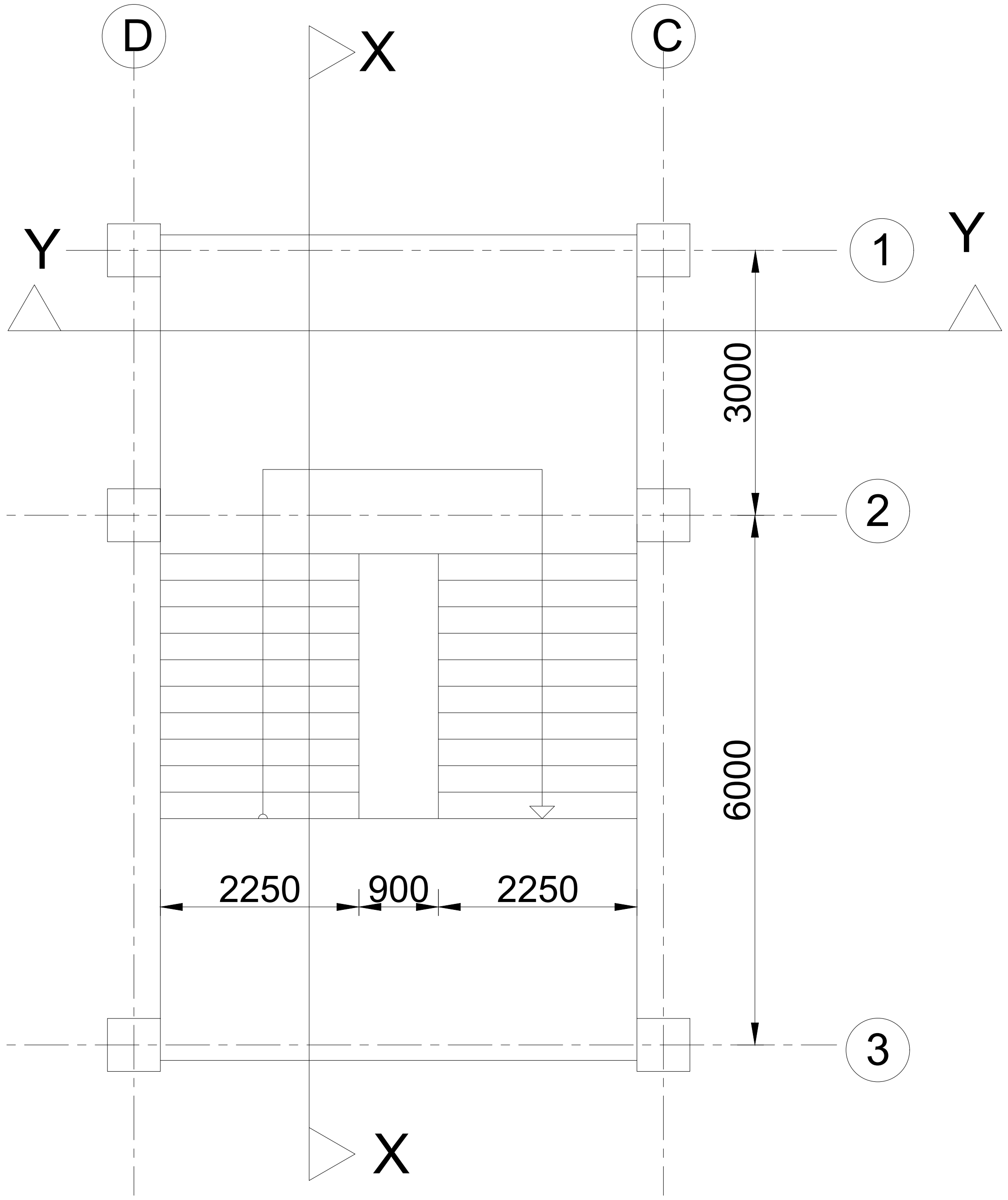
ADDITIONAL BOTTOM REBAR (BELOW COLUMN/SHEAR WALL)



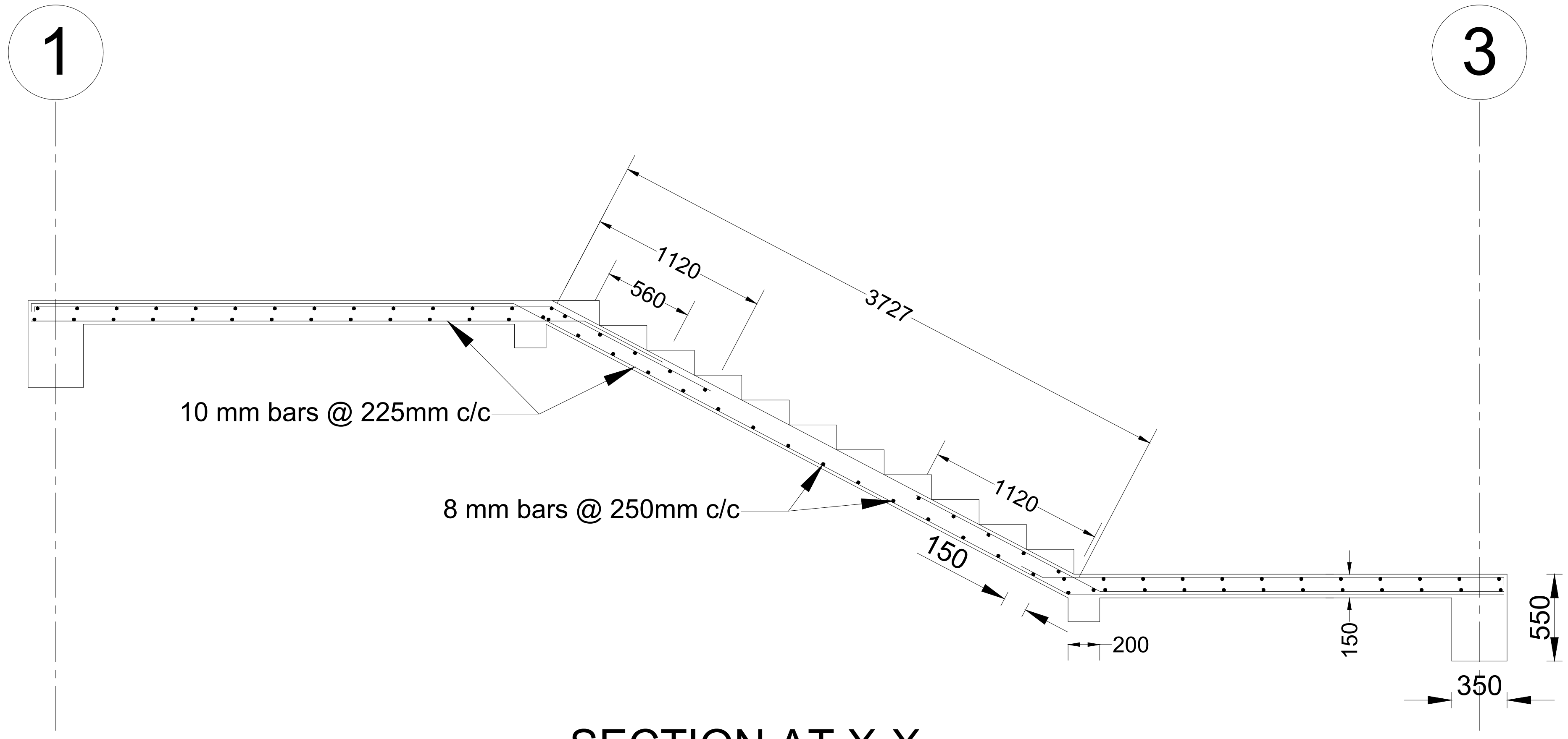
SECTION AT X-X

ALL DIMENSIONS ARE IN MM

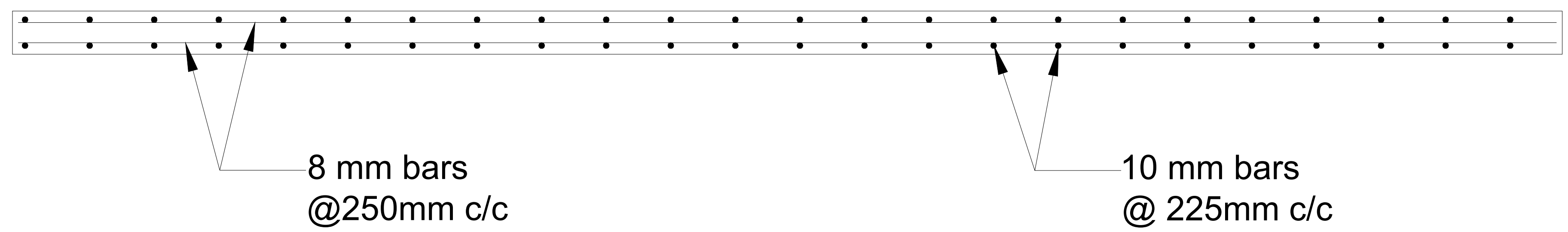
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Additional Reinforcement Detailing on Mat Foundation below Column/Shear Wall**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 27



STAIRCASE PLAN



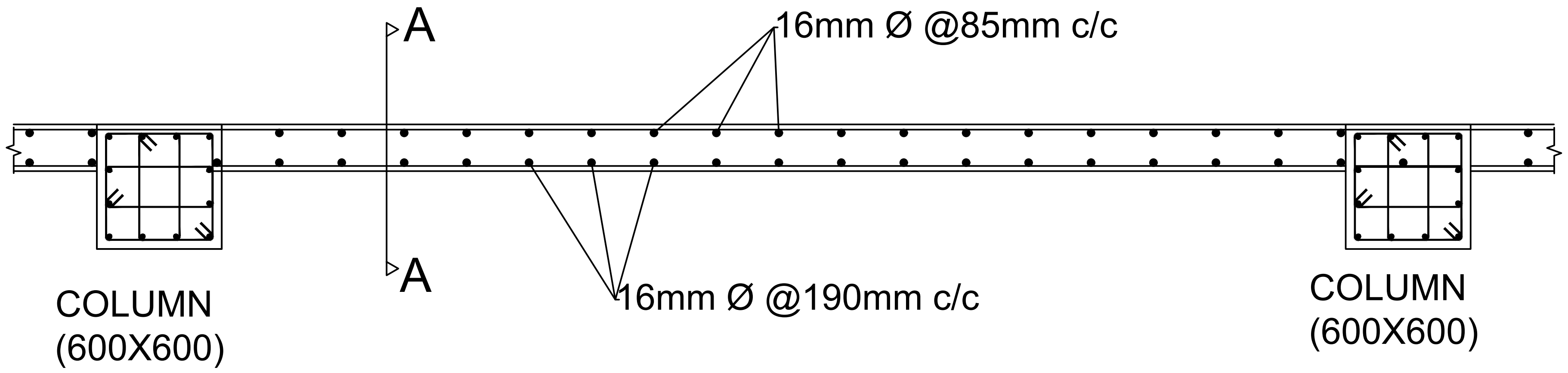
SECTION AT X-X



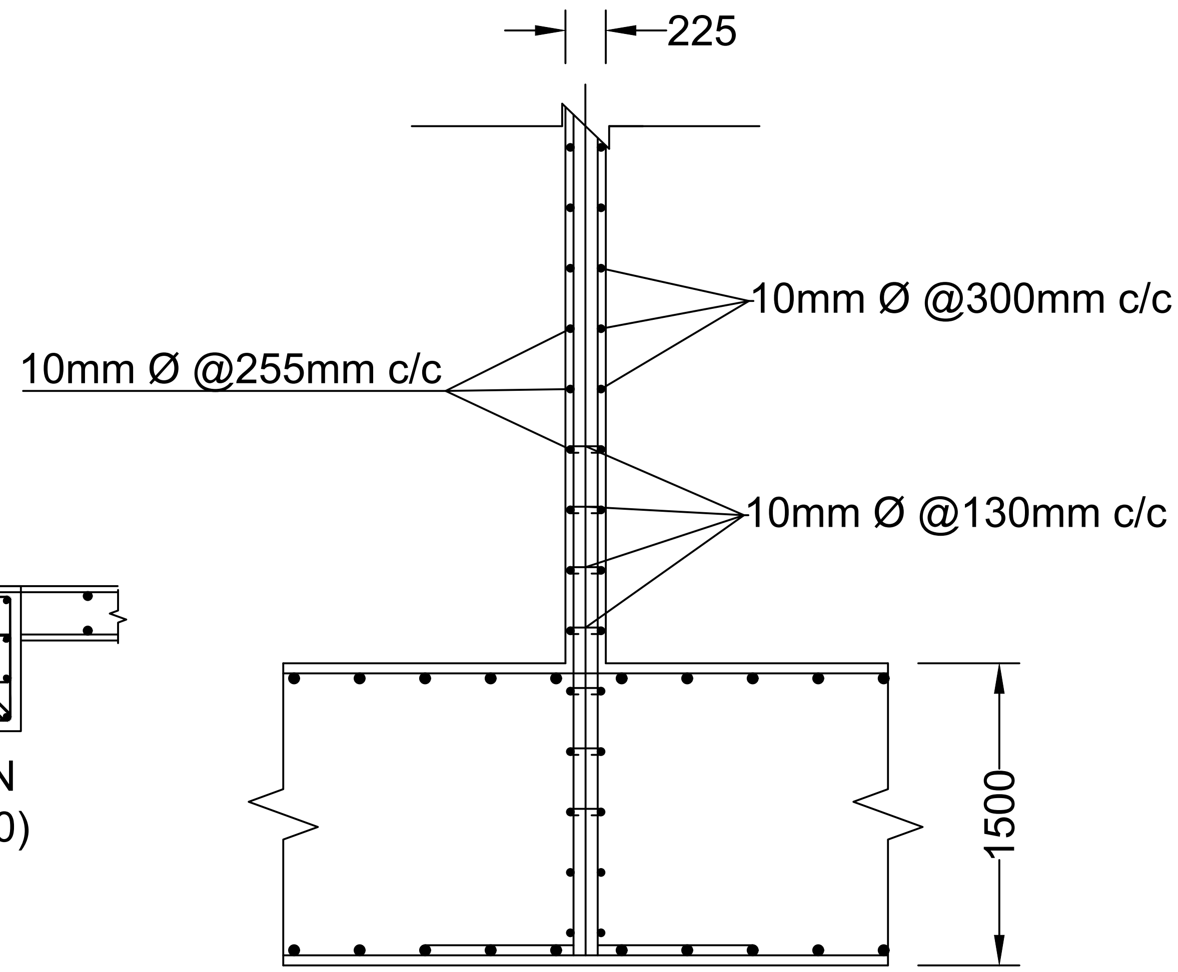
SECTION AT Y-Y

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Staircase**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 28



PLAN

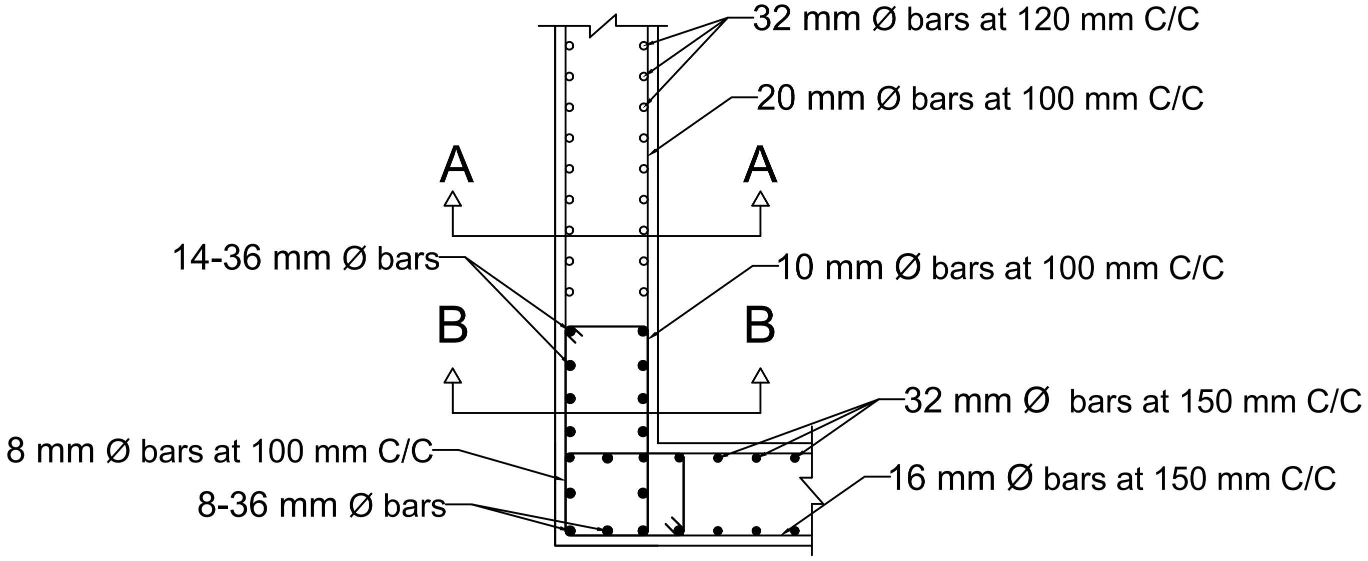


VERTICAL SECTION AT A-A

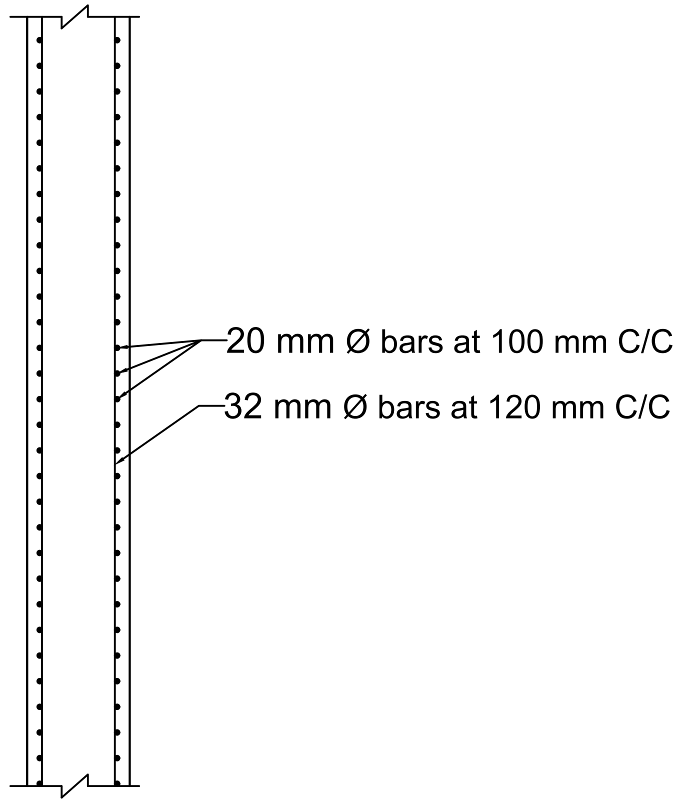
BASEMENT RETAINING WALL DETAILING

ALL DIMENSIONS ARE IN MM

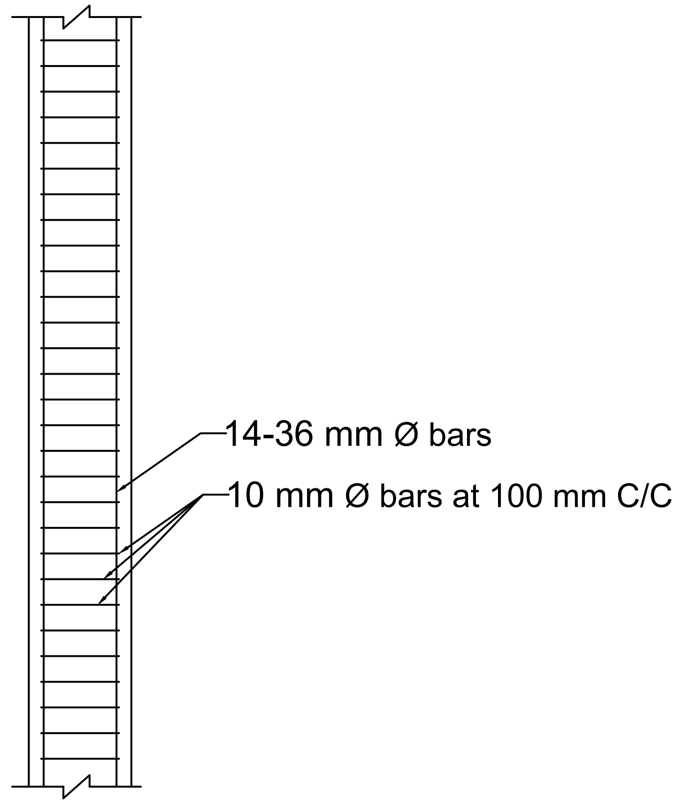
Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement Detailing of Basement Retaining Wall**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 29



REINFORCEMENT DETAILING PLAN



SECTION AT A-A



SECTION AT B-B

LIFT SHEAR WALL

ALL DIMENSIONS ARE IN MM

Tribhuwan University Institute of Engineering Pulchowk Campus Department of Civil Engineering	Project Title: **Study of Earthquake Resistant Analysis and Design of Multi-storey Building**	Sheet Title: **Reinforcement detailing of Lift Shear Wall**	Group Members: Santosh Pokharel 075BCE146 Saraswati Bhandari 075BCE147 Saroj Basnet 075BCE148 Saugat Dhakal 075BCE149 Shivam Kumar Sah 075BCE156 Uttam Dahal 075BCE189	Project Supervisor Asst.Prof.Sunita Ghimire Checked By:	Scale: Fit to scale Sheet No: 30